ANALYSIS AND TREATMENT OF INDEXES FOR DECISION-MAKING ABOUT ROAD PAVEMENT REHABILITATION UNDER SUSTAINABILITY CRITERIA: APPLICATION TO THE ROAD NETWORK OF BISCAY // ADIERAZLE ETA INDIZEEN ANALISIA ETA TRATAMENDUA ERREPIDEEN BIDE-ZORUEN BIRGAITZEARI BURUZKO ERABAKIAK HARTZEKO JASANGARRITASUNAREN IRIZPIDEEN ARABERA: APLIKAZIOA BIZKAIKO ERREPIDE-SAREAN

Ph.D. THESIS

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INGENIARITZA MEKANIKOA SAILA

DEPARTAMENTO DE INGENIERÍA MECÁNICA

À Ainhoa et à Oinatz,

on recupererà le temps perdu dans notre île de Ré

All models are wrong, but some are useful

(Box, 1979)

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En primer lugar, quiero agradecer a mis codirectores de tesis la ayuda prestada, tanto en las cuestiones técnicas propias de la tesis, como en las personales, motivándome para poder llevar a cabo este trabajo, incluyendo incluso un viaje conjunto a la fría Bialystok (Polonia). Tambien agradezco a Javier Canales las facilidades para realizar mis estancias en el extranjero.

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ABSTRACT

Due to the competition of funding among different sectors of the economy and society, Pavement Management Systems (PMS), defined as a systematic tool to manage, plan, assign budget, and programme all the pavement maintenance works, have become necessary for road agencies to desing more cost-effective and longer lasting pavements and to better allocate available funds. Pavement Management System consists of a database that must include the road inventory, the pavement materials, the traffic data and the present pavement condition data; an analysis module where available information is employed to predict the future condition of the pavements by means of performance models and a reporting module which produces reports to recommend the needed work and how to conduct it. Therefore, performance models are a key factor of any PMS.

The Regional Government of Biscay (RGB), due to its special political status in Spain, have managed since the 80s and is responsible of almost the entire interurban road network in the territory of Biscay, making a total of more than 1.300 km under its control. Aiming to efficiently manage its human and monetary resources, the Regional Government of Biscay has developed its own Pavement Management Systems, with a complete and exhaustive road inventory and a specific plan to introduce pavement structure information in the database. Moreover, the RGB conducted pavement data collection to know the real state of the entire road network in 2000, 2002 (partially), 2004, 2007, 2011 and 2016, including information about roughness (by means of the International Roughness Index, IRI), skid resistance, deflections and distresses, mainly cracking and rutting.

The aim of this PhD thesis is to develop performance models for IRI and the Side-force coefficient in order to predict the future condition of the road network of Biscay. IRI models are developed for new two-lane roads with flexible and semi-rigid pavements. For flexible pavements, influencing factors are the age of the pavement, the total thickness of bituminous layers and the accumulated number of heavy vehicles that circulated on the project lane. For semi-rigid pavements, the model requires the age of the pavement, the accumulated number of both heavy and total vehicles that passed over the pavement and a factor for considering the non bituminous materials. The proposed model for forecasting skid resistance can be deployed in any type of road, two-lane roads or double carriageway roads, and for all the usual materials employed in surface layers in Biscay: asphalt concrete, discontinuous mixes, porous asphalts and slurries. The model has a short version which includes as variables the Annual Average Daily Traffic (AADT) of the year of the data collection, a factor which considers the type of surface material and a factor for considering the type of road and the number of lanes per direction. Apart from these variables, the long version of the model also includes a factor that combines the AADT, the type of road and number of lanes, the surface material and the required Polished Stone Value according to Spanish regulations. The introduction of the Annual Average Daily Traffic of all the traffic volume and not the AADT of the heavy traffic represents a difference with the previously proposed models.

LABURPENA

Ekonomiaren eta gizartearen hainbat sektoreren arteko finantzaketa lortzeko lehia dela eta, Bide-zoruak Kudeatzeko Sistemak (BKS) beharrezko bihurtu dira bide-administrazioetarako luzeago irauten duten eta kostu eraginkorragoko bide-zoruak diseinatzeko eta dauden funtsak hobeto esleitzeko. Bide-zoruaren mantentze-lan guztiak kudeatzeko, planifikatzeko eta aurrekontua esleitzeko tresna sistematiko bezala defini daiteke Bide-zoruak Kudeatzeko Sistema. BKSek honako elementu hauek izan behar ditu: datu-base bat, errepide-inbentarioa, bide-zoruen materialak, trafiko datuak eta bide-zoruaren gaurko egoeraren datuak eduki behar dituena; analisi-modulu bat non informazio eskuragarria erabiltzen den bide-zoruen etorkizuneko egoera iragartzeko portaera-modeloen bitartez; eta informazio-modulu bat, beharrezko lanak eta nola gauzatu azaltzen duten txostenak prestatzeko. Horren ondorioz, portaera-modeloak oinarrizko faktorea dira edozein BKStan.

Bizkaiko Foru Aldundiak (BFA), Espainian daukan estatus politiko berezia dela eta, Bizkaiko lurraldeko hiri arteko ia errepide-sare osoa kudeatu du 80ko hamarkadatik eta horren arduraduna da. Beraz, 1.300 km baino gehiago daude bere kontrolpean. Bere giza- eta diru-baliabideak hobeto kudeatu nahian, Bizkaiko Foru Aldundiak bere Bide-zoruak Kudeatzeko Sistema propioa garatu du, errepide-inbentario zehatz batekin eta datu-basean bide-zoruen informazioa sartzeko plan espezifiko batekin. Gainera, BFAk datu-biltze kanpainak burutu zituen errepide-sare osoaren benetako egoera jakiteko 2000, 2002 (partzialki), 2004, 2007, 2011 eta 2016. urteetan honako datu hauek neurtuz: erregulartasuna, International Roughness Index (IRI) indizearen bidez, labainketarekiko erresistentzia, deflexioak eta akatsak, bereziki pitzadurak eta gurpil-arrastoak.

Tesi honen helburua IRI eta zeharkako marruskadura koefizienterako portaera-modeloak garatzea da, Bizkaiko errepide-sarean etorkizuneko egoera jakiteko asmoz. IRI modeloak garatu dira errepide konbentzional berrietarako (bi erreiko errepideak) bide-zoru malgu eta erdi-zurrunarekin. Bide-zoru malguentzat, eragina duten faktoreak dira honako hauek: bide-zoruaren adina, geruza bituminosoen lodiera totala eta proiektuko erretik pasatu diren metatutako ibilgailu astunen kopurua. Bide-zoru malguetarako, modeloak behar du bide-zoruaren adina, bide-zorutik pasatu diren metatutako trafiko totala eta astunen trafiko totala eta material ez bituminosoak kontuan hartzen dituen faktore bat. Labainketarekiko erresistentziarako proposatutako modeloa edozein errepide motarako erabil daiteke, errepide konbentzionaletan edo galtzada bananduko errepideetan, eta Bizkaian normalean erabiltzen diren errodadura geruzen materialetarako: hormigoi bituminosoa, nahaste etenak, nahaste drainatzaileak eta kare-esneak. Modeloak bertsio laburra dauka eta erabiltzen ditu aldagai hauek: datuak neurtzen diren Eguneko Batez Besteko Intentsitatea (EBBI), errodadura geruzaren materiala kontuan hartzen duen faktorea eta errepide-mota eta errei kopurua noranzkoko kontuan hartzeko faktore bat. Aldagai hauetaz gain, modeloaren bertsio luzeak beste faktore bat erabiltzen du, EBBIa, errepide-mota eta errei kopurua noranzkoko, errodadura geruzaren materiala eta Espainiako arauen araberako beharrezko Azeleratutako Leuntze Koefizientea konbinatzen dituena. Eguneko Batez Besteko Intentsitate totala erabiltzeak, eta ez ibilgailu astunen Eguneko Batez Besteko Intentsitatea, aurretik proposatutako modeloekin ezberdintasuna dakar.

RESUMEN

Debido a la competencia por la financiación entre los distintos sectores de la economía y de la sociedad, los Sistemas de Gestión de Firmes (SGF), definidos como una herramienta sistemática para gestionar, planificar, asignar presupuesto y programar todas los trabajos de mantenimiento de firmes, se han convertido en necesarios para las administraciones viarias para diseñar firmes más efectivos con relación al coste y de mayor duración y para mejor asignar los fondos disponibles. Los Sistemas de Gestión de Firmes consisten en una base de datos, que debe incluir un inventario de las carreteras, los materiales del firmes, los datos de volumen de tráficos y datos del estado del firme; un módulo de análisis donde la información disponible es utilizada para predecir el estado futuro de los firmes mediante los modelos de comportamiento; y un módulo de información que produce informes para recomendar los trabajos necesarios y cómo llevarlos a cabo. Por tanto, los modelos de comportamiento son un factor clave de cualquier SGF.

La Diputación Foral de Bizkaia (DFB), debido a su especial status político en España, gestiona y es responsable desde los años 80 de casi toda la red de carreteras interurbana en el territorio de Bizkaia, haciendo un total de más de 1.300 km bajo su control. Deseando gestionar eficazmente sus recursos humanos y económicos, la Diputación Foral de Bizkaia ha desarrollado su propio Sistema de Gestión de Firmes, con un completo y exhaustivo inventario de carreteras y un plan específico para introducir información de la estructura del firme en la base de datos. Además, la DFB ha llevado a cabo campañas de auscultación para conocer el estado real de la red viaria completa en el 2000, 2002 (parcialmente), 2004, 2007, 2011 y 2016, incluyendo información sobre la regularidad (por medio del International Roughness Index (IRI), la resistencia al deslizamiento, deflexiones y defectos, principalmente grietas y roderas.

El objetivo de esta tesis doctoral es desarrollar modelos de comportamiento para el IRI y el Coeficiente de Rozamiento Transversal para poder predecir el estado futuro de la red viaria de Bizkaia. Los modelos de IRI se desarrollan para nuevas carreteras convencionales de dos carriles con firmes flexibles y semirrígidos. Para los firmes flexibles, los factores que influyen son la edad del firme, el espesor total de las capas bituminosas y el total acumulado de vehículos pesados que ha circulado por el carril de proyecto. Para los firmes semirrígidos, el modelo requiere la edad del firme, el total acumulado de vehículos pesados y totales que han pasado sobre el firme y un factor para tener en cuenta los materiales no bituminosos. El modelo propuesto para prever la resistencia al deslizamiento puede ser utilizado en cualquier tipo de carretera, convencional de 2 carriles o de soble calzada y para todo los materiales habituales en las capas de rodadura en Bizkaia: hormigón bituminoso, mezclas discontinuas, mezclas porosas y lechadas bituminosas. El modelo tiene una versión corta que incluye como variables la Intensidad Media Diaria (IMD) del año auscultado, un factor que tiene en cuenta el tipo de material de la capa de rodadura y un factor para considerar el tipo de carretera y el número de carriles por sentido. Además de estas variables, la versión larga incluye un factor que combina la IMD, el tipo de carretera y el número de carriles, el material de la capa de rodadura y el Coeficiente de Pulimiento Acelerado requerido en las especificaciones españolas (PG-3). La introducción de la Intensidad Media Diaria del tráfico total y no la IMD del tráfico pesado representa una diferencia con los modelos anteriormente propuestos

PART I. STATE-OF-THE-ART

Chapter 1. Introduction

1.1. Background

The road network of a country represents one of the pillars of its economy and implies a huge investment in money for assuring a safe and efficient displacement of people and goods. Hence, economically feasible, technically proper decisions on design, construction, rehabilitation and maintenance of road pavements are vital for maintaining the road network in an acceptable condition. Moreover, the relative size and the physical condition of the highway network of a country are said to contribute enormously to the economy prosperity, since the roads are the most important component of the transport infrastructure of any country (Hudson *et al*., 1997). In developed countries, paved road density, expressed in km per million inhabitants, is higher than in developing countries, which could be observed as an important economic indicator.

Furthermore, with regard to investments in roads, it can be observed that the new construction and upgrading of road projects consume most road funds in many developing countries, whereas in developed countries, more funds are allocated to maintenance, rehabilitation and preservation of the existing pavement highways. Therefore, nowadays, the road agencies of the most developed countries changed their main tasks from projecting and constructing new infrastructures to managing and maintaining existing network (Xiong *et al*., 2012). For instance, real spending for highway investments in USA was $91 billion in 2013 (ASCE, 2013).

Due to the competition for funding among different sectors of the economy and society, there is a growing need for developing modern management systems. Engineering and economic tools are needed to assist decision makers for cost-effective and longer lasting pavement construction and for a more effective use of available funds (Uddin, 2006). This more rational use of monetary and human resources includes the timely intervention on damaged roads and the prolongation of the service life of pavements. This approach also considers the life cost cycle analysis.

Continuing with the previous example of investments for preserving roads in USA, estimates stated that $101 billion in annual capital investment, from 2008 to 2028 would be needed to maintain all highways of USA at present condition. Hence, every year road network gets more deteriorated (ASCE, 2013). According to the most recent data, the USA has been underfunding its road network for years and a backlog of $836 billion is estimated. The bulk of the backlog ($420 billion) corresponds to the need of repairing of existing roads (ASCE, 2017).

As a result, the efficient planning of road maintenance and rehabilitation, the main goal of any road agency, is developed by means of Pavement Management Systems (Pérez-Acebo *et al*., 2017a; 2018b). A Pavement Management System (PMS) may be defined as a systematic tool to manage, plan, allocate budget, and program all the pavement maintenance works, helps road agencies in the decision process.

A Pavement Management System must contain the following components (Fig. 1 1):

- A database, which should include this information as inputs:

- A data inventory, including geometric dimensions of the roads, such as number of carriage, number of lanes, lane width, shoulder width, etc.
- Pavement data, with information about the employed materials, their characteristics, the thickness, age of construction and history of rehabilitation and maintenance.
- Traffic data, in order to know the loads and actions that affect the pavement.
- Pavement condition data, the present condition of the pavement is totally necessary. There are some characteristics of the pavements, mainly roughness, skid resistance and texture, distresses and structural capacity, which are collected by means of a variety of devices that produce some indicators or indices.
- Environmental data, like climate information.

- A reference system, to establish the relation between previous data in a unique way.
- An analysis module. With the previous information is able to predict the future condition of the pavements, to assess of the funding needed to achieve a performance level, to recommend the optimal fund allocation and the prediction of the future pavement condition after the different treatments and scenarios. The analysis module is the backbone of the PMS, but the previous information must be reliable.
- Finally, a reporting module, which produces reports to recommend the needed work, indicating the characteristics of that intervention.

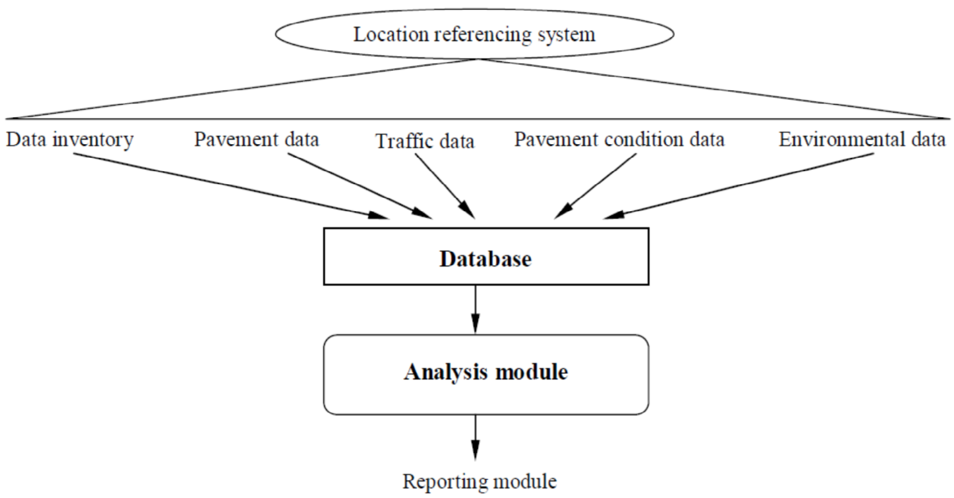


Fig. 1 1. Schema of a Pavement Management System (Adapted from Gutiérrez-Bolívar Álvarez and Achútegi Viada, 2003; AASHTO, 2012).

As explained, a key ability of any PMS is the ability to forecast the future condition of a pavement, according to some variables, which is conducted by the performance model. These models can predict the rate of change of some characteristics of the pavements. Therefore, for any property of the road, which is usually evaluated and rated by an index, deterioration models reproduce the evolution of that property according to some inputs (Fig. 1.2). If a minimum value for that index is established, it is possible to know the remaining life or the time until failure of the road (caused by the total dysfunction of that characteristic). Fig. 1.2 shows a schematic illustration of how a performance model is applied to an existing pavement section to estimate the rate of future deterioration, and the years until the failure, based on the minimum threshold established.

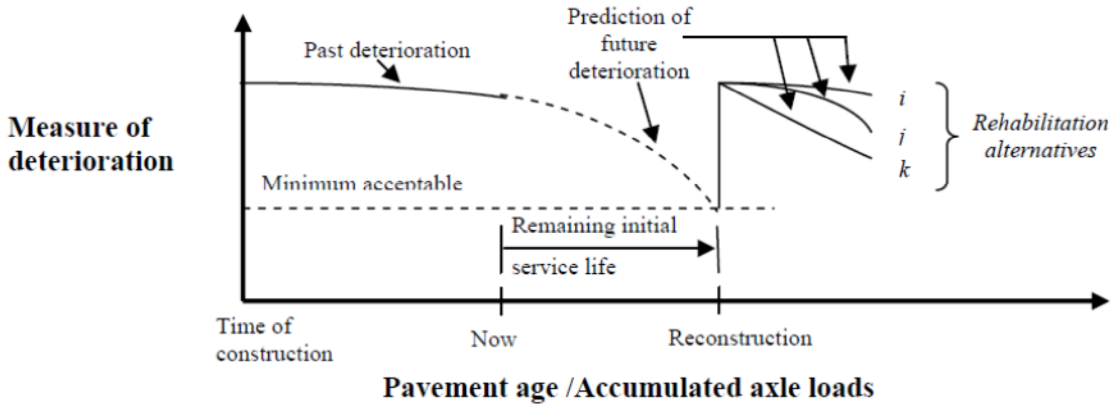


Fig. 1.2. Illustration of a deterioration model. Adapted from Haas et al. (1994).

Apart from explaining how a pavement deteriorates, performance models are useful to know the rehabilitation alternatives would perform (Fig. 1.2). Therefore, the best alternative can be chosen, i.e., the alternative which provides a lower deterioration rate. Some condition indices evolve towards lower values and others towards higher values (Fig. 1.3).

Additionally, different thresholds can be fixed, in order to remark the different states of a characteristic, and, hence, conduct different rehabilitation and maintenance works. Thanks to the development of performance models, road agencies are able to allocate their funds by applying the optimized solutions. Consequently, it is possible to answer to the usual question: “Which is the best solution? It is better to allocate lower quantities of money to maintain roads in shorter intervals of time or to allocate a higher amount after a longer time?” (Fig. 1.4).

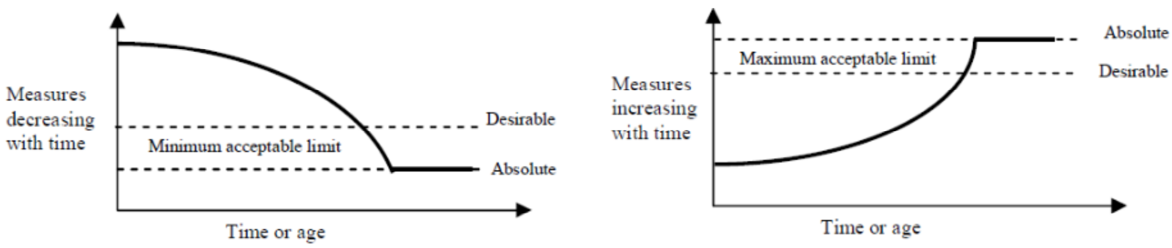


Fig. 1.3. Limits or thresholds for various measures of deterioration (usually indices). Adapted from Haas et al. (1994).

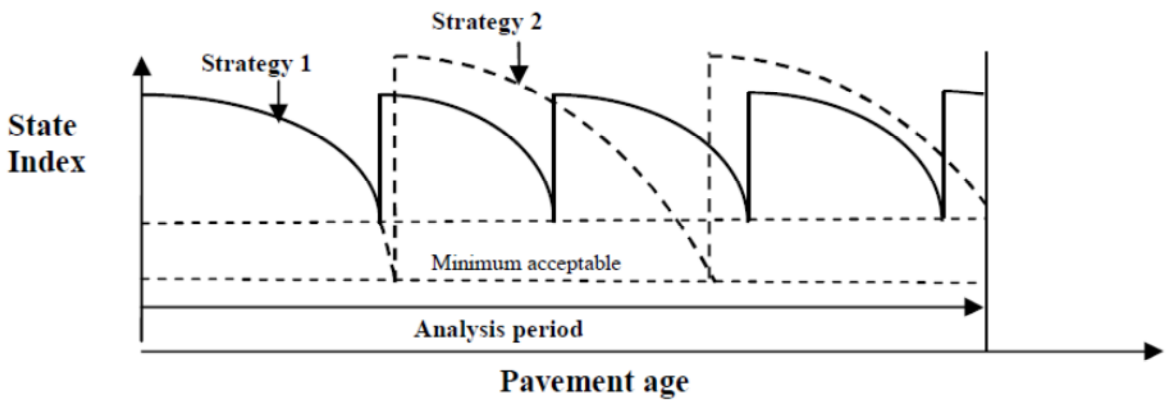


Fig. 1.4. Different road maintenance strategies. Source: Author

These are usual questions. It is better to conduct a preventive and routine maintenance or to wait until the accepted minimum level and then rehabilitate the pavement? Apart from the costs of the maintenance and rehabilitation costs, costs for users (vehicle operating costs), cost of user delays, accident costs die to traffic hazards and environmental damage must be taken into account.

Road agencies and researchers developed several performance models for each of the indices collected on a road. They can be applied for some highways if some circumstances are met. A road agency must select the most appropriate deterioration model according to the available data and selected models are usually adapted to the variables of the area, to reflect the factors that really influence in the deterioration rate of the pavements.

1.2. Pavement Management System of the Regional Government of Biscay

Biscay is one of the three provinces of the Basque Country, an autonomous region in the north of Spain. Due to the special status of the region, each province has the competence about roads, allowing the Regional Government of Biscay (RGB) to plan, project, construct, maintain, finance, use and manage all the roads in its territory (Pérez-Acebo, 2018; Pérez-Acebo *et al.*, 2017a). Therefore, the RGB manages a road network of more than 1200 km.

Since 2010 the RGB has developed its own Pavement Management System, called “State Agenda”, which include a wide database about the road network of Biscay. It was aimed to take advantage about the benefits that the PMS provide. Previously, decision about rehabilitation and maintenance works were made according to experts’ knowledge. A more rational method was pursued in order to better allocate available funds on road maintenance and rehabilitation according to optimized solutions.

The Regional Government of Biscay collected data about pavement condition, including;

- Roughness, by means of the International Roughness Index (IRI);
- Skid resistance or pavement friction, by means of the SCRIM Coefficient, and texture
- Structural capacity, by means of the deflections provided by the Deflectograph and Dynatest.
- Distress, by means of the distress extension
- Rutting

1.3. Objectives

Considered the information previously commented, the objectives of this PhD thesis are:

- Firstly, a complete review of data needed in a Pavement Management System will be conducted, and existing pavement performance models will be discussed and commented
- The information of the Pavement Management System of the Regional Government of Biscay will be analyzed. Information about pavement materials, maintenance history and available condition data will be considered in order to select the most adequate performance model for predicting the

future condition of the different pavement characteristics.

- Once the performance model type is selected, an IRI deterioration model for the pavement section of Biscay will be developed. Taken into account the factors than other proposed models for IRI evolution considered and the available information of the road network of Biscay, factors than can affect IRI deterioration are evaluated and introduced or discarded in the model. Different models will be created for different pavements, such as flexible or semi-rigid.
- Similarly, a performance model for the SCRIM coefficient will be developed. Once again, after considering proposed models from other authors, a model that really forecast the future condition of the road in Biscay will be created, using the variables that really affect the skid resistance.

1.4. Content and structure of the thesis

According to the characteristics exposed in previous sections and taking into account the objectives presented for this thesis, the content of the document was organized in the following parts and chapters.

Part I. State-of-the-art

In **Chapter 1**, the necessity of Pavement Management Systems for road agencies is commented, highlighting the benefits obtained from them. The main objectives of the thesis are also presented.

Chapter 2 describes more deeply the Pavement Management Systems, listing the required inputs and the usual data that must be included in the database of any PMS. Moreover, the possible referencing systems are commented and evaluated

Chapter 3 explains more deeply the pavement condition data that are usually collected in the roads. The main properties of the roads, which are important for users and for road agencies, are listed. For each characteristic, the range of available equipment is described, and the most employed indices for those properties are explained.

In **Chapter 4** a general revision of the deterioration models is provided, with a special emphasis in the most employed ones, the deterministic and the probabilistic models, generally referred as the basic groups. Circumstances that recommend each type of model are presented, providing clues for selecting the most adequate models in each case.

Chapter 5 comments the performance deterioration models that were developed by researchers and highway administrations for predicting the evolution of the roughness, mainly expressed as IRI, and the evolution of the skid resistance, with a special attention to the SCRIM coefficient.

Part II. Available data and methodology

In **Chapter 6** the Pavement Management System of the Regional Government of Biscay is deeply described. The main characteristics of the PMS are commented and the information that can be obtained is exposed.

Chapter 7 shows the steps made to make a decision about the selection of a performance model for the IRI and the SCRIM Coefficient in the roads of Biscay, according to the available information and condition data in the PMS. Then, the main characteristics and properties of the model selected for being employed for the data in Biscay are described.

Part III. Results: Proposed models

In **Chapter 8** the performance models for IRI in Biscay are developed. It is explained the entire process for creating the models for the different pavement structures, flexible and semi-rigid. The selection of the factors that really influence the evolution is commented.

Chapter 9 presents the performance models for the skid resistance evolution in the road network of Biscay. From all the possible factors than can have impact on the skid resistance, the ones that really influence the deterioration are selected.

Part IV. Conclusions and future research lines

Finally, the **Chapter 10** summarizes the conclusions from the thesis and the future lines of research are proposed.

Chapter 2. Pavement management systems

2.1. Introduction

In this chapter, pavement management systems (PMS) are defined and described, underling their importance and necessity for road preservation. A brief historical overview of the evolution of the concept of PMS was included, since their birth in the 60s and 70s in North America until now, with the arrival of the computers.
Moreover, the basic level of the pavement management systems are exposed and discussed. Generally, two main levels are defined, the network-level and the project-level. However, a third one, the strategic level, is also commented because, although decisions at this level are not made by road engineers, these decisions are based on the results obtained in the other two levels. Differences among the three levels are also underlined.
Additionally, the main components of any pavement management system are listed and commented. The structure of a PMS consists of some inputs, which include inventory, traffic, environmental and pavement condition data; a database to gather all the information; analysis parameters, such as performance and costs models; an analysis module, a reporting module and a feedback loop.
Finally, location referencing systems are also discussed, underlining the use of the Geographic Information Systems nowadays.

2.2. Definition

One of the most employed definitions for pavement management systems is:

"*A set of tools or methods that assist decision makers in finding the optimum strategies for providing, evaluating, and maintaining pavements in a serviceable condition over a period of time*"

(FHWA, 1989; AASHTO, 1990, AASHTO, 1993).

In other words, it can be said that "*a PMS is systematic approach that provides quantifiable engineering information to help administrators and engineers manage road pavements*" (Uddin, 2006). Therefore, the decision making process is based on information from working PMS, and includes engineering experience, budget constraints, scheduling needs, public inputs, management priorities and political consideration.
A well organized and developed Pavement Management System enables road agencies to carry out the following functions (AASHTO, 2012):

- Asses both current and future pavement conditions.
- Estimate funding needs to achieve targeted condition levels.
- Identify pavement preservation and rehabilitation recommendations that optimize the use of available funding.
- Illustrate the consequence of different investments levels and treatment strategies on both short- and long-term pavement conditions.
- Justify and secure increased funding for maintenance and rehabilitation.
- Evaluate the long-term impacts of changes in material properties, construction practices, or design procedures, or some combination of all of them, on pavement performance.

It was reported that road administrations that used pavement management tools improved their efficiency in resources employment and better transparency (AASHTO, 2001; Cowe Fall *et al*., 1994).

The idea after pavement management can be included in a wider concept that is called Transportation Asset Management (TAM). Transportation asset management is defined as

"*a strategic and systematic process of operating, maintaining, upgrading and expanding physical assets effectively through their lifecycle. It focuses on business and engineering practices for resource allocation and utilization, with the objective of better decision-making based upon quality information and well defined objectives*" (AASHTO, 2006).

The idea of managing transport assets in a more effective way is not new for transport administrations since they have been managing roads, bridges and other transport assets for decades. At present, transport administrations are conscious about the typical life cycle of a pavement and admit the need for periodic maintenance, rehabilitation and reconstruction works. Nevertheless, in the past, funding for preserving the condition of existing assets usually competed with funds for more politically and publicly-driven project to expand the network to address capacity and mobility issues. The main change in recent years is the inadequacy of funding to develop all the needs identified by transport administrations. Consequently, stewards of transport agencies prioritized the preservation of existing infrastructures and a better identification of real priorities.

Consequently, transport administration shifted this way of thinking, considering the interrelationships between funding decisions, implying the development and use of asset management principles for managing transport assets (AASHTO, 2012).

2.3. Historical overview of PMS

2.3.1. Before the concept of pavement management system

It is stated that the Romans were the first in establishing good pavement construction and maintenance practices in the PreChristian era. They built a broad road network throughout Europe and Middle East for military and for commercial use (Uddin, 2006; Pérez-Acebo, 2018). For hundreds of years the road network and pavement technology did not improve until the pioneering pavements constructed in the late 1700s and early 1800s in the kingdoms of France and Great Britain. Later, due to the invention of the motor vehicle, the modern highway pavement techniques were developed (Uddin, 2006; Pérez-Acebo, 2018). Initially, they were based on methodical specifications, and the maintenance functions were carried out principally on an ad hoc basis according to local experience. In modern times, in the second half of the XX century, the economic development is focused on general public and commercial users who take advantage of a well maintained road network. Therefore, and with the aim of providing a valuable help to decision makers, the concept of PMS was formulated in the 60s.

2.3.2. Birth and early years of the pavement management system

The mid 60s is referred as the moment when the pavement management concept is born as an answer to the numerous unanticipated pavement failures on the US Interstate and Canadian Highway Systems (Uddin, 2006; Haas *et al*., 2015). These roads were designed, projected and constructed using the best know pavement design technology at that time, following the result of the expensive AASHO Road Test in Illinois (AASHO, 1962). It was observed that it was impossible to make accurate singe-point predictions of pavement performance because of the national statistical variability of the major inputs. At that time, design methods included as inputs estimated traffic, material properties and environmental conditions for a 20-30 year life of pavement. However, these techniques did not consider the effects of pavement maintenance on pavement, nor the life-cycle cost after the design period. They did not include the overlays that were common practice.

A number of important civil engineers were involved in this analysis, and they concluded that it was necessary to integrate planning, design, construction, maintenance, and rehabilitation into a coordinated systematic method in order to provide the required pavement performance over 30, 40 or 50 year life. Fig. 2.1 was produced to describe what a Pavement Management System should be and it illustrates the many of the factors that rules the pavement performance at present.

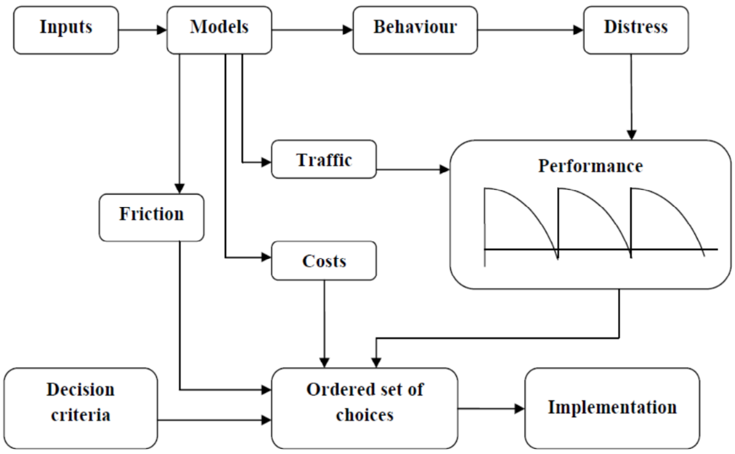


Fig. 2.1. Major components of a Project Level Pavement Design System as initially formatted in the 1960s (Nowadays, these ideas remain true) (Hudson and McCullough, 1973).

Similarly, in Canada the Roads and Transportation Association of Canada conducted a comprehensive Pavement Management Guide (Haas, 1977). This research from Canada and the precedent from USA criticized that existing methods wanted to predict performance directly from materials and weather inputs from empirical evidence such as the AASHO Road Test. These initiatives from the 60s and the 70s showed the necessity to measure pavement behaviour in intermediate steps. Previously, all the known theoretical pavement equations only predicted behaviour in the form of stress, strain or deformation, but not the performance directly. Behaviour is carried to its limit becomes distress in different forms as cracking, permanent deformation, and disintegration. Distress shown as a function of accumulated traffic loads yields the required performance curve which can be employed to evaluate the effective life of the pavement structure (Uddin, 2006).

If these factors are conveniently studied, the required maintenance overlay and rehabilitation needs for the pavement can be determined, including expected moment of those interventions and the consequences or effects of those works on pavement performance life. Then, with the complete project level pavement management process, it is possible to compare the predicted pavement performance life as a function of total life-cycle costs.

In the late 1970s, the concept of PMS evolved to a process that was able to address the maintenance needs for the entire highway network under management of the road agency (Hudson *et al.*, 1979). It was observed that the same concepts of behaviour and performance could be applied to evaluate a range of sections, as the one that forms a road network, by means of evaluation of all the factors and by developing performance prediction models for each individual section. Consequently, when compared to the rest of pavement section of the network, it was feasible to establish the needs for each pavement and to determine when to intervene in each section and to identify the priority between all the competing sections, in order to optimize the budget expenditures and maximize the total performance of the network. The other level of the PMS arose: the network level. Further explanations about project and network level are provided in subsection 2.4. Reference documents from this period are listed in Table 2.1.

Table 2.1. List of key milestones that contributed to the development of successful PMSs.

Document	**Author**	**Contribution**
The AASHO Road Test Report 5	AASHO (1962)	The AASHO Road Test gave the first comprehensive concept of relating pavement serviceability to performance and method of their measurement
NCHRP project 1-10:	Hudson *et al*. (1968)	Application of systems approach for pavement design and PMS concept
A Systems Approach Applied to Pavement Design and Research	Hudson *et al*. (1970)	Application of PMS concept for the development of pavement design systems
NCHRP Report 215	Hudson *et al*. (1979)	Application of PMS concept for highway network
PMS Workshop and conferences in North America	Kher (1985); MOT (1987).	Contribution to the dissemination of the knowledge
FHWA – PMS Policy	FHWA (1989)	The US Department of Transportation Federal Highway Administration (FHWA) required all state highway agencies in the US to implement basic PMS by 1993
Guidelines of Pavement Management systems	AASHTO (1990)	The AASHTO issued the PMS guidance to support state highway agencies in the USA in their effort to implement PMS

A literature review of early documents about PMS shows the initial work in pavement performance modelling, optimization of project selection with limited budget and overall network condition constraints (Finn, 1994). One of the first PMS for highway network maintenance and rehabilitation planning was carried out in Washington State (LeClerc and Nelson, 1982).

Moreover, the World Bank developed the Highway Design and Maintenance model (HDM III) from the results of the Brazilian study on road performance and vehicle operating cost (VOC), including inadequately maintained paved and unsurfaced roads (Watanatada *et al.*, 1987b). The inclusion of the VOC was a major advance in life-cycle cost analysis since it was possible to calculate user cost savings by bringing pavements to good condition in timely manner. The requirement from the Federal Highway Administration (FHWA) to all the state highway agencies to implement the major components of the PMS in the 1990s, and the publication of the first PMS guide (AASHTO, 1990) were the definitive step forward in the successful implementation of PMS in the USA.

2.3.3. Arrival of computers

The use of pavement management tools changed completely in the 1980s, and especially in the 1990s, with the arrival of personal computers. Most of the early PMS operated on mainframe computers, limiting their application in small agencies. As technology continued to evolve, engineers dramatically changed the pavement data collection, analyses that were carried out, in the methods used for sharing and reporting pavement inventory and condition information.

Nowadays, the modern PMS concept is a widely accepted approach around the world for cost-effective planning of road investment and maintenance management of road networks.

All these activities at both the project and the network level require data that defines the material properties, loads, environment, behaviour, distress and actual performance. All these information must be stored in a central data base which must be accessible during the entire pavement management process (Fig. 2.2). Further description of data needs is discussed in subsection 2.5. The continuous feed-back from each section must be accumulated in the database and used to update the necessary performance data.

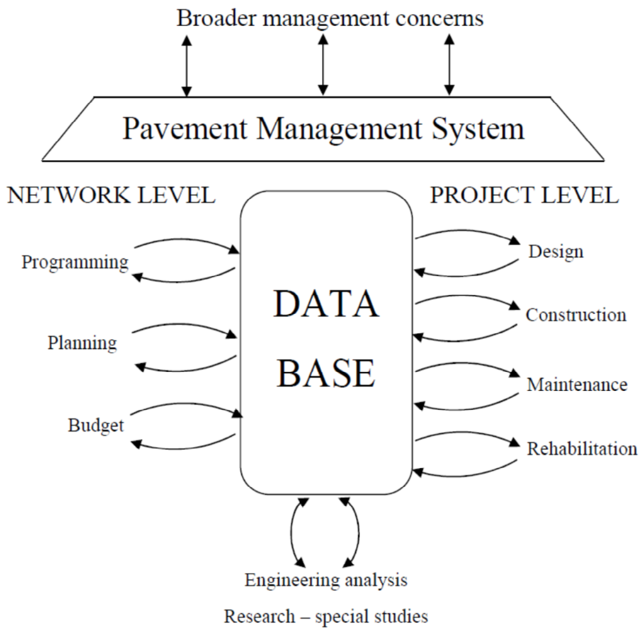


Fig. 2.2. Components of a PMS (Hudson and McCullough, 1973).

2.4. Project Level and Network Level

In literature about pavement management systems, 2 basic levels are referred: project level and network level (Uddin, 2006; Haas *et al.*, 2015). These levels are stated as primordial levels in any PMS, and differences between them must be clearly identified and adopted. Nonetheless, in the Guide to Pavement Management published by AASHTO (2012), a superior level is included, the strategic level, where political decision and investment decision are carried out. Although it could be discussed that it does not belong to the pavement management process due to the lack of engineering treatment and interpretation of data, it was concluded that it must be included as it begins the process of pavement (and asset) management when establishing strategic decisions; and also ends the process when those decisions, supported by PMSs, become real. Therefore, road agency decisions are supported by pavement management at three levels: strategic, network (tactical) and project (operational).

2.4.1. Strategic level

Strategic decisions are made at the highest level within the road administration, by persons in charge of making policy and investment decision, like elected officials, transportation boards and commissions, city councils, and agency upper management. At this level, strategic long-term decisions are taken by mentioned individuals, reflecting agency and stakeholders' priorities. They are also responsible of establishing targeted performance levels. Generally, these strategic decisions are less structured than decisions made at other levels. Results obtained from the network must be employed to assist decisions at strategic level. If this information is not available, political priorities may prevail.

2.4.2. Network level (tactical level)

At the network level (tactical level), after assessing the global condition of the road network, a prioritized work programme and schedule is produced by examining several different scenarios of time and budget constraint. In other words, the overall needs of the highway network are taken into consideration and multi-year programmes are developed with the aim of achieving the agency's goals (established in the strategic level).

When considering pavements, a wide range of strategies can be proposed. One example is the worst-first strategy, which implies that roads in the works conditions are the administration's target. An alternative option would be a mix of preventive maintenance, rehabilitation and reconstruction activities in the whole network. Selected analysis and strategies are presented to the decision makers at the strategic level to help them establishing realistic performance goals and evaluating investment options.

The network-level analyses require information and data from the whole road network. Hence, administrations must find an equilibrium point between the level of detail that can be provided and the resources used to collect the information. Consequently, information at this level is said to be moderate in terms of sophistication. It is not functional to determine the future condition of an individual road of the network, which would not be exact, but the general average rate of pavement deterioration is regarded as reasonable and representative.

As previously explained, although initially PMS were focused on individual road performance models and projects, from the 70s, pavement management activities are primarily oriented to network-level decisions.

2.4.3. Project level (Operational level)

At the project level (also operational level) the implementation of a programmed project is performed in the most cost-effective way so that the invested funds are employed to construct, rehabilitate, or maintain longer lasting pavements. Generally, decisions are made about specific projects and are related to a particular portion of the network. When a road (or part of it) is identify as a candidate for being repaired at network level, a more detailed project-level analysis and evaluation is carried out to define the best solution for it based on in situ conditions. As the project-level analysis is focused on such small portion of the network, road administration can afford to collect a more detailed information about the pavement to determine the characteristics, such as cores and material testing, which could be impractical at the network level.

Decisions made at this level are not as important as those made at the strategic level. Nevertheless, if a series of poor project-level decisions are taken over time; cumulative negative effects could not allow the agency meet required needs. Table 2.2 shows the types of decisions made at each level, their level of details and breadth of decisions.

Table 2.2. Differences in Strategic-, Network-, and Project-Level decisions (AASHTO, 2012).

Decision level	Types of decision/activities	Range of assets considered	Level of detail	Breadth of decision	Examples of job titles at this level
Strategic	• Performance targets • Funding allocations • Pavement preservation strategy	All assets under control of the administration	Low	Broad	• Legislator • Commissioner • Chief Engineer • Council Member
Network (tactical)	• Project and treatment recommendations for a multi-year plan • Funding needed to achieve performance targets • Consequences of different investment strategies	Entire road network in the area	Moderate	Moderate	• Asset Manager • Pavement Management Engineer • District Engineer
Project (operational)	• Maintenance activities for current funding year • Pavement rehabilitation thickness design • Material type selection • Life cycle costing	Specific road of the network	High	Focused	• Design Engineer • Construction Engineer • Material Engineer • Operations Engineer

2.5. Components of a Pavement Management System

Despite the fact that pavement management characteristics may vary importantly from one agency to another, according to their needs of information to support their decision and available resources, there are some basic components that are used in most systems, schematically shown in Fig. 2.3.

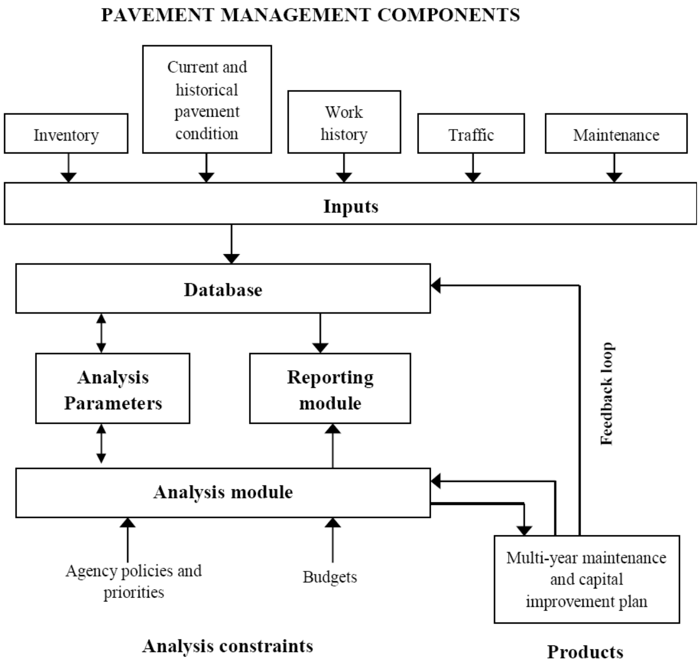


Fig. 2.3. Pavement management components (AASHTO, 2012)

2.5.1. Inputs

Any pavement management system relies in the data upon which all the decisions are based. The minimum inputs in the most elemental PMS include inventory information (road identifier, location, number of lanes, and width, pavement type) and pavement condition data (both current and historical values). Generally, an estimate of pavement age is required in PMS to predict future condition. Traffic counts may be employed to estimate pavement age, but generally, administrations to use the year in which the road was constructed or the last major rehabilitation or maintenance activity was performed.

Despite the fact that a pavement management system can work with the basic information listed above, if additional detailed information is included a more sophisticated analysis can be carried out. For example, with detailed construction history information, it is possible to link differences in pavement performance to pavement structure distribution. If this information is not available, more general rates of deterioration should be developed (Pérez-Acebo *et al.*, 2017b; 2018a).

The usual inventory, traffic data and environmental data included in PMSs are described in section 2.6. Pavement condition information, which is essential as input of PMS, is deeply described in Chapter 3.

2.5.2. Database

Although pavement management works can be carried out even without a computerized system, computers help to make use of technology for storing, sorting and retrieving the inventory and condition information. Data storage forms can range from simple spreadsheets to relational, self-contained database, to a data warehouse, in which the entire agency's data are stored. For the last years the use of Geographic Information Systems (GIS) has become more habitual. Further explanation about the integration of GIS in data storage is provided in section 2.10.

2.5.3. Analysis parameters

Certain analysis parameters are created to develop pavement projects and rehabilitation/maintenance recommendations. Usually, four types of analysis parameters are deployed in PMS: pavement deterioration models, treatment rules, impact rules and cost models.

Pavement deterioration models are the key tool to predict future pavement condition in the entire network. They are fundamental for the estimation of future funding needs and for the determination of the moment when the rehabilitation or maintenance works must be carried out. Deterioration models help demonstrating the consequences of different maintenance strategies and, hence, help road agencies choosing the best solution that fulfils their goals. If pavement evolution models are developed, they can be employed in different manner. For example, they can be utilized to compare the performance of different pavement structures in which some materials differ; to compare the cost-effectiveness of different rehabilitation or maintenance strategies; or to plan maintenance cycle over the entire life-cycle. Normally employed model types are deeper described in Chapter 4 since they constitute the main goal of this thesis.

Treatment rules are also deployed in pavement management. They define the conditions for maintenance, rehabilitation or reconstruction activities. Treatment rules range from simple rules recommending the needed repair (preventive, maintenance, minor or major rehabilitation) to very complex rules that takes into account many factors, such the type of last treatment applied, the location of the road or traffic level. The more or less sophistication of the rules depends on the needs of the road administration and the availability of information.

Apart from treatment rules, the definition of the impact of the different rules is also required in pavement management systems. These definitions are usually referred as impact rules, and they show how much a pavement can be improved after a certain treatment (generally expressed numerically) and how the pavement will evolve after that treatment. Impact rules can be regarded as deterioration models for rehabilitated or reconstructed pavements and they assist forecasting the consequences of various scenarios on pavement conditions according to different treatments. Once again, the degree of sophistication determines the developed models.

Finally, cost information must be also included in the analysis. At a minimum, the costs for each treatment are required in the analysis, in order to anticipate future budget necessities.

2.5.4. Analysis module

Once the performance models are developed, it is possible to process and analyze pavement information that allows road agencies choosing decisions among all the possibilities. This information is provided by the pavement management analysis. The level of sophistication also varies according to the agency's needs. The main goal of this module is to help agencies assessing the consequences of applying different investment levels and treatment strategies over the road network. The typical outputs from this module are:

- An evaluation of the funding needed to achieve the performance level goal.
- Recommendation for the optimal employment of available funds.
- Prediction of future pavement conditions according to different treatment and scenarios.

Moreover, the information can be employed in order to establish criteria for performance-based specifications or determining performance goals under warranty contracts.

2.5.5. Reporting module

Since the central repository about pavement information, the pavement management system is the source of several types of reports about the road network. Consequently, most PMS have a reporting function that generates standard and ad hoc reports. At present, due to the introduction of GIS and web programmes, accessibility to pavement management information has increased considerably.

2.5.6. Feedback loop

During the employment of a pavement management system, it must be verified that predicted performances are similar to those really obtained. Consequently, in any PMS pavement management records are updated with pavement performance and construction information from the field. Ideally, this feedback loop should be carried out automatically by means of systems that link relevant and similar information together. Nonetheless, in real PMS, this work is carried out by agency individuals. The usual information provided to the PMS by means of the feedback loop is:

- Finishing dates of maintenance and rehabilitation activities
- Changes in the treatment conditions or material properties
- Real performance of treatments under some conditions

- Primary mechanisms causing pavement deterioration
- Average performance of some pavement designs and works

2.6. Pavement Management Software

Highway administrations developed public agency programmes by means of public funding. These programmes usually have a standard analysis structure followed by all users and they are relatively low costs. There are some interesting pavement management programmes developed by public agencies, with successful results. For instance, MicroPAVER pavement management system developed by the US Army Corps of Engineers and the StreetSaver, developed by the Metropolitan Transportation Commission in the San Francisco Bay area (FHWA, 2008) can be cited.

On the contrary, private corporations also dealt with the development of pavement management software. These programmes are more expensive than public domain programmes but they are more flexible when customizing and adapting to each user's needs (AASHTO, 2012).

2.7. Inventory information needs

As shown in Fig. 2.1 and Fig. 2.3, pavement data are vital information in any pavement management systems. By means of these data, the pavement network can be identified, classified and quantified according to various aspects. The following list presents general PMS data needs at network- and project-levels:

a) Inventory data, including section-specific road classification, physical dimensions and pavement material data. Further description is provided in section 2.7.1.
b) Traffic history data, including traffic volume in each road section, directional split, vehicle mix, truck load data, equivalent single axle load (ESAL), etc. More details will be explained in section 2.7.2.
c) Environmental data. General climatic data in the region, environmental data history must be stored. Some authors include this information in the inventory data. It will be described specifically in section 2.7.3.
d) Pavement condition data. It is related to the evaluation of present condition of the pavement. A deep description is shown in Chapter 3.

2.7.1. Inventory data

The most basic information in the inventory data in order to describe the general pavement section includes the following characteristics (Dewan and Smith, 2003; Gutiérrez-Bolívar Álvarez and Achútegi Viada, 2003; Gonzalo-Orden, 2004; Uddin, 2006; Khattak *et al.*, 2008):

- **Road or/and segment identification data**. It should include: road name and identification (ID or code), segment identification (ID or code) if the road is divided in some discontinuous segments), segment beginning and ending points, segment length, location referencing codes, geographic coordinates, road classification, function classification of the road, administrative zone (in case the region is divided in some maintenance areas). Ownership information (jurisdiction: state or local road agency).

- **Carriageway geometric data**. It should include: Single or double carriage, number of lanes in each traffic direction, lane width, inside and outside shoulder type and width, sidewalks,
- **Construction and design data**. It should include construction date, project identification (ID, code), construction costs, design traffic mix, design ESAL or typical annual traffic volume.
- **Pavement structure and material data**. It should include: layers (Hot Mix Asphalts, Portland Cement Concrete (PCC)), pavement surface type (bituminous or concrete), layer thickness (all layers above subgrade), material data (with codes), layer material properties (strength, gradation), rigid pavements specifications (joint spacing in PCC pavements, transverse joint load transfer), shoulder material types (if different from lanes), pavement marking materials, drainage information
- **Maintenance and rehabilitation (M&R) data**. It should include: project identification (ID or code), rehabilitation or maintenance description, design data (traffic volume, design ESAL, etc.), M&R data, M&R costs, resulting pavement structure, annual maintenance costs.
- **Other structure data**. It should include: location of interchanges, bridges, culvert and drainage structure, ditch, crossing over or under roads, railroads, utility pipelines, rivers or streams, etc.
- **Appurtenance features**. It should include: Traffic control devices, signs, lighting, guardrail, barriers, video surveillance equipment, etc. (identified by a point, line, area or a reference file).

Construction and maintenance and rehabilitation history data should be recorded from the original construction project (which can be identified as Project 1 in XX road). Subsequent other M&R activities should be included and referred as Project 2 in XX road. The construction ID code may be used as a key identifier to indicate any change in the inventory data items that can have any effect on future pavement performance. The record of all historical construction data is a vital issue for evaluating deterioration and performance models, essential for the life-cycle cost analysis (Uddin, 1995). Maintenance, rehabilitation and reconstruction activities costs must be also included and must be updated annually or as soon as available.

The cost of inventory data collection and processing varies according to the existing resources, level of details, and scope of the inventory data collection. The extend of geometry, structural, and material data can range from some compulsory items to hundreds of items, as a function of the facility and intended use of the data. Consequently, in an initial pavement management system, only essential and key items must be prioritized and collected, and, in next stages, more details could be added to the database. The main efforts in data collection and development for inventory database are the initial one, when the PMS database for the network starts to be implemented. The Spanish Ministry of Public Works edited a guide to establish how to introduce inventory data in its database (Ministerio de Fomento, 2011a).

2.7.2. Traffic history data

Traffic volume and loading have a big influence in all phases of the pavement life cycle. Existing and projected traffic demands have an impact on both network- and project-level pavement management decisions. Moreover, pavement design methods and guides consider traffic loads and material properties as the key inputs variables for the pavement structure design (Li *et al.*, 2009a).

Traffic volume and traffic weight distribution data in each road section are essential inputs for any PMS, both for performance modelling and design of pavement thickness in M&R works. Traffic volume and truck weight are usually employed as weighting factors for priority ranking in M&R activity programs at the network level. Consequently, volume measurements are vital inputs of any inventory data.

Traffic data are collected by means of traffic counters or other sensors in specific locations during several days along the year, and these data are expanded to calculate the Annual Average Daily Traffic in each road segment of the network.

Load information can be estimated weighting axle loads from random vehicles of different types and by doing automatic classifications of vehicles in various categories. There are a wide range of automatic data collection devices for collection traffic volume and traffic weight data, such as, portable and fixed counters, weigh-in-motion (WIN) devices, portable scales and permanent weigh stations. Traffic volume and vehicle types are obtained from portable and fixed counters. WIN devices, portable scales and permanent weigh stations provide vehicle weight and axle load distributions. A device for weight data is recommended in any PMS.

Obtained data are analyzed and the following information are recommended to be introduced in the inventory data of the PMS (Uddin, 2006; AASTHO, 2012):

- Traffic volume data, including Annual Average Daily Traffic (AADT), percentage of truck volume, directional split and lane distribution, growth factors and cumulative past traffic.
- Vehicle classification, including traffic mix with different truck axle groups, wheel and axle configuration, tyre pressure and vehicle speed.
- Truck weight data, including gross vehicle load, wheel load and axle load
- Truck Load Equivalency data: calculation of equivalent single-axle load (ESAL) applications (using acceptable load equivalency factors) for past cumulative and future design applications.

As traffic loads applied on the pavement surface range from light passenger cars to heavy trucks, in pavement design, the damage caused by all axle loads that will be applied on the pavement along its designed life must be considered. Consequently, the AASHTO (1993) proposed the axles with different magnitudes and different numbers of repetitions are converted to an equivalent number of repetitions of a standard axle load that causes the same damage to the pavement. 80 kN applied on a single axle with a dual wheel at each was selected as equivalent standard axle load (ESAL). The ESAL is the equivalent number of repetitions of the 80 kN standard axle load that causes the same damage to the pavement caused by on-pass of the axle load in question. The AASHTO Guide for Pavement Design (AASHTO, 1993) developed Equivalent Axle Load Factors to relate the damage caused by different load magnitudes and axle configurations to the ESAL.

This conversion of traffic loads to ESALs has been used for decades in the USA and all over the world (Banerjee *et al*., 2009). Nevertheless, the MEPDG (AASHTO, 2008; AASTHO, 2010; AASHTO, 2015) proposed the use of axle load spectra instead of ESAL, representing a dramatic change in pavement design (Li *et al*., 2009a). The employment of axle load spectra provides a more accurate representation of the traffic loads for lane design, and this new approach also includes environmental condition, time of the day and season and vehicle classification. Therefore, the use of WIM becomes indispensable. In Spain, an ESAL of 13 t (128 kN) is usually employed.

2.7.3. Environmental data

Generally, pavement design procedures only took into account traffic loads and materials properties, but not the effect of environment. The detrimental effects of environment and its interaction with traffic loads on pavement performance can be seen in “aging” as condition deterioration accelerates with time. Environmental factors must be identified and taken into consideration in improved project-level design and network level maintenance programmes, in addition to traffic volume. The combined effect of traffic loads with one or more environmental mechanisms is more critical for road deterioration than the effect of the environmental factor alone. Impact of environmental inputs are widely established in the literature (Uddin and Chung, 1997; Haas, 1973)

Climate data is not an expensive data input in PMS. In most countries the environmental data are available from meteorological departments. Usually collected environmental history data for pavement performance analysis include:

- Ambient air temperature (daily maximum, minimum, and average)
- Average wind direction wind speed (daily maximum, minimum, and average)
- Daily total precipitation,
- Typical monthly solar radiation
- Seasonal subsurface water content changes

Apart from these meteorological data, some section-specific data contributing to environmental related deterioration are:

- Surface drainage,
- Frost depth, freeze-thaw information
- Pavement temperature

Most of these data items are included in the environmental database and priority should be assigned to most items for project-level evaluation only. For the network-level activities only general environmental region categories are deployed, such as arid, temperate, dry-freezing or wet-freezing climates.

2.8. Location referencing systems

2.8.1. General comments about location referencing systems

All data obtained from the road must be referenced. This system must allow knowing all the roads in the network, their location, and any point in them. It must be a “biunivocal”, meaning that any point in the office could be located in the road, and vice versa. The location referencing system (LRS) must be universal for all the people working in the road agency, and independent from personal rules (Gonzalo-Orden, 2004; Uddin, 2006).

Existing location referencing systems (LRSs) at local levels are almost exclusively linear and highway or street oriented. Nevertheless, the extended use of GPS and other spatial technologies are making develop a

LRS that can integrate data expressed in multiple dimensions. Transportation agencies already deal with data referenced in one, two or three dimensions. Nevertheless, the data are usually used in incompatible formats and data employ technologies and databases that cannot be integrated.

The main advantage of a LRS for becoming the ideal tool to integrate data from different systems for decision making is its ability to relate multiple location referencing methods for a single asset. For instance, IRI data are expressed as linear segments, for 100 m long segments, while falling weight deflectometer are based on point data, punctual data. An LRS contains multiple location reference methods, each one referring to assets or attributes, or both for a single asset. This characteristic is why an LRS is a useful tool for integrating data among different systems.

The LRS is the essential base of a road data management system, and is indispensable for data integration. Due to the changes of roadway geometrical features (road real alignments, adding or deleting lanes, etc.), any LRS is temporal.

2.8.2. Location referencing methods

As commented, a location reference system or method is employed in situ to ensure that the proper address is employed to describe a location and that the proper *location* can be found using its *address*. In general, all location referencing methods have the following components:

- Identification of known point
- Direction (e.g., increasing or decreasing)
- Distance measurement (i.e., a displacement or offset).

There are two common location referencing methods:

- Linear: gives an address that consists of a distance and direction from a known point, which can be a kilometre point, kilometre post, reference point or reference post.
- Spatial: gives an address consisting of a set of coordinates.

It must be noted that one location can have many address, as shown in Fig. 2.4, where the same location can be described by five different addresses.

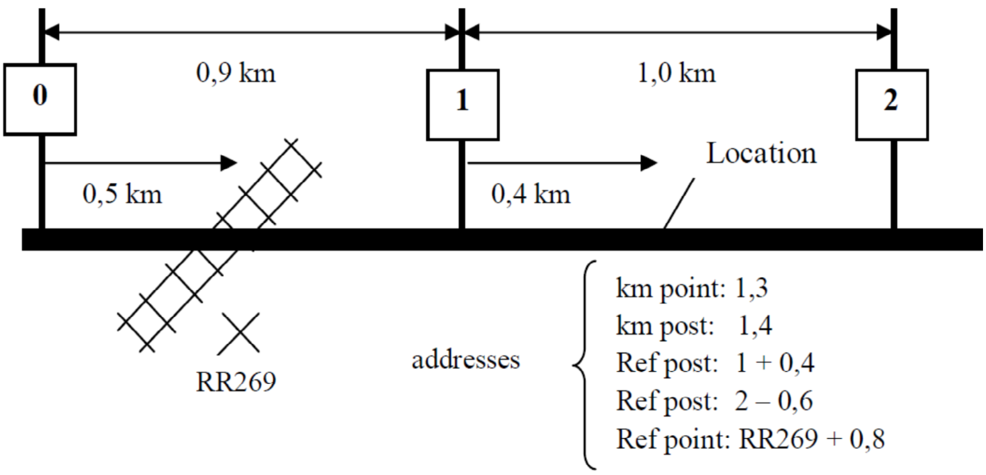


Fig. 2.4. Example of various addresses applied for the same location (adapted from Bennett et al., 2007).

Links and nodes are the main tools of any referencing system. The nodes refer to specific locations on the roads, and the links are unique segments connecting the nodes. Almost all the referencing methods can be deployed with link-node system.

2.8.2.1. Linear Referencing

Linear referencing is said to be the most commonly used referencing methods for road data. Furthermore, it is reported that almost in every case, where spatial data are employed, it is recorded in conjunction with linear referencing data (McGhee, 2004). It does not need any sophisticated technology, as the spatial referencing methods do and can be easy understood and applied. Moreover, McGhee (2004) highlights that discussions with people in charge of road maintenance strongly suggest that linear referencing will be in use also in the future. There are four basic linear referencing methods, described in next sections.

2.8.2.1.1. The kilometre post or milepost method

The kilometre post or milepost method is probably the most commonly used methods (Gutiérrez-Bolívar Álvarez and Achútegi Viada, 2003; Gonzalo-Orden, 2004). The main difference between kilometre points and kilometre posts is that post implies the use of physical posts and sings placed at regular intervals along the road, usually spaced 1 km approximately (Fig. 2.5a). It must be underlined that posts are almost never exactly 1 km apart, mainly to changes in the alignment.

One of the main problems is the inclusion of new road stretches, such as a bypass of a village, which makes the kilometre posts vary. One of the solutions could be to modify the kilometre post from the point of the modification. This would be very expensive and it could carry a lot of problems for referencing. It is more adequate to maintain the existing position of the kilometre post and to vary the distance between them (Gutiérrez-Bolívar Álvarez and Achútegi-Viada, 2003). Therefore, some of the one-kilometre distance between kilometre posts would suppose more than 1.000 meters and others would suppose less than 1.000 meters. This way, the kilometre post acts as nodes in the network and every point in the road is referenced as a distance to a specific kilometre post. Table 2.3 summarizes the advantages and disadvantages of this referencing method.

Most data collection technologies utilize linear referencing when recording data, since the data are recorded from a start to an end point. The addresses are generally expressed referenced to the start point and, ideally, to intermediate points. The employment of intermediate reference points improves the general accuracy because it limits the accumulated error in the distance measurements.

Table 2.3. Advantages and disadvantages of the kilometre post referencing method.

Advantages	Disadvantages
• Location information is readily understood by all users • Information is available for public use • Numerical sequence provides easy orientation	• During the initial survey must be accurately positioned. • Replacements must be place in exactly the same location • The distance measurement displayed on a post is never the exact distance, which could lead to confusion for those who use the posts as references. • If realignments are carried out, in order not to moved all downstream signs, longer or shorter distance between consecutive kilometre posts must be introduced

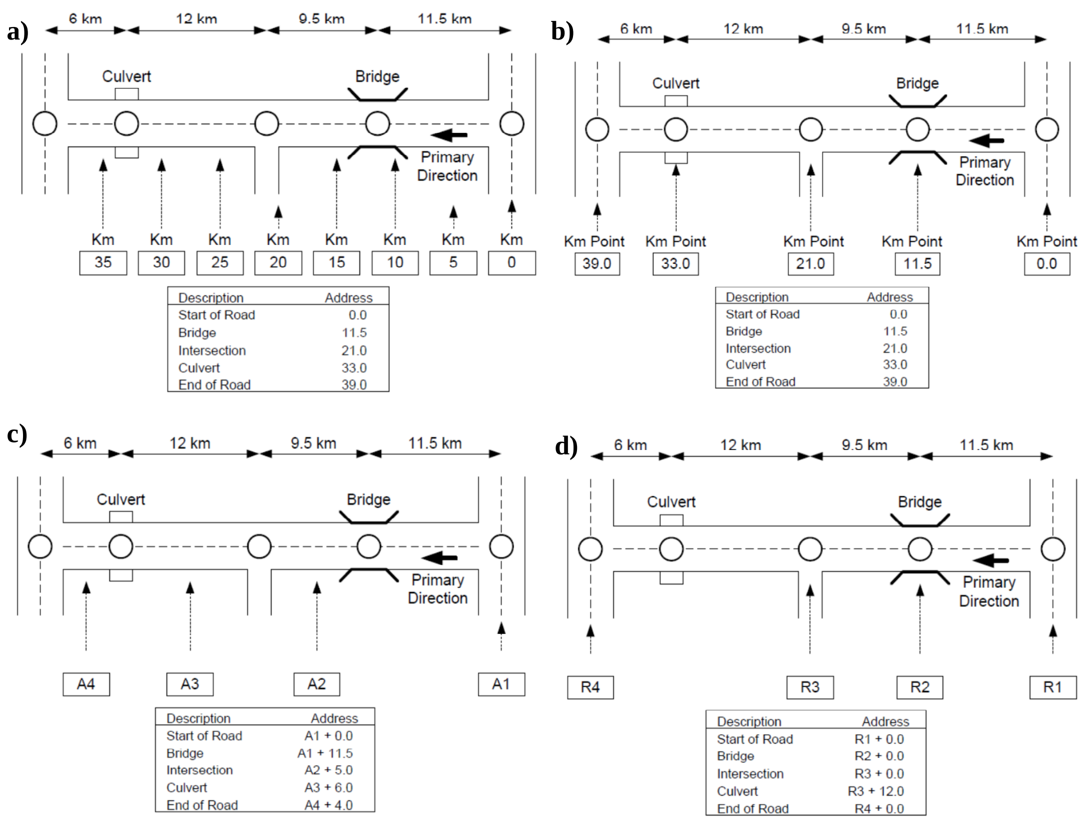


Fig. 2.5. Referencing methods, a) Kilometre post or Milepost method, b) Kilometre point method, c) Reference post method, d) Reference point method (adapted from Bennett et al., 2007).

2.8.2.1.2. The kilometre point method

The kilometre point method utilizes the measured distance from a given or know point to the referenced location. The start point is usually the beginning of the road or the point where it enters a city or a district. The address of any point along the road is the numerical value of the distance of the point from the beginning of the road (Fig. 2.5b). Advantages and disadvantages of this method are listed in Table 2.4.

Table 2.4. Advantages and disadvantages of the kilometre point method

Advantages	Disadvantages
• There is no need to maintain reference posts or signs • The distance between two points is simply the difference between them • They are easily understood and calculated	• Field workers need to measure from the kilometre point to get a reference. The start of the route and the primary direction must be known. • Addresses are unstable. If the road is realigned, all the points downstream change. While the effects can be minimized by having regular reference stations, it still creates the problem of reconciling historical data.

2.8.2.1.3. The reference post method

The reference post methods is similar to the kilometre post method, but with the exception that the signs are not at regular intervals. A sign or marker is positioned next to the road with a unique identifier. This identifier may be a distance or just a number. Although a reference post never changes, the kilometre point associated with the post may change. If the distances are properly maintained, the method is successful. Events in the road are measured as a displacement from these posted references (Fig. 2.5c). Advantages and disadvantages of the method are displayed in Table 2.5. Some authors consider kilometre posts as reference posts since they are almost never exactly 1 km apart.

Table 2.5. Advantages and disadvantages of reference post method

Advantages	Disadvantages
• Easy to use in the field • Easier to maintain than kilometre posts • It does not need to measure from the start of the route, only from the nearest point • Less expensive than kilometre posts, as appropriate locations are used and less signals are utilized. • A single set of signs can be used on overlapping routes • If changes are made to the road length, it is not necessary to change all the other signs, only the affected section.	• Location information is not clear for all users • Public generally not able to use information • Field crews needs to measure distances if they are not marked • Damaged signs must be replaced at the exact location

2.8.2.1.4. The reference point method

It is similar to the reference post method, but in this case features, such as bridges, culverts or intersections are used as identifiers instead of signs (Fig. 2.5d). Events in the field are measured as displacements from these posted references. Advantages and disadvantages are displayed in Table 2.6.

Table 2.6. Advantages and disadvantages or reference points

Advantages	Disadvantages
• It is not necessary to measure from the start point of the route, only from the nearest reference point • Minimal maintenance requirements, as not posts are used • No need to change all the signs if realignments are conducted, only the affected section.	• Cumbersome to use in the field • Reference points may spaced at very long intervals, particularly in rural areas • Location information is not clear to all users • The public is not able to use the information • Road crews need to know details on reference points and distances

2.8.2.2. Spatial referencing

Spatial referencing is accomplished using UTM coordinates, where every point of the road is referenced to some coordinates that fix it in the space. Spatial referencing is usually applied as part of the Global Information Systems (GIS), further commented in section 2.9.

2.8.3. Segmentation

A Location Referencing System (LRS) should support changes between location referencing methods and segmentation of data. This functional need is a key component to integrate data from different sources. Segmentation is the process of transforming linearly referenced data, stored in tables into features than can be presented and analyzed on a map using transforming rules. For instance, a road administration may divide (make segments) of the road network by pavement type or condition. Attribute information characteristics specific to each road segment can be maintained without splitting the road network. This process makes easier the transformation of data from variable sources to defined segments according to user's needs. An example can be seen in Fig. 2.6.

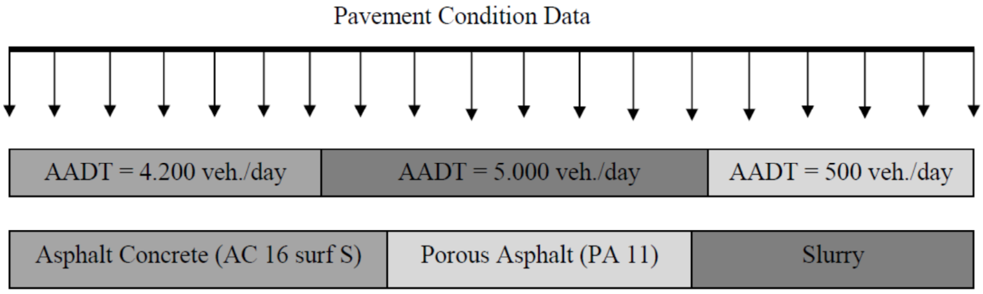


Fig. 2.6. Example of data transformation from different sources. Source: Author.

Fig. 2.6 shows the segmentation and transformation of data from different sources (IRI for pavement condition, AADT for traffic data and pavement type for pavement for pavement inventory data). Data after transformation allow the user to evaluate the pavement condition as a function of pavement type and the traffic category of those segments. Transformation rules should be identified for each data item so that the data can be transformed accurately. For example, in this case, IRI data are transformed based on calculating an average IRI since the IRI segments length are constant, whereas AADT is based on weighted average by length since the segments are variable in length.

2.9. The importance of Geographic Information Systems in data integration

As seen over this chapter, any pavement management systems need a big data inventory, consisting of enormous quantities of information. Despite of the fact that it is not necessary to store all the information in a central database, all data must be readily accessible and comparable. Data integration and data sharing are key factor of any management system.

Data integration can be defined as " *the process of combining or linking two or more data sets from different sources to facilitate data sharing, promote effective data gathering and analysis, and support overall information management activities in an organization*" (FHWA, 2001).

Decisions about pavement activities are a good example for the need of integrated data. For instance, a pavement manager can access to all the maintenance history of a section, even it is stored by another branch of the administration.

Consequently, one of the most employed database integration systems is the geographic information systems, which links any point or polygon object (spatial data) to fixed coordinates in the space, and it is possible to add some attribute data (Gonzalo-Orden, 2004).

A geographic information system (GIS) is "*a computerized database management system for storing, managing, retrieving, querying, analyzing, and presenting spatial data*" (AASHTO, 2012). A GIS is based on two types of information: spatial data and attribute data. Spatial data define line, point or polygon objects (also called elements) placed on the surface of the Earth. Information linked to these objects or elements is stored in attribute tables. The main difference between e GIS and other graphical presentation systems, such as CAD (Computer-aided design) is that the information linked to each graphic element is associated to a database efficiently. The efficient link between the map and the information allows the PMS that uses a GIS to be very fast and efficient when retrieving data related to a specific physical place and allows for complex data analysis.

A traditional GIS database, without dynamic segmentation capabilities, is limited when managing and presenting multiple integrated data sets of a single characteristic, like pavement section. The GIS database usually includes all data sets combined in a single database table. New records, and corresponding graphic elements, are then produced when attribute values change. As all data sets are reduced to the smallest segmentation, it results in a big amount of redundant data.

Another way for managing data sets is to maintain each attribute table separately. A one-to-one relationship between attribute data and graphic elements must be maintained within the GIS and it results in a network graphic redundancy. This redundancy can be reduced if all the records in all tables have the same linear extension. Consequently, multiple attribute records could share the same graphic representation. Nevertheless, this separation limits flexibility in data collection and maintenance.

Therefore, the most robust method is certainly the dynamic segmentation. As explained in previous section, dynamic segmentation is the process by which linearly-referenced data can be transformed so the information can be used for other goals. It makes easier data integration and makes possible to share agency data internally and externally.

The spatial nature of GIS can employ the spatial referencing methods to identify the location of attribute data, i.e., coordinates identify the place of a point or linear segment along a road. The coordinates can be either geographic (longitude and latitude) or projected (mile posts). It is usual to develop maps from GIS tools, where colour codes identify different categories, for communicating the information from the PMS.

2.10. Conclusions of the chapter

Pavement management systems are used by road agencies as a key tool to help engineers when making decision about the best rehabilitation and maintenance strategy for the road network managed by them. PMS emerged in the late 1960s and early 1970s in North America (Canada and the USA) and has expanded to the entire world as they provide benefits. A PMS identifies strategies for using available resources more effectively to improve pavement performance and allows justifying increased funding for pavement maintenance and rehabilitation activities.

A PMS is composed by various parts, a database with information related to pavement structures, activities history, traffic data, environmental data and pavement condition data. All this information must be properly references, usually by the kilometre post or milepost. Another part, the analysis module provides deterioration models, which all allows forecasting the future condition of the road network. The analysis module provides optimized strategies for pavement maintenance according to available funds.

Chapter 3. Data collection and pavement condition assessment

3.1. Introduction to the chapter

In the previous chapter it was said that different information must be introduced in the database of a Pavement Management System, such as inventory data, traffic history data, environmental data and pavement condition data. This chapter describes deeply the pavement condition data collection. Firstly, it presents the characteristics and properties that are important for users and engineers and gives an overview about the usual classifications employed.

Taking into account the 4 main groups in which pavement properties are classified; the characteristics of each property are defined and commented. Moreover, for each property, the usual equipment and devices are exposed and discussed. Additionally, the most employed indicators or indices are listed and discussed. The skid resistance and the pavement roughness are more deeply commented as they are the properties that are used for developing deterioration models for roads in Biscay. Finally, some comments about composite indices are introduced

3.2. General overview about pavement condition assessment

Pavement condition assessment can be defined as the process of collection and processing indicators and indices of pavement condition (AASHTO, 2012). As stated in Chapter 2, the information about present condition of pavements is an essential and vital component of any pavement management system (Fig. 2.3). By means of pavement condition data, road agencies can know the condition of the pavements under their responsibility and can make decisions based on that information. It is clear that the pavement performance is a discussion about the results of a condition assessment of the pavement. Hence, evaluating pavement condition is a key part of any road administration's pavement management system.

There is a wide range of approaches to condition assessment, distinguished by the extent of the evaluation, the level of the detail collected, and the evaluation tools employed. For instance, a project-level investigation can be regarded as the most detailed level, carried out on isolated pavement sections. At project-level, investigations are conducted to establish the reasons to specific performance problems and to explore the possible solutions, or they can be carried out to obtain inputs needed to project an appropriate rehabilitation activity. In any case, project-level assessments are generally very detailed and exhaustive, implying destructive and non-destructive testing tools.

On the contrary, for pavement management purposes, the evaluation is usually conducted at the network level, and it is called "*network-level survey*". Network level surveys are less detailed than project-level evaluations and are carried out on the majority (or all) of the pavement network of the road administration. Result from a network-level survey identify and prioritize treatments, to determine the quantity of funds that are needed, and to allocated budgeted funds. The evaluation at network level gives an overview about present pavement preservations and rehabilitation activities needed. Apart from the present evaluation of the pavement condition at network level, it is more important if the pavement condition data were collected over time, which allows generating a historical record of pavement condition. This information is a key component to model pavement deterioration and to forecast future condition. In few words, pavement condition evaluation is a fundamental input that is required in PMSs, it describes present pavement performance, if historical data are recoded, pavement performance over time can be observed and it is employed to predict future pavement condition (both with and without maintenance and rehabilitation works).

In network-level assessment, it is important to determine the data quantity and quality. Data quality, referring to what and how much is measured, is associated with time and cost. Typically, if a greater volume of data are collected (or more detailed data), the data collection will imply a higher cost. However, better decisions are expected to be taken if a greater volume of data is available for analysis. Therefore, road administrations must find an equilibrium point between collecting all the condition data that could use and collecting enough data to make appropriate decisions. This trade-off is also influenced by administration needs and available resources.

Data quality is important in data collection too. Quality must not be regarded only as a synonymous of accuracy, which is usually associated with used tool. There are other causes for lack of quality in condition data, such as, distresses that are not properly identified, severity of distresses that is not adequately measured, or distresses that are not located in their real place. Any kind of inaccurate data leads to poor decision. Moreover, if data are known to be inaccurate, administration confidence in the system deteriorates, and data are regarded as useless.

Data consistency also plays an important role in data collection. It can be stated that at present, there are no standardized data collection and processing techniques being employed consistently by road agencies all over the world, making difficult to compare results and values among them. Additionally, different condition indices may be used from one year to another by road agencies if different devices have been employed. This variability can be also a consequence of changes in equipments or the technology (or both) applied to data collection. Efforts are being made to standardize some aspects of data collection processes, for instance in the USA, by the AASHTO and ASTM and in Europe, by COST ACTION programmes.

Road administrations must consider data collection costs when planning pavement assessment, allocation some resources to this activity and not only to M&R activities. People in charge of PMSs want as much information as possible, but it must be determined the essential information that is needed for keeping the pavement management system up to date and reflecting present situation. Consequently, an important decision is to exactly determine what information is necessary to support the important decision and then, establish the most effective methods to collect them.

3.2.1. Types of pavement condition data

Good decisions in pavement management are dependent on available data. There is a wide range of information that can be get from pavements and there even more methods and techniques to collect that information. Each road agency has its own approach about data collection that addresses the unique aspects of the agency's network, including its own characteristics, like size, available funds, past practices and other factors, which usually determine the approach.

Depending on the characteristic is surveyed; a pavement evaluation can be classified as functional or structural.

- **Functional Evaluation**: A functional evaluation provides information about surface characteristics that directly affect users' safety and comfort, or serviceability. The main characteristics surveyed in a functional evaluation are skid resistance and surface texture in terms of safety, as well as roughness in terms of serviceability.
- **Structural Evaluation:** A structural evaluation provides information on whether the pavement structure is performing satisfactorily under traffic loading and environmental conditions. Surveyed characteristics may be related to structural performance, pavement distresses, and mechanical/structural properties. Note that several pavement distresses indirectly lead to functional problems such as asphalt pavement bleeding, which affects skid resistance.

Nevertheless, the pavement condition data that are usually collected by road agencies can be included in one of the following three categories (AASHTO, 2012):

a) **Distress**: observations of visible conditions on pavement surface that can be employed to identify the cause of the performance problem or reflect the underlying performance problems. Distress information is useful for selecting specific pavement selection and rehabilitation treatments and for planning long-term management programmes.
b) **Structural capacity**: it is a measure of the pavement remaining life, by means of load-carrying capability of the pavement (Bryce *et al*., 2013). Employed devices measure the pavement's response to applied loads, identify sub-surface conditions that can be the cause of structural problems and provide indirect measurements of intrinsic strength/stiffness properties.
c) **Surface characteristics**: they are performance measures that are related to customers' concerns. The most representative characteristics are, on one hand, the pavement's longitudinal profile or roughness and, on the other hand, the surface texture and friction properties.
d) **Sub-surface characteristics**: Visual distress and surface characteristics are related to performance characteristics observed or measured at the pavement surface. These observed surface characteristics may indicate that the existence of a sub-surface problem. Nevertheless, sub-surface characteristics are possible to be directly assessed, reporting information about future problems. The most used device for evaluating sub-surface characteristics is the ground penetrating radar (GPR).

With regard to the techniques used to collect pavement data some classifications can be made. A first division among data collection methods led to classify them in manual, automated and semi-automated (a combination of automated and manual). Traditionally, inspectors walking along the roadway section carried out detailed manual inspection survey. Nevertheless, the present size of pavement networks, the need for objective measurements, the safety of survey officials and the complexity of processing and analyzing large amount of data led to employ automated devices in data collection.

Testing methods can be also grouped as non-destructive testing and destructive testing. The former technique allows obtaining material properties, like layer types and thicknesses, layer strength and stiffness, material quality and the location of voids and saturated materials. Nonetheless, despite the precision obtained in this kind of testing, it is a slow technique, which provides fewer data points and it usually interrupts the traffic. Consequently, at network-level, the majority of the methods are non-destructive, while destructive methods are associated with project-level investigation.

Finally, a last classification can be made taking into account if the data collection is carried out by means of agency resources or by contractors. Generally, a combination of both is done.

The last discussion is about which properties and characteristics are collected by road agencies in the world. A report from the Federal Highway Administration in the USA identified the condition data normally collected at network level (FHWA, 2004). Fig. 3.1 shows the quantity of states in the USA that collect each type of data in hot mix asphalt pavements. A similar report was conducted in 2009 (Flintsch and McGhee, 2009) where percentage of road agencies collecting each kind of pavement data at network- and project-level are indicated (Fig. 3.2).

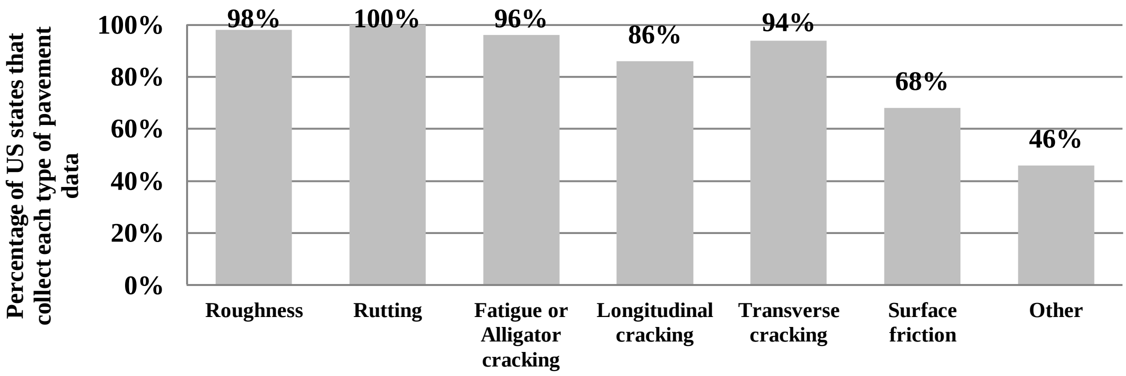


Fig. 3.1. Condition data normally collected at the network-level (FHWA, 2004)

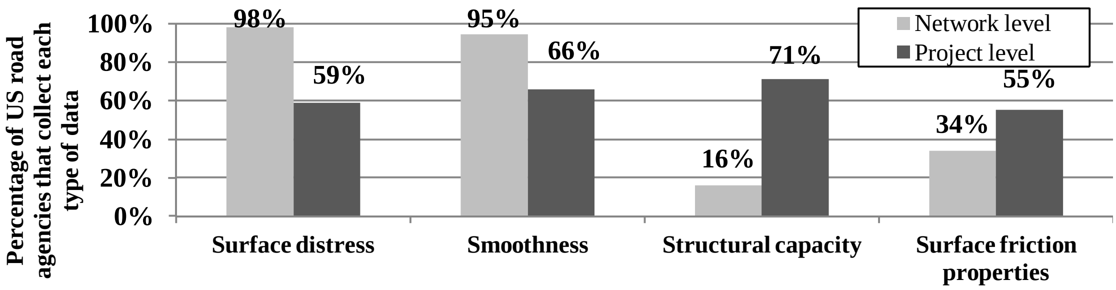


Fig. 3.2. Types of pavement condition data collected for pavement management systems (Flintsch and McGhee, 2009).

In Europe, the COST Actions have been conducted over the last decades. COST (European Cooperation in Science and Technology) is a European Union funded programme that enables researchers to set up their interdisciplinary research networks in Europe and beyond (COST Association, 2017). Some of them are focused on pavement management systems. They are briefly introduced as they are commented in other sections along this thesis. The “COST 324 Long-Term Performance of Road Pavements” (COST-Transport, 1997a; Ministerio de Fomento, 1998a) dealt with the evaluation of road condition and the development of performance prediction models. The “COST 325 New Pavement Monitoring Equipment and Methods”

(COST-Transport 1997b; Ministerio de Fomento, 1998b) and “COST 336 Use of Falling Weight Deflectometers in Pavement Evaluation” (COST-Transport, 1998) were focused on condition monitoring of road pavements. In COST 325 it is mentioned that the 95 % of road agencies in the European Union collected surface distress data and 82 % of them collected structural capacity data (COST-Transport 1997b).

The COST 354 Performance Indicators for Road Pavements has the main objective of defining uniform European performance indicators and indices for road pavement considering the needs of road users and road operators (Litzka *et al.*, 2008). It defines a Performance Indicator for a road pavement as a value derived to represent a technical road pavement characteristic, that indicates the condition of it (e.g. transverse evenness, skid resistance, etc). According to COST 354 the key properties of road pavements are:

- Longitudinal evenness
- Transverse evenness
- Macro-texture
- Friction
- Bearing Capacity
- Noise
- Air Pollution
- Cracking
- Surface defects

The World Bank proposes the following characteristics as key inputs in an evaluation (Bernett *et al.*, 2007):

- Roughness
- Texture
- Skid resistance
- Mechanical / structural properties
- Surface distress

Finally, the World Road Association (PIARC) has recently published the document “State of the art in monitoring road condition and road/vehicle interaction” (PIARC, 2016), where the present state of the art regarding the collection of road condition and road/vehicle interaction data is summarized. It provides a general view of practice at present and emerging technologies. The commonly collected road condition and road/vehicle interaction data are gathered in the following groups:

- Surface evenness
    - Transverse evenness
    - Longitudinal evenness
- Vehicle/road interaction characteristics
    - Surface texture
    - Friction
    - Traffic noise
    - Rolling resistance

- Surface defects
- Structural condition

Consequently, the following groups of pavement condition data are adopted in this PhD thesis:

a) Skid resistance (friction) and surface texture (Section 3.3.)
b) Longitudinal profile and roughness (Section 3.4.)
c) Pavement distress (defects) (Section 3.5.)
d) Structural condition (Section 3.6.)

The first two are described more deeply, as the parameters and indices obtained from the measurement of those properties are employed for performance modelling in the thesis. Pavement distresses and structural condition are only briefly commented. Finally, in section 3.7, composite indices are briefly described.

3.3. Skid resistance and surface texture

3.3.1. Introduction

The measurement and management of pavement friction or skid resistance[1] is a vital factor for highway administrations around the world with regard to safety (Wang *et al.*, 2013; Fernandes and Neves 2014; Chen *et al.*, 2016b; Feighan, 2006). The interest lies in the frictional resistance generated between the vehicle tyres and the pavement surface because it allows drivers to control and manoeuvre their vehicles in a safe way, in both the longitudinal and transversal directions. If a higher friction is available at the pavement-tyre contact, drivers can control better their vehicle (Feighan 2006; Hall *et al.*, 2009; Ongel *et al*., 2009; Buddhavarapu *et al*., 2013; Huang and Huang, 2014). Furthermore, its value is essential in the geometric design of roads. It is used in order to calculate the minimum stopping sight distance, minimum horizontal radius, minimum radius of crest vertical curves, and maximum super-elevation in horizontal curves (Pérez-Acebo, 2016).

Due to its importance, highway administration must maintain an acceptable level for skid resistance in their road network in order to minimize the road accidents. Highway crashes are complex events that normally are due to one or more contributing factors (Hall *et al*., 2009; Chen *et al*., 2016), which are usually classified as related to driver, related to vehicle and related to road condition (Noyce *et al.*, 2005; 2007). From these causes, road agencies can only control and verify road conditions, which can be done by conducting and establishing effective design, construction, maintenance , and management practices and policies (Hall *et al.*, 2009)

Despite the fact that multiple factor are involved in highway crashes; researches have established a relationship between road accidents and pavement surface conditions or characteristics, such as friction and texture (Hall *et al.*, 2009). Road accidents analyses estimated that between a third and a quarter of all wet road accident are caused or are related to surface or skidding characteristics (Kennedy *et al.*, 1990; Rado 2000). Although there have been proved consistently this relationship, it is difficult to quantify and, hence, most of the studies are empirical. For example, Rizenberg *et al.* (1972) showed that wet crashes rates

[1] Along this PhD thesis, friction and skid resistance are constantly employed as synonyms.

increased when pavement friction values were low (Fig. 3.3). Other authors obtained similar results with different friction indices and supported this idea (Gothie, 1996; Fernandes and Neves, 2013; Araujo *et al.*, 2015). Furthermore, a higher pavement friction was concluded to reduce the crash rate significantly (Wallman and Astrom, 2001). This empirical evidence underlines the probability of taking place an accident with low friction levels, and if those levels are below a site-specific threshold value, the probability increases significantly (Kuttesch, 2004). A research in the United Kingdom showed that crash risk approximately halves as pavement friction doubles over normal ranges (Viner *et al.*, 2004). Finally, a recent research demonstrated that friction also influences dry road accident rate (Najafi *et al.*, 2017).

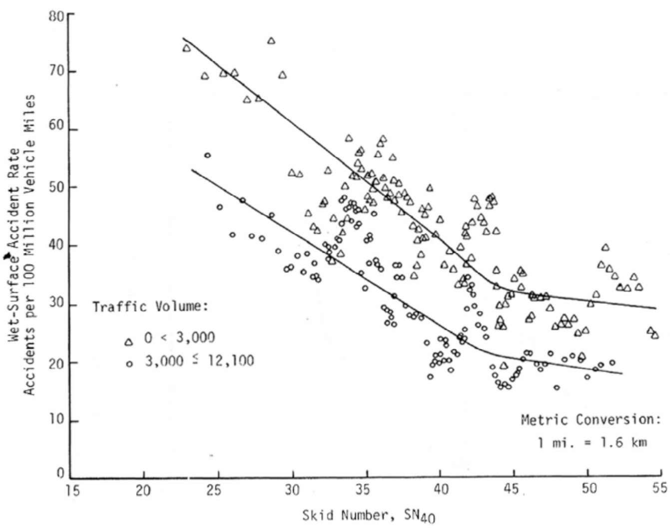


Fig. 3.3. Relationship between wet-weather accident rates and pavement friction for Kentucky roads (Rizenberg et al., 1972).

Consequently, highway agencies must control the friction level provided in their roads, by designing and maintaining pavement surfaces that provide maximum frictional resistance to braking or cornering vehicle tyres (Feighan, 2006; Hall *et al.*, 2009). Therefore, a skid resistance predictive model is necessary in pavement management systems, which allows knowing in advance the available friction on roads, as a function of different factors (Kogbara *et al.*, 2016; Pérez-Acebo *et al.*, 2017a).

3.3.2. Pavement friction definition

As defined by the AASHTO Guide for Pavement Friction, "*pavement friction is the force that resists the relative motion between a vehicle tyre and a pavement surface*" (Hall *et al.*, 2009). The resistive or friction force, *F*, is generated while the tyre rolls or slides over the pavement surface (Fig. 3.4).

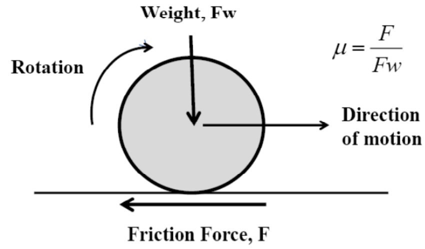


Fig. 3.4. Simplified force body diagram on a rotating wheel. Source: Author

Friction is the generally characterized by means of a dimensionless coefficient, the coefficient of friction, μ, which is the ratio of the tangential friction force between tyre tread rubber and the horizontal travelled surface, F, to the perpendicular force or vertical load, F_w, as expressed by Eq. 3.1.

$$\mu = \frac{F}{F_W} \quad [3.1]$$

3.3.2.1. Longitudinal frictional forces

This longitudinal force appears between a rolling pneumatic tyre (in the longitudinal direction) and the pavement surface when the tyre is free rolling or is constant-braked. The slip speed, S, is the relative speed between the tyre circumference and the pavement surface, as expressed in Eq. 3.2 (Meyer, 1982).

$$S = V - V_P = V - R \cdot \omega \quad [3.2]$$

Where V is the vehicle speed; V_P is the average peripheral speed of the tyre; R is the wheel radius and ω is the angular speed of the tyre. If the tyre is free rolling in a straight line, the slip speed is zero, as both speeds have the same value. Another magnitude, the slip ratio, SR, is also used, defined by Eq. 3.3 (Meyer, 1982).

$$SR = \frac{V - V_P}{V} \times 100 = \frac{S}{V} \times 100 \quad [3.3]$$

Where SR is the slip ratio expressed in percentage. Again, at the free-rolling mode of the tyre, V_P has an equal value as the vehicle, V, thus SR is zero percent. In a locked or fully braked wheel, V_P is zero, as it does not rotate, so the slip speed, S, is equal to vehicle speed, V. Hence, the locked tyre is referred as 100 % slip ratio.

In a braking manoeuvre, the obtained friction between the tyre and the pavement surface depends on the sliding percentage and the vehicle speed. This relationship can be represented graphically by means of a 3-axis graph with friction, slip speed and slip ratio in the axis (Fig. 3.5). This figure can be divided into two curves: friction vs. slip ratio and friction vs. slip speed, which are generally employed to explain the factors affecting the friction. The coefficient of friction varies with slip ratio, SR, as shown in Fig. 3.6 (Henry 2000). The coefficient of friction increases quickly with increasing slip, from zero to a peak value, usually at relatively low slip (between 15 and 20% of slip ratio), called *critical slip* (Bergman, 1976; Do and Roe, 2008). From this point, the friction decreases gradually until a value known as the coefficient of sliding friction, which occurs at 100% slip ratio. The sliding friction may be the half of the value of the critical slip speed. Fig. 3.6 represents a fundamental diagram for the anti-locking brake system (ABS), which tries to use the peak friction, avoiding the fully-locked wheel, which would provide less friction. Vehicles with ABS apply the brakes on and off repeatedly in order to held the slip near the critical speed. The braking is turned off before reaching the peak and turned on some time after, depending on the design of the manufacturer.

It is said that from zero to the peak value, the characteristics of the tyre mainly influence in the friction value, while, beyond the maximum value, the pavement surface is the most influencing factor in the performance (Fig. 3.7). Consequently, friction measurement equipments that operate at low slip speed are principally influenced by the tyre properties and, hence, specifications can be very tight. Friction measurement devices operating at relatively high slip speeds, the surface have a larger influence on the results than the tyre characteristics (Rado, 2000).

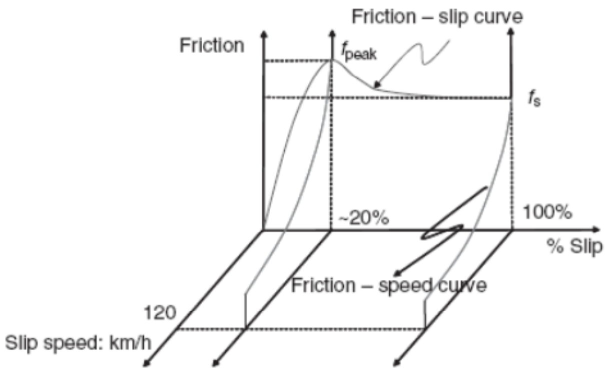


Fig. 3.5. Graph relating friction, slip ratio and slip speed (Solminihac et al., 2009).

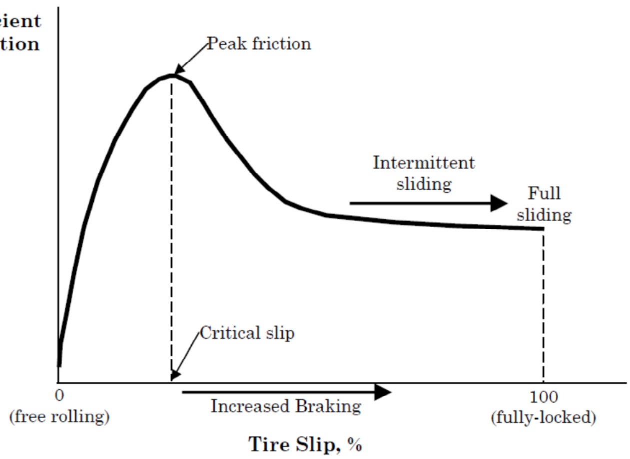


Fig. 3.6. Pavement friction versus slip rati (Hall et al., 2009).

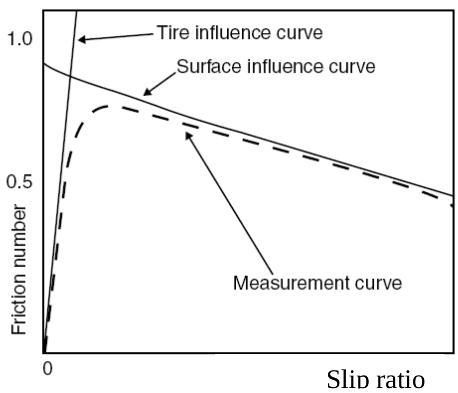


Fig. 3.7. Factors influencing Friction vs. Slip Ratio curve (Feighan 2006).

When analyzing friction coefficient with speed (slip speed), from measures from test devices, it can be observed that it decreases as the speed increases. Fig. 3.8 shows the typical nature of this relationship, where greater falls at lower speeds and shallower gradients at higher speeds. Generally, friction vs. slip speed is represented as an exponentially decreasing curve (Feighan, 2006, Austroad, 2011a).

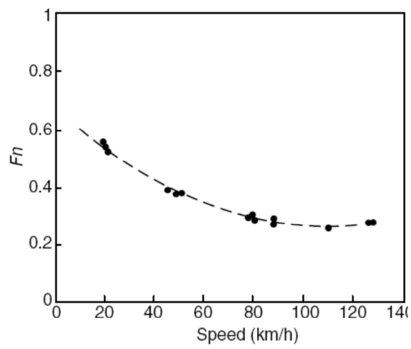


Fig. 3.8. Pavement friction, as Friction Number (section 3.3.9.2.3) versus slip speed (Feighan, 2006)

3.3.2.2. Lateral friction forces

When the vehicle steers around a curve or changes lanes the lateral or side-force friction force is generated at the tyre pavement contact (Shahin, 2005, Hall *et al*., 2009, Flintsch *et al*., 2012). The angle between the wheel and the direction of travel is called *yaw angle*. Fig. 3.9 represents the force-body diagram of a vehicle steering on a curve.

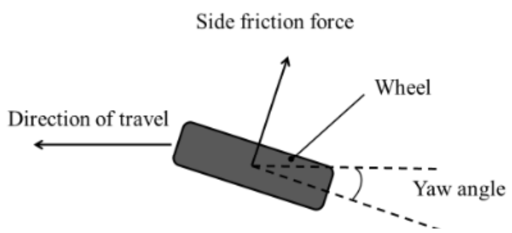


Fig. 3.9. Forces acting on a rotating wheel travelling around a constant radius curve at a constant speed. Author

According to Fig. 3.9, the side friction coefficient (SFC) can be obtained as the ratio between the sideways forces (F_y) and the vertical reaction between tyre and road surface (Fz) by Eq. 3.4 (Shahin, 2005):

$$SFC = \frac{F_y}{F_z} \tag{3.4}$$

The slip speed is calculated multiplying the slip and the yaw angle (Achútegi Viada, 2005) (Eq. 3.5).

$$S = V \cdot \sin(\alpha) \tag{3.5}$$

Where α is the yaw angle. Hence, the slip ratio in this case can be calculated as Eq. 3.6 (Flintsch *et al*., 2012).

$$SR = \sin(\alpha) \tag{3.6}$$

When a driver brakes and corners, both frictions are demanded. The application of longitudinal braking reduces the lateral force significantly and vice versa (Gillespie, 1992). This phenomenon is commonly known as the friction circle or friction ellipse (Radt and Milliken 1960), where the vector sum of the two combined forces (longitudinal and lateral) remains constant, following the shape of a circle or near constant (with the shape of an ellipse) (Fig. 3.10). If the required friction is within the limits of the tyre grip, the driver can control the vehicle. The form of the ellipse (or circle) depends on the tyre and pavement properties.

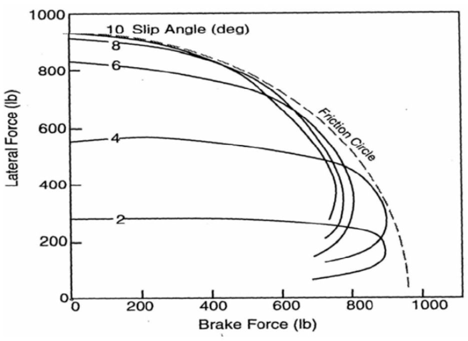


Fig. 3.10. Lateral force versus longitudinal force at constant slip angles (Gillespie 1992).

3.3.3. Friction mechanisms

Pavement friction is the result of a complex interaction between adhesion and hysteresis (Fig. 3.11). Adhesion is due to the molecular bonding between the tyre rubber and the pavement surface when they are in contact with each other, and it is a function of the interface shear strength and contact area. On the other hand, the hysteresis, or distortion of the tyre surface, occurs as the surface of the tyre passes over projections and depressions in the pavement surface. It results from the energy loss due enveloping of the tyre around the texture. When a tyre is compressed against the pavement surface, the stress distribution causes the deformation energy to be stored within the rubber. As the tyre relaxes, part of the stored energy is recovered, while the other part is lost in the form of heat (hysteresis), which is irreversible. That loss of energy implies a net frictional force to help stop the forward motion.

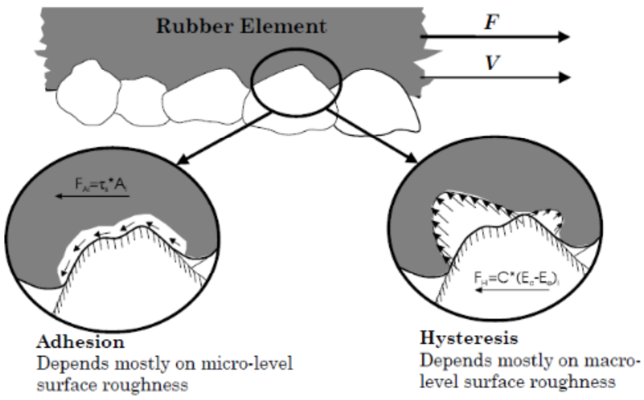


Fig. 3.11. Interaction of adhesion and hysteresis as main factors for pavement-tyre friction (Hall et al., 2009)

Other components take place in pavement friction. In a braking process, apart from adhesion and hysteresis, the shear also takes place (Fig. 3.12), only indicated for a non-rigid surface material (Hall *et al*., 2009).

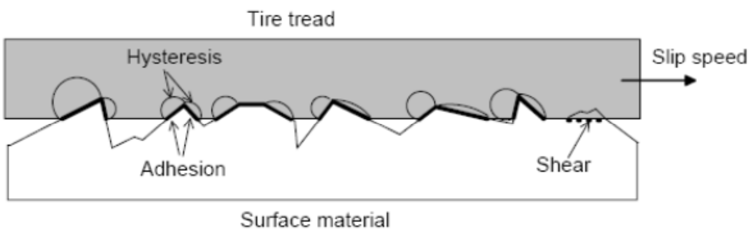


Fig. 3.12. Mechanisms of braking slip friction (Hall et al., 2009).

In Fig. 3.13 the usual contribution of the braking slip friction mechanisms are represented for different surface and the same tyre. Fig. 3.13.a) reproduces the contribution of each mechanism in a rigid surface, like a dry and bare pavement. Fig. 3.13.b) represents a wet pavement, and Fig. 3.13 c) represents a non-rigid surface material. In a rigid surface contact, the shear force is estimated as small, supposing that the adhesion and hysteresis contribute more than 80 or 90 % of the braking slip force. Parts of the tyre rubber are torn off when interfacing with a rigid surface, and hence, being the sacrificial part of the braking process. If shear force represent a major contributions, it means that the sacrificial component is sheared. The shear force can be calculated as the product of the ultimate shear stress of the surface material and the real area of shearing contact.

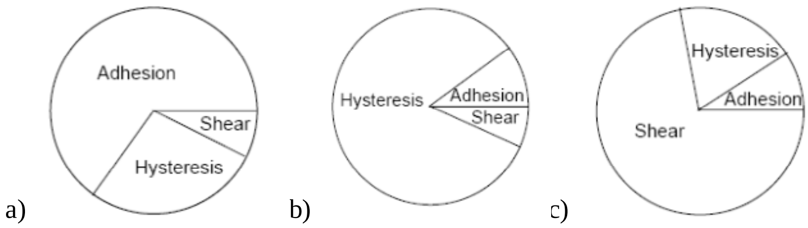


Fig. 3.13. Influence of each mechanism in a braking slip in different conditions (Hall et al., 2009).

As exposed, despite the fact that there are other components in the pavement friction, they are insignificant if compared with the adhesion and hysteresis forces. Thus, the friction (*F*) can be defined as the sum of the contribution of the adhesion force (F_{AD}) and hysteresis frictional force (F_{HY}) (Eq. 3.7).

$$F = F_{AD} + F_{HY} \tag{3.7}$$

As adhesion appears at the pavement-tyre interface, it is said to be related to the micro level asperities of the aggregates do the pavement, i.e. the microtexture. On the contrary, the hysteresis is due to macro level asperities of the surface, i.e. the macrotexture. Further description of the different classifications of the texture is provided in section 3.3.4.

3.3.4. Pavement surface texture

3.3.4.1. Definition

Pavement surface is defined as the deviations of the pavement surface from a true planar surface. These deviations appear at different levels of scales. The Permanent International Association of Road Congresses (PIARC) in the XVIII World Road Congress in 1987 in Brussels defined 4 levels of scales for the texture according to the wavelength, λ, and the peak-to-peak amplitude, A, as shown in Table 3.1 (PIARC, 1987). Fig. 3.14 shows the texture ranges that exist for a pavement surface (Sandburg, 1998).

Table 3.1. Classification of the deviations of a pavement (PIARC, 1987).

Level of texture	**Wavelength, λ (mm)**	**Amplitude, A (mm)**
Micro-texture	$0 < \lambda < 0.5$	$0.001 < A < 0.5$
Macro-texture	$0.5 < \lambda < 50$	$0.1 < A < 20$
Mega-texture	$50 < \lambda < 500$	$1 < A < 50$
Roughness or unevenness	$\lambda > 500$	$1 < A < 200$

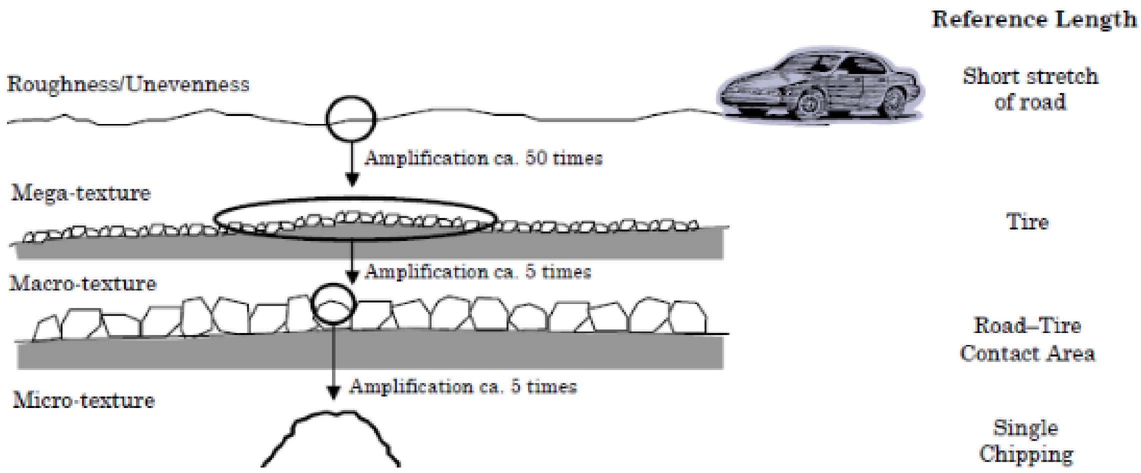


Fig. 3.14. Simplified illustration of the various texture ranges on a pavement surface (Sandburg, 1998)

Deviations with a wavelength below 0,5 mm are known as microtexture and they cannot be seen directly, but they can noticed by touch. They are placed at the sub-visible or microscopic level. It depends on the surface properties of the aggregates and on the bituminous or concrete mortar employed in the asphalt or concrete paving material. It is governed by the crystalline nature of the aggregate itself, i.e. the mineralogy.

Deviations with a wavelength between 0,5 and 50 mm are known as macrotexture, and can be seen directly. Macrotexture is a function of the coarser texture of the mixture properties of the asphalt paving mixture, defined by the shape, size and gradation of aggregates; and the method of finishing or texturing.

Irregularities with a wavelength between 50 and 500 mm are called megatexture, and have the same order of size as the pavement-tyre interface. They are due to cracking, potholes and other superficial defects.

Finally, irregularities in the profile with a range between 0,5 m and 50 m are called roughness or unevenness. Fig. 3.15 represents the influence of each surface texture level in each vehicle-road interaction.

The microtexture controls the contact between the tyre rubber and the road surface. It provides the primary source of frictional resistance at low speeds (V < 50 km/h) in all circumstances due to the direct contact between the pneumatic and the aggregate surface. Hence, in urban area the microtexture becomes a key factor.

The importance of the macrotexture arises in wet conditions. Each aggregate of the paving mixture can penetrate a water film on the pavement surface, while the spaces between aggregates create drainage paths for the water to be dispersed. It also contributes to the tyre wire and to the noise in high frequencies (Fig. 3.15).

As mentioned before, the macrotexture is a vital factor in tyre hysteresis. Although hysteresis effect to convert kinetic energy into heat is much smaller than the braking actions, it has demonstrated that hysteresis losses make a significant contribution to energy dissipation before skidding occurs, reducing the probability of an accident or its severity. (Roe *et al*., 1991). Therefore, it is said that the macrotexture is the main factor for friction resistance at higher vehicles speeds (V > 50 km/h). Fig. 3.16 shows the influence of these factors on the friction.

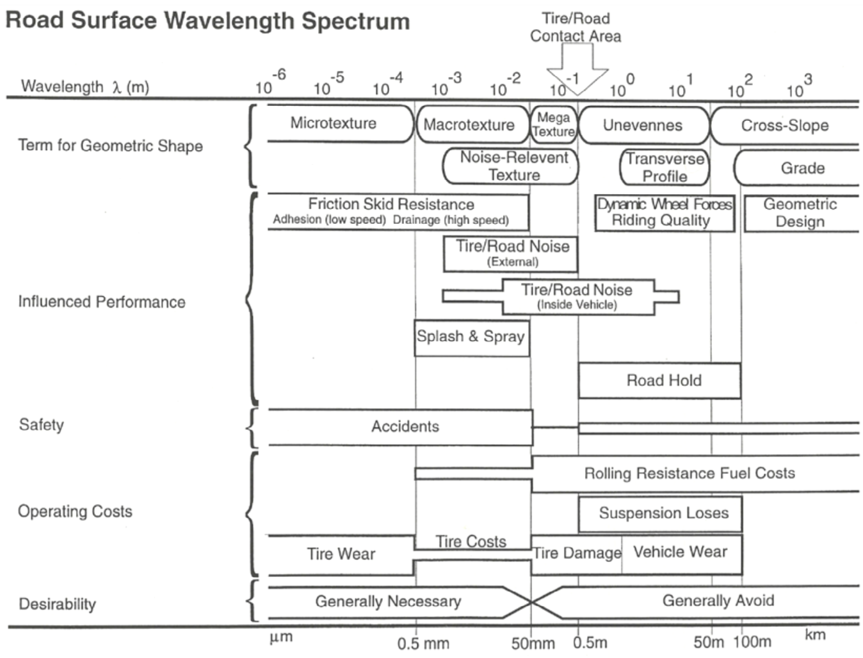


Fig. 3.15. Influence of surface texture levels in different vehicle-road interactions. (PIARC, 1987)

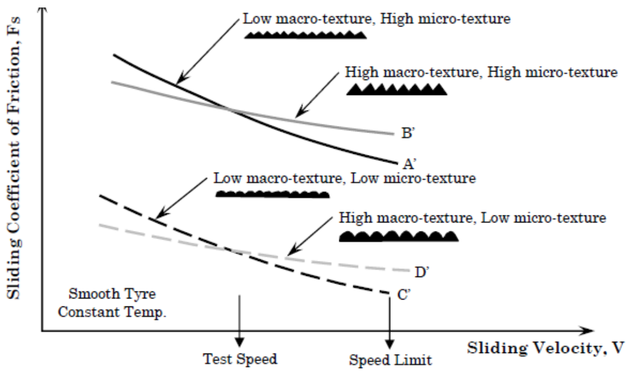


Fig. 3.16. Effect of microtexture and macrotexture on pavement-tyre friction at different sliding speeds (Flintsch et al,. 2002).

There are two types of macrotexture: positive and negative. The aggregates protruding above the plane of the road surface are referred as positive macrotexture. This kind of macrotexture is provided in the usual bituminous mixes, like surface dressing, and hot mix asphalts. Other materials, such as porous asphalt and other thin-surfacing materials create negative texture, which is composed of voids below the plane of the road surface. These voids may be interconnecting or not. In a porous asphalt material, the voids are interconnecting and, hence, water can drain laterally below the pavement surface. In both cases, the macrotexture is able to provide a safe and rapid dispersion of the water away from the tyre-pavement contact.

Nevertheless, it must be said that microtexture also has some influence on frictional resistance at higher speeds, and macrotexture also at lower speeds. Roe *et al.*, (1998) highlighted that the level of high-speed friction of surfacing depends on the microtexture, and that the texture depth (a measure of macrotexture) influence in friction loss at low-speeds (Fig. 3.17).

On the other hand, while the macrotexture also contributes in a negative way to the consumption of combustible as it increases the resistance to the movement and to the noise, it improves the visibility and reduces the water projections, and allows a better contrast to the horizontal marks (Kraemer *et al.*, 2004).

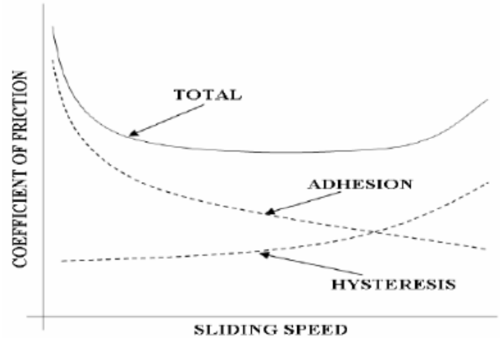


Fig. 3.17. The contribution of adhesion (mainly dependent on microtexture) and hysteresis (mainly dependent on macrotexture) to the friction coefficient as a function of sliding speed (Khasawneh, 2008).

Therefore, despite the inconveniences pointed out previously, adequate microtexture and macrotexture for frictional resistance on all the range of vehicle speeds. In wet condition, at high speeds, the macrotexture provides drainage paths for the water and provides particle to tyre rubber contact. As the vehicle slows due to the frictional resistance, the interaction between the tyres and the aggregate faces (the microtexture) becomes more important in providing the required frictional resistance to stop the vehicle. On the contrary, as shows in Fig. 3.15, the megatexture and unevenness must be avoided in all cases.

3.3.5. Factors affecting available pavement friction

Factors affecting pavement friction are usually classified in four categories: pavement surface characteristics, vehicle operational parameters, tyre properties and environmental factors (Wallman and Astrom, 2001). Table 3.2 lists the main factors in each category. As each factor in Table 3.2 has an effect on the pavement friction, it can be concluded that friction is as a process and not as an inherent property of a road pavement. Once all the factors are specified, friction takes on a definitive value (Hall *et al*., 2009).

As explained previously, the most critical factor for available friction is whether the pavement surface is wet or dry. Under dry conditions, skidding resistance is very high and similar in almost all the kind of surfaces. However, there is significant drop in friction values if the surface gets wet, even slightly; and greater as the water film thickness becomes wider. Consequently, the vast majority of skid resistance measuring tests are conducted under self-wetting conditions to know the lower range of friction reached by a wet road surface.

Table 3.2. Factor affecting available pavement friction (adapted from Wallman and Astrom, 2001)

Pavement surface characteristics	Factors related to the vehicle	Tyre properties	Environment
Microtexture	**Slip speed**, as a function of:	**Tread design and condition**	**Temperature**
Macrotexture	- Vehicle speed, *V*	**Inflation pressure**	**Water** (rainfall, condensation)
Mega-texture / unevenness	- Slip ratio, *SR*	Rubber composition and hardness	**Snow and ice**
Material properties	Driving maneuver:	Foot print	Wind
Temperature	- Turning	Load	**Contaminants:**
	- Overtaking	Temperature	- Salt, sand
			- Dirt, mud

Note: Key factor in each area are shown in bold.

Related to pavement surface characteristics, as previously explained, microtexture and macrotexture are key factors due to their contribution to the adhesion and hysteresis phenomena. At the same time, different values of the texture depend on the material properties of the surface layer, such as the type of aggregate, mix characteristics and texture patterns in the case of concrete pavements. The resistance to be polished of these materials define the behaviour of the surface to resist the abrasion of traffic and environmental factor. Further detail is provided in section 5.3.

Regarding to vehicle operating parameters, slip speed is the most important factor and, as exposed in subsection 3.3.2, it has a maximum with a slip ratio between 10 and 20 % (Fig. 3.4). After that value, friction decreases to a value known as the coefficient of sliding friction, achieved with a slip ratio of 100 %.

Referred to tyres, the tyre treads and condition helps draining water on the pavement surface by means of the channels of the tyre tread. Therefore, a sufficient depth of the tread is necessary for vehicles at high speeds. Used pneumatics can reduce the wet friction until 70 % the friction provided by new tyres (Henry 1983).

Tyre pressure affects significantly the friction too. An under inflated pneumatic allows a concave figure of the centre of the tyre tread, causing a reduction of the drainage channels and reduction of contact pressure. On the other hand, an over inflated pneumatic does not reduce so significantly pavement friction (Henry 1983; Kulakowski *et al.*, 1990). It provides a narrower area of contact with the pavement and reduces the tapping effect and yield higher pressure for water under the tyre.

With regard to environmental causes, temperature affects the properties of the pneumatic components, e.g. thermal conductivity and specific heat. It has been demonstrated that friction decreases as temperature increases, but no quantified. Further description is provided in section 5.3. Water has a great influence on pavement friction, which is measured by means of the water film thickness, *WFT*. Its effect at low speeds ($V < 32$ km/h) is insignificant, but can be important at higher speeds ($V > 40$ km/h). As the *WFT* increases, the friction decreases exponentially (Henry 2000). Even a minimum quantity of water can reduce the friction: 20-30 % of reduction with a water film of 5 mm. (Harwood *et al*., 1987). Hydroplaning is a phenomenon that can appear when vehicle tyre and pavement surface are non in contact due to the water film pressure that builds up at the contact, resulting in friction values near zero. More analysis is shown in section 5.3. Snow and ice can be a problem for vehicle when braking or cornering. If these materials cover the road, no friction can be guaranteed (Al-Qadi *et al.*, 2002) Further explanation is given in section 5.3..

There is a wide variety of contaminants that can appear on the road: dirt, sand, oil, salt, etc. All of them have an adverse effect on the friction, as they act as lubricants, reducing the friction. More detailed investigation on contaminants is presented in section 5.3.

3.3.6. Skidding resistance measurement devices

There are two main procedures to characterize the skidding resistance of a pavement (Achútegi Viada 2005):

- Measuring directly the friction coefficient between the tyre and the wet pavement surface at a determined speed
- Measuring the macrotexture or the drainage capabilities of the pavement surface

There is a wide variety of skidding resistance measurement devices, which can be classified according to different criteria. The subgroups are not exclusive, and hence, the same tool can be placed in different subgroups according to a property. These are some of the classification that can be made (Achútegi Viada 2005):

a) According to the measured characteristic:
   - Friction
   - Texture
   - Friction and texture (equipment with double function)

b) According to the way of displacement:
   - Manual
   - Towed
   - Incorporated to a vehicle

c) According to the displacement speed:
   - Stationary
   - Mobile (slow or quick)

The last classification is related to the type of research that is carried out. Stationary devices are used in laboratory or researching analysis, but they are not suitable for network data collection in pavement management system. Therefore, for data collection in PMS, high performance equipments are necessary in order not to disturb traffic flow (Achútegi Viada 2005).

There are two main groups between the equipment used to measure the friction, according to the gadget they use to measure it; a sliding rubber or a tyre, which is usually accepted (Achútegi Viada 2005):

a) Rubber sliders
   - Pendular movement
   - Rotational movement
   - Lineal displacement

b) Sliding tyres
   - Locked wheel devices
   - Fixed slip devices
   - Variable slip devices
   - Sideway-force devices

Fig. 3.18 classifies the skid resistance measuring devices with the most employed testers.

3.3.6.1. Static/portable testers based on rubber sliders

These devices measure the friction coefficient or skidding resistance produced when a rubber pads slides on a pavement surface under prefixed conditions, by determining the loss in kinetic energy. The most representative devices are the British Pendulum Tester and the Dynamic Friction Tester (DFT). Both are portable and easy to handle.

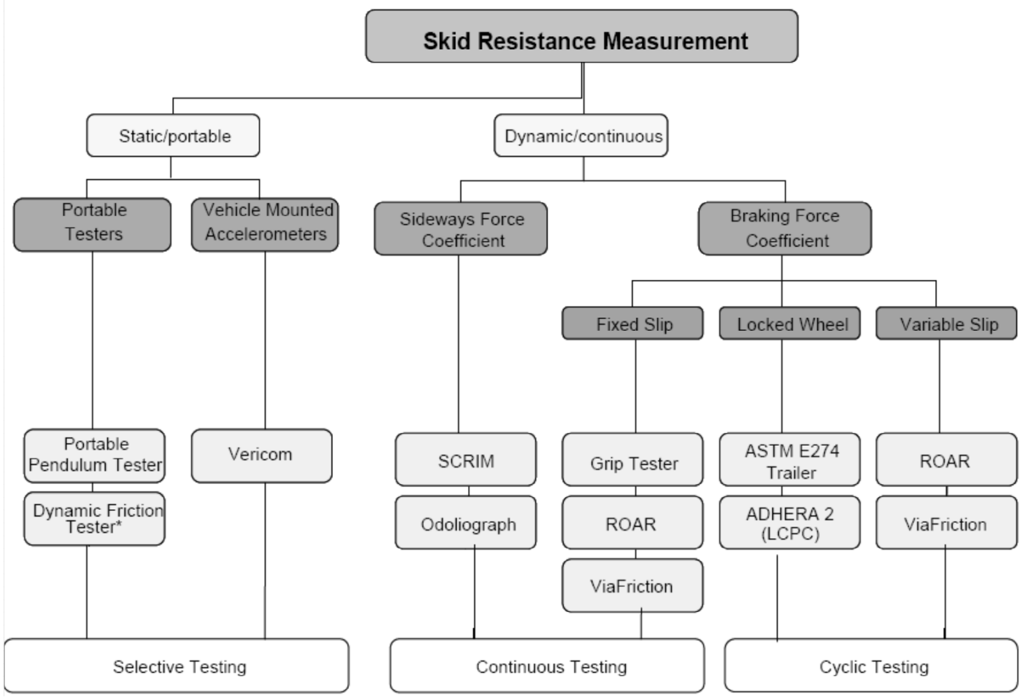


Fig. 3.18. Classification of Friction measuring contact methods and the most used devices in each group. (Wilson, 2006).

3.3.6.1.1. British Pendulum Tester

After a series of tests with rubber sliders at low speed on pavements and on road marks carried for years, in 1952 the British Administrations decided to develop a pendulum in order to measure the skid resistance on pavements. After several tests and verifications, a portable pendulum was designed and it has started to be used from the 60s (Achútegi Viada 2005).

The British Pendulum Tester (BPT) is specified in ASTM E303 (ASTM, 2013) and AASHTO T 278 (AASHTO, 2017). The BPT has a swinging arm with a rubber slider base (Fig. 3.19). The arm is released from a specified height, and allowed to swing down, following the movement of a pendulum. At the lowest point of the arc, the rubber slider comes in contact with a prepared surface of aggregate or with a real pavement surface to be tested. If a greater friction is developed between the rubber slider and the tested surface, the swing of the pendulum is retarded more. There is a drag pointer in the pendulum arm, which shows the height reached but the pendulum arm after it has been in contact with the test surface. The elevation to which the arm swings after the contact provides the value of the friction. A low height measurement means that a great friction is available. Data from five tests are usually collected and recorded by hand. The height measurements are expressed as British Pendulum Number (BPN) values.

The BPT measures the energy loss produced when a standard force, achieved by dropping a specified weight at the bottom of the swing arc from a standardized height, across the test surface, propels the rubber slider. Because of the low speed, around 10 km/h, the friction is directly related to the microtexture of the aggregate surface. Hence, the British Pendulum Tester is said to measure the friction and the microtexture. It can be used in a laboratory with prepared samples or in the field in any pavement and it can be employed to measure both longitudinal and lateral pavement-tyre (sideway force) friction (Fig. 3.19). As the test is stationary, if it is carried out in a road, the lane must be closed to traffic.

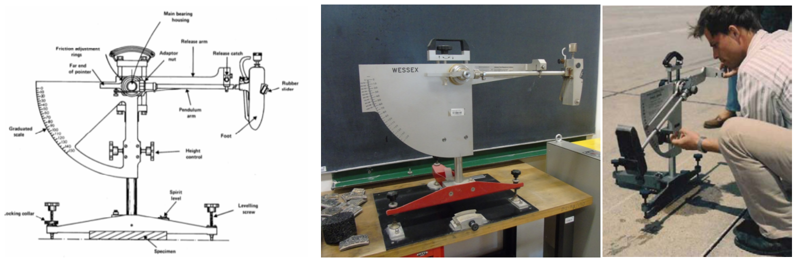


Fig. 3.19. Schematic of British Pendulum Tester for use in laboratory (left) (AUSTOROADS, 2011a), British Pendulum Tester in a laboratory (middle) (author) or measuring in situ (Hall et al., 2009)

3.3.6.1.2. Dynamic Friction Tester

Another stationary test device is the Dynamic Friction Tester (DFT), developed in Japan. It is specified under ASTM E 1911 standards (ASTM, 2009a). It also determines the loss in kinetic energy, in this case of a rotating disc, when it is contact with the pavement surface. The DFT is equipped 3 rubber sliders (Fig. 3.20), similar to the British Pendulum Tester, and measures the torque necessary to rotate the spring-loaded rubber sliders in a circular path over the pavement surface at different speeds, from 5 to 89 km/h. Rotational speed, rotational torque and downward load are measured and recorded electronically. It needs a water supply, 3,6 l/min, usually by means of a water tank and a portable computer.

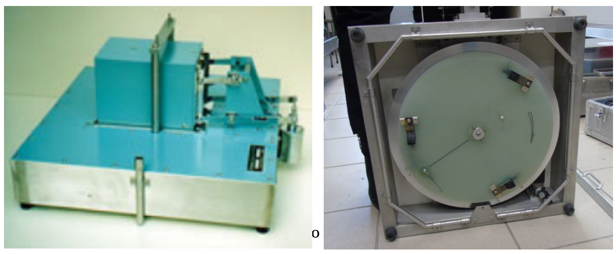


Fig. 3.20. DFTester unit (left) (Wilson, 2006) and DF Tester spinning disk and rubber sliders (author)

It is usually used in field, in a closed lane, or in laboratory testing. The DFT provides good repeatability and reproducibility and it is not affected by operator experience or wind. It provides friction values (coefficients) at different speeds, including high speeds. Consequently, the graph speed versus available friction can be obtained (Fig. 3.4), as shown in Fig. 3.21) and, hence, it can establish the peak value (Saito *et al*., 1996; Himeno *et al*., 2000). It can be used for IFI statistics (section 3.3.9.4.1), as it was included in the International Experiment of the PIARC (1995). However, it was not assessed in the HERMES project (Descornet *et al*., 2006) It has a good correlation with BPN, but it cannot be employed for network evaluation.

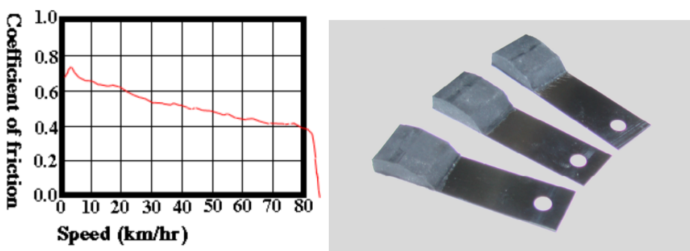


Fig. 3.21. a) DF Tester rubber sliders (Wilson, 2006); b) DF Tester typical result output (Wilson, 2006)

3.3.6.2. Sliding tyres

Slider devices previously described have the disadvantage of being stationary, and, hence, the test is slow and needs lane closure. Therefore, when high performance is required, devices are included in vehicles or towed. They can be divided according to the measure provided: the longitudinal friction coefficient or de side force coefficient. Longitudinal friction measurement conducts measurements from a braked test wheel travelling in a straight line. This kind of devices measure reaction forces developed in the tyre/road contact area and aim to simulate tyre skidding over the surface by controlling the slip ratio. They are divided in locked wheel, fixed slip and variable slip devices. On the contrary, side force friction measuring equipments are based on measuring side force friction for a vehicle travelling in a curve when the vehicle’s front wheels are turned so that there is an angle between the vehicle direction and the rotation plane of the wheel. These devices try to represent reaction forces developed n the tyre/surface contact area and simulate angular tyre slipping over the surface, by controlling slip or yaw angle..

3.3.6.2.1. Locked wheel testers

This kind of devices measures the force on a tyre that is sliding but not rotating. Hence, as the wheel does not rotate, the sliding speed is the same as that of the test vehicle, so the slip ratio has a value of 100 % (Fig. 3.4). The test wheel is braked, becoming fully locked. It simulates the emergency braking process for non-ABS vehicle brake systems. Among locked wheel testers, the most employed device is the ASTM E-274 Locked Wheel Tester (Feighan, 2006, Hall *et al*., 2009). Other used devices are the French ADHERA and Skiddometer BV8 and SRM. In Spain, this kind of devices is not used (Achútegi Viada, 2005).

3.3.6.2.1.1. ASTM E-274 Locked Wheel Tester

This type of equipment is very common in the USA, where all the 50 states employ it (Henry 2000, Hall *et al*., 2009). Some countries of Europe and Asia employ equipment, too. The equipment specification is exposed in the ASTM E247-06 standard (ASTM, 2006).

The test wheel may be built into the testing vehicle or as part of a trailer towed by the testing vehicle, as the Dynatest 1295 Pavement Friction Tester (PFT) shown in Fig. 3.22. Usually deployed testers are trailer that have two testing wheels built into the trailer. The test tyre may be a ribbed tyre, according to ASTM E501-08 (ASTM, 2015a) of a smooth tyre, according to ASTM E524-08 (ASTM, 2015b). Wet and dry conditions are possible but it is generally conducted in wet conditions, with water supplied from a storage tank incorporated in the trailer at a specific rate. The test can be performed at any speed, but the standard requires that a speed between 64 and 100 km/h must be maintained with a deviation of ± 1 km/h from the specified speed. The ASTM E247-06 standard (ASTM, 2006) specifies a test speed of 64 km/h (40 mi/h).

The procedure is as follows: The test vehicle runs at the desired speed, water is supplied ahead of the test tyre and brakes locks the test tyre. When the tyre is totally locked, the force measurement system measures and averages the vertical and longitudinal forces over 1 second period. Then, some type of force transducer measures the frictional force. At the same time, test vehicle speed is measured, too (Fig 3.26). The obtained value is the Skid Number (*SN*) or friction number (*FN*), which is further described in section 3.3.9.2.3. Some of the characteristics of the test are summarized in Table 3.3.

Fig. 3.22. ASTM E-274 locked wheel device, towed by testing vehicle (Wilson, 2006).

Table 3.3. Standard test-conditions for ASTM E-274 Locked Wheel Tester (ASTM, 2006).

Characteristic / property	**Values during the test**
Pavement status	No pollution
Test wheel	ASTM E501-08 ribbed tyre or ASTM E524-08 smooth tyre, inflated at 0.165 MPa
Condition of tyres	Run at normal traffic speeds for at least 300 km before tests.
Method	Locked wheel (SR = 100 %)
Static wheel load	4800 N ± 65 N
Operating speed	Test speed, 64 km/h (40 mi/h). Also 64 - 100 km/h
Theoretical film water thickness	0,5 mm

Five stages can be observed during the test. When the vehicle is at desired speed, the stages are (Roe *et al.*, 1998):

- Start water pump before testing (0,5 second)
- Brake test wheel from rolling to locked (usually less than 2 seconds)
- Allow locked wheel to settle (0,5 second)
- Calculate locked wheel friction (at least 1 second)
- Allow test wheel to spin back up to vehicle speed

The load and drag force are measured every 0,01 second during the fourth stage and averaged over the 1 second interval (stage 4). After the wheel reaches a free rolling state again, a new measure can be conducted.

The ribbed tyre is insensitive to the pavement surface water film thickness, and hence, insensitive to the macrotexture. On the other hand, the smooth tyre is more sensitive to the macrotexture changes and the ribbed tyre is more sensitive to microtexture changes (Hall *et al*., 2009). It can be deployed in field testing or for network level friction management.

The main advantage of these devices is that slip speed of the locked wheel is the same as the testing vehicle speed, and hence, the obtained frictional value is similar to the experienced by a driver when braking on the same pavement surface. Nonetheless, the main disadvantage is that fully locking stress enormously the test tyre and it is not practical or realistic to measure continuously the skid resistance over a whole road network. Measures are undertaken at spot locations not continuously, assuming that the road surface is largely homogeneous (Wilson, 2006). Consequently, other devices were developed to be able to continuously measures friction over the road network. Moreover, it can be only used on straight elements, not in curves or roundabouts. As the measurements are intermittent, some slippery sections can be missed (Hall *et al.*, 2009).

3.3.6.2.1.2. ADHERA

In France, another locked-wheel equipment is used, ADHERA, (Fig. 3.23) and was also employed in the International Experiment of the PIARC (1995). ADHERA is a single-wheeled trailed towed behind a vehicle carrying water and the recording equipment. It is supposed to represent a quarter of a passenger car. Although it measures longitudinal friction coefficient by means of a locked wheel (SR = 100 %), the slip ratio can be modified between 0 to 100 %, for research use.

Fig. 3.23. General view (up left) and towed trailer with cover opened (up right) (Do and Roe, 2008).

It simulates a locked braking situation. Braking sequences consist of braking and free-wheeling sections at specific test speeds, generally 40, 60 and 90 km/h on main roads and 60, 90 and 120 km/h on motorways. It measures horizontal and vertical forces..

3.3.6.2.1.3. Other locked wheel devices

Another locked wheel device is the Skid Resistance Tester, SRT-3, with a patterned tyre, from Poland, which took part in the International Experiment of the PIARC (1995) and in the HERMES project (Descornet *et al.*, 2006) Other testers are the SRM (Stuttgarter Reibungsmesser), developed by Forschungsinstitut für Kraftfahrwesen und Fahrzeugmotoren Stuttgart (FKFS) (Fig. 3.24a) and the Skiddometer BV-8 (Fig. 3.24b), developed by the Statens Väginstitut (National Swedish Road Research Institute) of Stockholm (Do and Roe, 2008). These two devices can also operate at fixed slip.

Fig. 3.24. a) SRM mounted on the rear part of the vehicle, b) Skiddometer BV-8 (Do and Roe, 2008).

3.3.6.2.2. Fixed slip devices

Contrary to locked wheel tests, fixed slip devices can measure continuously frictional resistance over an entire road network as a small slid ratio is selected. Normally, fixed slip equipment operates between a slip ratio of 10 and 20 %. For example, if the vehicle test is moving at 70 km/h, and the selected slip ratio is 20 %, according to Eq. 3.6, the tyre speed is 14 km/h. With a slip ratio between 10 and 20 %, values near the friction peak (maximum value) are achieved. Moreover, as this kind of tool usually operates at low speed (with some exceptions), values are mainly conditioned by the microtexture of the pavement surface (Fig. 3.7)

(Feighan 2006). Test tyre rotation is inhibited to selected percentage (slip ratio), by a chain or belt mechanism or a hydraulic braking system. Tyre loads and frictional forces are measured by means of force transducers or tension and torque measuring devices. Some of the devices included in this category are the Griptester, the Swedish Skiddometer BV-11 or Saab Friction Tester (SBT), and the RWS NL from the Netherlands.

3.2.6.2.2.1. Gripstester

The Griptester is said to be the representative of fixed slip devices (Feighan 2006; Wilson 2006; Do and Roe, 2008; Sandberg 1998; Wallman and Aström, 2001; Austorad 2005a; Hall *et al*., 2009). It was developed by Findlay Irvine Ltd in the UK in 1980s and its standards were referred in the British Standard specification, BS 7941-2 (BSI, 2000b) and after a European Level Standardization, in CEN/TS 15901-7 (CEN 2009b).

The Griptester, originally design for airport runway operations (Cenek and Jamieson 2000), is a small trailer-towed equipment, approximately 1 m in length and 0,8 in width, that can be used on roads and airports. It can even be employed manually (pushed) in areas inaccessible for vehicles like footpaths or cycling lines due to its small size (Fig. 3.25) (Feighan 2006; Austroad, 2011a).

Fig. 3.25. Griptester operating. (Feighan 2006; Kogbara et al., 2016).

When performing roadway measures, the operational speed ranges from 5 km/h to 100 km/h, as long as the trailer is constantly in contact with the road surface. Frictional measures are collected continuously, and resulted are provided after averaging values for a specified interval, usually 10 or 20 m long. The trailer has three wheels, with a 254 mm diameter, in a tricycle disposition, constantly in contact with the pavement surface (Fig. 3.26). Two of the wheels are drive wheels, standard and with patterned tyres. The single test wheel is mounted on a stub axle and has a smooth tread. This test wheel is mechanically braked by a gearing system linked to the drive wheels (by a chain or belt mechanism or a hydraulic system), with a gear ratio of 27:32 in relation to the drive wheels, and hence maintaining a fixed slip ratio of 15 %. Force transducers or tension and torque measuring devices measure wheel loads and frictional forces. In Table 3.4 values during the test are shown.

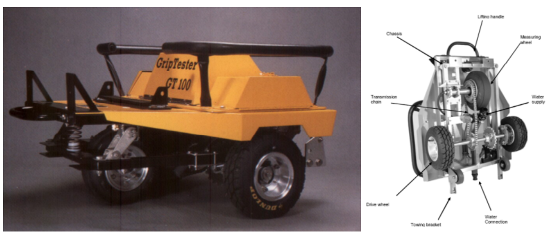


Fig. 3.26. Side view of the GripTester (left) (Wilson 2006) and GripTester measuring wheel and chain transmission (right) (Do and Roe, 2008)

Table 3.4. Standard test-conditions for GripTester (CEN 2009b)

Characteristic / property	Values during the test
Air temperature	> 4 °C
Pavement temperature	> 5 °C and < > 50 °C
Pavement status	No pollution
Test wheel	Smooth ASTM-tyre 254 mm in diameter inflated at 0,14 MPa
Method	Constant slip ratio, 15 %
Static wheel load	250 ± 20 N
Operating speed	Test speed: 50 km/h. Also from 5 to 100 km/h
Theoretical film water thickness	0,5 mm
Length for the mean value	Optional, usually 10 m to 20 m
Wheel path	Nearside wheel path or as required

For normal wet road tests, water is deposited in front of the test tyre from a water tank with a control valve, controlling the amount of water. In towing mode, water flow rate is further controlled by a pump and may be monitored with a flow meter.

The equipment provides summary GripNumber outputs averaged over specified intervals. Further explanation is provided in subsection 3.3.9.2.5. It can be employed in field testing (straight segments), and for network level and project level monitoring.

In Griptester, as the slip ratio is not over 20 % values are mainly influenced by the microtexture (Fig. 3.7). In Spain, the Griptester was employed, with a fixed slip rate of 14,5 % or 15 %, according to the model (Achútegi Viada 2005).

3.2.6.2.2.2. BV-11 and Saab Friction Tester

This device can be built either as a towed trailer (Skiddometer BV-11) or built into a vehicle (Saab Friction Tester) (Fig. 3.27). The measuring wheel is located between two reference wheels. It is engineered to give a fixed slip ratio of 17 %. The wheel slips as it is towed along the wetted pavement surface at a constant speed and the slipping force is measured. Friction measurements are conducted by a sensor that provides continuous data, which are collected, processed and stored. The water control enables to provide a constant water film thickness of 1,0 mm, as usually required.

Fig. 3.27. Towed BV-11 device (right) (Achutegi Viada 2005) and Saab Friction test (left) (Do and Roe, 2008).

3.2.6.2.2.3. RWS NL Skid Resistance Trailer

The Dutch RWS skid resistance trailer is a single axle trailer with a measuring wheel mounted in the middle of the road wheels (Fig. 3.28). Water and control equipment are placed in the towing vehicle. The measuring wheel works at fixed slip ratio (86%), connected via a mechanical transmission to one of the bearing wheels.

Fig. 3.28. RWS NL skid resistance trailer (Do and Roe, 2008)

3.3.6.2.2.3. ROAR DK and ROAR NL

The ROAR DK and the ROAR NL are the version of the ROAR device (Section 3.3.6.2.3.1), which operate in Denmark and in the Netherlands, respectively. While the Danish version is mounted within in a trailer with drive wheels and single loaded test wheel and measures at a sip ratio of 20 %, the Dutch version is a three axle tanker truck with two measuring systems mounted at the rear of the chassis and measures at a slip ratio of 86 %. Both testers can work with a variable slip ratio and took part in the HERMES project, but not in the PIARC Experiment (PIARC, 1995).

3.3.6.2.3. Variable slip devices

Variable Slip Devices are designed to be able to measure at any specified slip ratio. They can carry out automated measurements over a range of specified slips, and even they can be programmed to look for a maximum friction level through the specified range of slip ratios. They are able to measure friction as a function of slip in the whole range, from 0 to 100% SR. In the test, wheel speed is reduced gradually while vehicle speed, travel distance, tyre rotation speed and wheel load are recorded every 2,5 mm. All these raw data are subsequently filtered to be reported. Consequently, the variable slip devices are also very useful to calculate the IFI (section 3.3.9.4.1) as obtained values by means of the variable slip devices are directly used in IFI calculations, because the underlying approach to the IFI derivation is closely related to the variable slip device approach (Feighan 2006, Wilson, 2006). No variable slip devices are employed in Spain (Achutegi Viada 2005).

Some of the devices classified in this group are Japanese Komatsu Skid Tester, ROAR, SALTAR Dagmar/Petra trailer, being the ROAR device, manufactured by Norsemeter in Norway, the most employed equipment (Feighan, 2006; Hall *et al.*, 2009, Wilson 2006; Wallman and Aström, 2001).

3.3.6.2.3.1. Norsemeter Road Analyser and Recorder (ROAR)

The Norsemeter ROAR is a trailer-towed device, with a similar size than the Griptester (Fig. 3.29). Its standards are described in the ASTM E1859/E1859M-11 specifications (ASTM, 2015c). This standard can be applied to equipment operating in fixed slip or variable slip modes. ROAR is a compact unit with a single test wheel, mounted to a host vehicle (Fig. 3.29).

In few words, the device can control and maintain the rotational speed of the test wheel at a fixed slip ratio gradient, similarly to the fixed slip devices. Nevertheless, in this case, variable slip devices can establish a wide range of slip ratio gradients by means of feedback sensors, which can measure the rotational velocity of the test wheel and modify the braking procedure in the test wheel. Generally, the brake systems are able to control the rotational velocities in a range of ± 7 % and a deviation of ± 2 % for any specified slip ratio. The obtained value is calculated as the ratio of the longitudinal friction force to the vertical load force, at any slip ratio, as in other devices. It can be presented the relationship of the friction number against slip speed over a range, being able to identify the peak slip factor as the maximum value recorded and the corresponding critical speed (Fig. 3.6). It can also provide the Rado shape factor for detailed evaluation (section 3.3.8.4). It can also test at different slip speeds (at any fixed slip ratio) by changing the survey speed. Nonetheless, it is a large and complex equipment that requires high maintenance costs and complex data processing and analysis. It can also be employed in field testing (straight or curved segments), and for network level and project level monitoring. However, variable slip mode testing is not carried out continuously but at a certain distance interval (Austroad, 2011a). Main test conditions are listed in Table 3.5.

Fig. 3.29. a) ROAR device with testing wheel and hydraulic braking mechanism (Wilson 2006), b) ROAR operating (Austroad, 2011a).

Table 3.5. Standard test-conditions for ROAR Norsemeter (ASTM, 2015c).

Characteristic / property	**Values during the test**
Test wheel	ASTM 1551 smooth tyre, 400 mm diameter, inflated at 0,207 MPa
Method	Variable slip, from a free rolling tyre (0 % slip ratio) to a fully locked wheel (100 % SR). It can also work at constant slip ratio
Static wheel load	1500 N
Operating speed	Test speed 50 km/h, possible from 20 to 130 km/h
Theoretical film water thickness	0,5 mm
Length for the mean value	Generally. 7 m
Wheel path	Movable, it can masure in the left, right or centre wheel track, or at any point in between ()

It was employed in the International Experiment of the PIARC (1995).As previously discussed, any slip ratio can be indicated. For example, in New Zealand, a fixed slip ratio of 34 % is established when using the Roarmeter in order to give a good correlation with Gripstester and with SCRIM.

3.3.6.2.3.2. Other variable slip speed devices

Other variable slip speed devices are the Japanese Komatsu skid tester and the Dagmar/Petra trailer from Denmark. The Komatsu skid tester employs a smooth tyre and has a working range from 10 to 30 % of slip ratio (Achutegi Viada, 2005). The second one is a patterned one. Both testers participated in the International Experiment of the PIARC (1995).

The VTI Skiddometer BV12, developed by VTI in Sweden, has been only used for research goals Test wheel can be progressively braked through a sequence of SR from 0 to 100 %. Moreover, the tyre can be rotated to change the slip angle (yaw angle) up to 20° in either direction.

Finally, the SALTAR device was developed initially for measuring skid resistance on ice and snow without adding supplemental water with a patterned tyre. It measures with variable slip ratio from free-rolling to locked wheel (0 – 100 % SR).

3.2.6.2.4. Sideway force devices

Sideway force devices aims to measure continuously the frictional resistance of the pavement, like the fixed slip devices, but they employ a different principle. Sideway force systems maintain the test wheel with its vertical plane at an angle to the longitudinal plan of the test vehicle. The test wheel is allowed to freely rotate, and it is maintained in a controlled slipping condition by means of the angular difference between the plane of rotation of the test wheel and the plane of the vehicle.

The relative speed between the test tyre and the surface is quite low, as it is calculated by means of Eq. 3.8,

$$S = V \cdot \sin\alpha \qquad [3.8]$$

where V is the speed of the vehicle and α is the angle between plane of the test wheel and the longitudinal plane of the test vehicle. For example, the SCRIM devices run at a test speed of 50 km/h and the yaw angle is 20 degrees. Hence, the sliding speed has a value of 17 km/h.

Some of the most deployed sideway force devices are the Mu-Meter and the Sideway force Coefficient Routine Machine (SCRIM). As the last one is the one used in the Pavement Management System of Biscay, it is further described in next section.

3.3.6.2.4.1. SCRIM

The Sideway-force Coefficient Routine Investigation Machine (SCRIM) is by far the most employed sideway-force measuring device. It is a British device. Its predecessor was a motorcycle with a sidecar, where its wheel was yawed (Fig. 3.30) (Salt 1977).

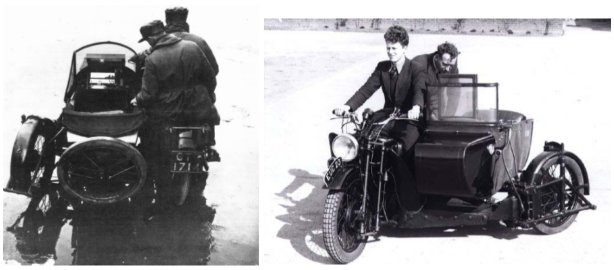


Fig. 3.30. Early motorcycle and sidecar for measurement of sideway force coefficient in 1929 (Salt 1977, Roe and Sinhal 2005)

Later, some devices with a fifth wheel installed in automobiles from different firms (Fig. 3.31) were developed until 1968 when they produced the prototype of the SCRIM on a truck (Fig. 3.32). The present device was produced in the early 1970s in Great Britain. The SCRIM is mounted on a truck chassis, with a large capacity water (around 4000 l) (Fig. 3.33). It is designed for measuring large lengths of road network, being able to measure around 200 km per day.

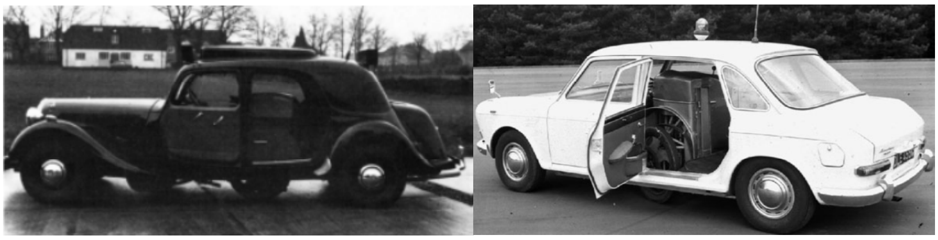


Fig. 3.31. a) Citroën car for measurement of SFC (until 1960), b) Other car for SFC measurement (Roe and Sinhal 2005)

Fig. 3.32. Prototype of the SCRIM (1968) (Salt, 1977).

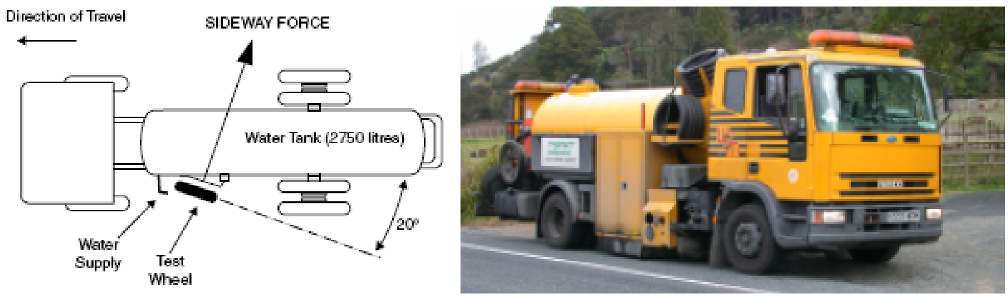


Fig. 3.33. a) SCRIM diagram, b) SCRIM truck (Wilson, 2005)

The SCRIM can be single-sided or double-sided. If it is doubled-sided, it is possible to test the inner and the outer wheel paths simultaneously. The test wheel is contained within a loading frame, placed in the middle of the truck, between the front and rear axles. Two vertical shafts are mounted within the frame, and the test wheel assembly moves up and down on the vertical shafts.

The test wheel assembly (Fig. 3.34) consist of the test wheel, a single spring suspension unit, connections to the vertical shafts and load-cells to measure the vertical and horizontal force. The load-cell has been recently added. Previously, it was assumed that the mass of the test assembly provided a constant vertical force of 200 N, and the drag force was the only force measured. The vertical plane of the test wheel is fixed at 20 degrees to the line of the truck chassis, with a tolerance of +1 and +0.5 degrees.

The test tyre is allowed to rotate freely in its own plan. When the truck moves forward, the test wheel is rotating, but slides in the forward direction because of the angular difference. The standard test speed is 50 km/h in UK, and in other countries, such as in Spain, being allowed to range between 30 to 67 km, or even up to 80 or 90 km/h. On curves with low radii (R < 100 m) and on roundabouts, test is performed at 20 km/h. The test tyre is a pneumatic, rubbed tyre with a smooth tread, inflated to 3,5 kg/cm^2 at ambient temperature. There are wear indicators that indicates when the tyre must be discarded. As previously explained, the sliding speed is very low, approximately 17 km/h, allowing continuous evaluation of the skid resistance over a road network with low rates of tyre consumption. Nevertheless, that low slip speed implies that the microtexture has a great influence on the value. Hence, road agencies that uses SCRIM trucks must complete the friction evaluation with a macrotexture measurement. Table 3.6 summarizes standard test conditions.

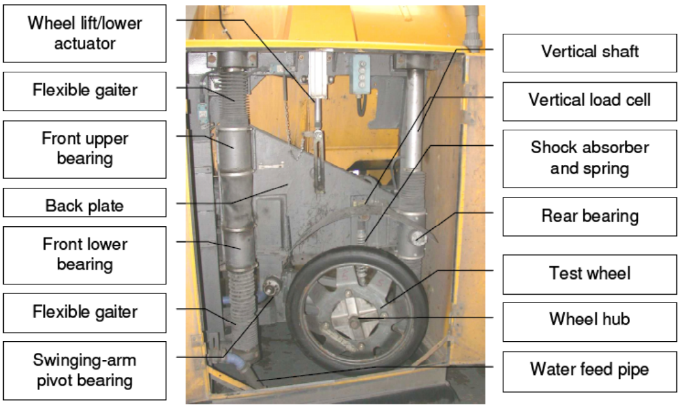


Fig. 3.34. SCRIM test wheel apparatus (Wilson 2006)

Table 3.6. Standard test-conditions for SCRIM (CEN, 2009a)

Characteristic / property	**Values during the test**
Air temperature	> 4 °C
Pavement temperature	> 5 °C and < 50 °C (testing season: April – November)
Pavement status	No pollution
Test wheel	Smoth tyre 76/508 mm inflated at 0,35 MPa
Method	Constant slip ratio from slip angle. Slip angle 20°
Static wheel load	1960 N
Operating speed	Varies from country to country. Generally 50 km/h is used as a reference speed, but other speeds are possible.
Theoretical film water thickness	0,5 mm
Length for the mean value	Generally, minimum 10 m, but other option are possible
Wheel path	Normally nearside wheel path or as required

There are some variants to the SCRIM device. As mentioned previously, there is a double SCRIM device, with two testing wheels, called SUMMS, from Italy (Fig. 3.35a). In Germany, the SCRIM/SRM device is used (Fig. 3.35b, which allows to be used as SCRIM or as SRM, being able to provide sideway force and locked wheel measurements. Both participated in the International Experiment of the PIARC (1995).

Fig. 3.35. Italian SUMMS, b) German SCRIM/SRM, both at the International Experiment (Achutegi Viada 2005).

Nowadays, in the same SCRIM truck a laser texturemeter is installed, called SCRIMTEX, which is able to measure the sideway force coefficient and the texture, resulting in a double function devices. SCRIM is also geo-referred during the test by GPS equipment; to accurate position the machine along the road network.

There is a wide list of countries that deploy the SCRIM machines; including the United Kingdom, Ireland, Belgium, France, Italy, Spain, Germany and Denmark in Europe; Singapore and Malaysia in Asia; and Australia and New Zealand in Oceania (Brittain 2015; Achutegi Viada, 2005, MFOM 2015; Austroad, 2011a, Austroad, 2011b, CEN, 2009a).

3.3.6.2.4.2. Mu-Meter

The Mu-Meter is a three-wheel-trailer, with two yawed wheels, at 7,5 degrees, as shown schematically in Fig. 3.36). Unlike other side-force devices, Mu-meter uses two measuring wheels, which are pushed apart by the frictional forces rather than the one which is mounted in a vehicle and is forced towards it. Each testing tyres is angled at 7,5° to the direction of travel, supposing a slip ratio of about 13 % (Eq. 3.6). Wheels can be smooth or ribbed. It computes the Mu Number (section 3.3.9.2.6). The Mu-Meter is primarily only used for frictional resistance measurement on airports in the USA (Hall *et al.*, 2009). The airfield measurements are usually performed to determine real friction conditions in each moment and to communicate them to pilots. Hence, pilots know the real condition when landing.

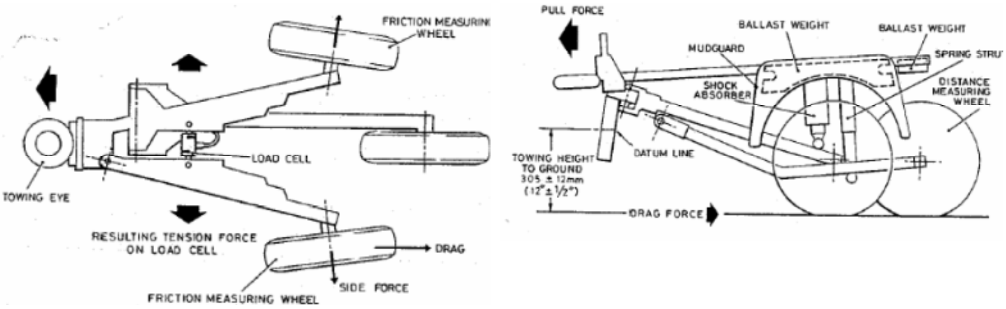


Fig. 3.36. Diagram of the Mu-meter (Wilson, 2006).

A sensor mounted between the two test wheel arms measures the tension force developed, whereas the third wheel provides distance measurement and helps keeping the trailer operating in a straight line. A separate water tank is employed, that can be mounted on a separate trailer (Fig. 3.37) or fitted within the towing vehicle. Its main application is on airfields, where well-defined wheel paths are not generated. Speed test varies from 20 to 80 km/h). It provides the Mu Number.

3.2.6.2.4.3. Odoliograph

The Odoliograph, developed in Belgium, is used in Wallonia and Flanders (Belgium) (Do and Roe, 2008). The testing wheel is mounted within a front-wheel drive car, which follows an independent water spray tanker wet the road. The test tyre is set to the normal straight line when the car is travelling, but it set to an angle of 20° when conducting a test. The static vertical load is 2700 N and the normal test speed is 80 km/h. The water film thickness must be 0,5 mm (CEN, 2009c).

3.3.6.2.4.4. Other devices

The SKM, developed in Germany, was based on SCRIM (section vbn), but modified to be used in Germany. It employs a narrow test tyre, similar to a motorcycle tyre, set an angle to the direction of travel and mounted on one side in the middle of a tanker truck.

Fig. 3.37. Mu-meter device (Hall et al., 2009)

The wheel is lowered on the pavement surface under by a static vertical load defined by the mass of the wheel assembly, which is able to move freely up and down on vertical linear guides. The force acting along the axle of the test wheel is measured and used to calculate the SFC. The device also measure road, air and water temperature, measurements needed in Germany to adjust the reported SFC value.

Another device, the Stradograph, developed in Denmark, was used in the International Experiment of PIARC (1995), using a test blank tyre set at 12°. It was retired in 1998 (Wilson, 2006). Table 3.7 lists and summarizes the main characteristics of the main slidding tyre devices identified.

Table 3.7. Summary of the main characteristics of the slidding tyre devices

Device name	Parameter measured	Measurement method	Slip ratio (%)	Other variants	Tyre	Usual test speeds	PIARC (1995)?[a]	HERMES 2006?[b]
ASTM E-274 Trailer	LFC	Locked wheel	100 %	No	Smooth/ribbed	64	Yes	Yes
ADHERA	LFC	Locked wheel	100 %	0-100% SR	smooth	40,60,90,120	Yes	Yes
SRM	LFC	Locked wheel	100 %	15% or ABS cond	ribbed	40, 60, 80	Yes (Locked and 20% SR)	No
Skiddometer BV-8	LFC	Locked wheel	100 %	14 %	Ribbed	40, 60, 80	Yes (20% SR)	No
Skid Resistance Tester	LFC	Locked wheel	100 %	No	Patterned		Yes	Yes
Griptester	LFC	Fixed slip	15 %	No	Smooth	5 – 100	Yes (14,5 %)	Yes (15%)
Skiddometer BV-11	LFC	Fixed slip	17 %	No	Patterned	70	Yes (17 %)	No
RWS NL Skid Resistance Trailer	LFC	Fixed slip	86 %	No	Smooth	50, 70	Yes (DWW Trailer)	Yes
ROAR DK	LFC	Fixed slip	20 %	No		60, 80	No	Yes
ROAR NL	LFC	Fixed slip	86 %			50, 70		Yes
TRT	LFC	Fixed slip	25 %	1–100% SR, variable	Smooth	40 – 140	No	No
RoadSTAR	LFC	Fixed slip	18 %	Yes	Ribbed	30 - 60	No	No
Norsemeter ROAR	LFC	Variable slip	Variable	Fixed slip	Smooth	50	Yes	Yes(18%)
Komatsu skid tester	LFC	Variable slip	10–30%	No	Smooth		Yes	No
Dagma/Petra Trailer	LFC	Variable slip			Patterned		Yes	No
SCRIM	SFC	Side force (20°)	34 %	No	Smooth	50 (30 – 90)	Yes	Yes
SUMMS	SFC	Side force (20°)	34 %	No	Smooth	50	Yes	No
Mu-meter	SFC	Side force (7,5°)	13 %	No	Smooth/ribbed	20 – 80	Yes	No
Odoliograph	SFC	Side force (20°)	34 %	No	Smooth	80	Yes	Yes
SKM	SFC	Side force (20°)	34 %	No	Smooth	40, 60, 80	No	No
Stradograf	SFC	Side force (12°)	20 %	No	Smooth		Yes	No

a: Participated in International Experiment of the PIARC (1995)?
b: Participated in HERMES project (Descornet 2006)?

3.3.7. Texture measurement

As previously explained, the frictional resistance depends on the hysteresis and adhesion phenomena, which are mainly influenced by the microtexture and the macrotexture of the pavement. Therefore, it is useful to incorporate these measurements to establish the minimum values required in any Pavement Management System. Moreover, their values are necessary of the International Friction Index, IFI, established by the PIARC (1995). At present, methods employed for macrotexture and microtexture measurements are surrogate measurements rather than actual measurements. While macrotexture test methods are based on some form of texture depth measurement, microtexture test methods measure frictional resistance in the microtexture range (Feighan 2006).

3.3.7.1. Macrotexture measurements

Macrotexture test methods are primarily based on texture depth measurements.

As defined previously, the pavement texture is the deviation of a pavement surface from a true planar surface, with a texture wavelength less than 0,5 m (PIARC, 1987; ISO, 2002). On the other hand, the texture profile is the two-dimensional sample of the pavement texture generated if a sensor such as the tip of a needle or a laser spot, continuously touches or shines on the pavement surface while it is moved along a line on the surface. Two coordinates describe the profile of the surface: the distance (abscissa) and the amplitude (the ordinate) (Fig. 3.38) (ISO, 2002).

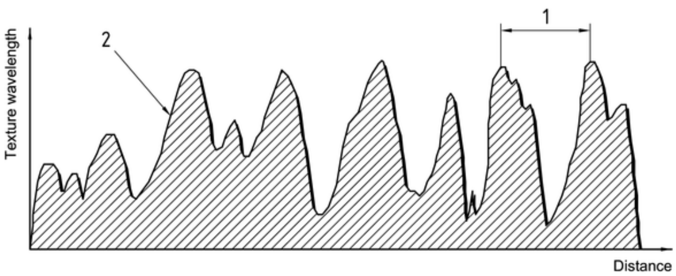


Fig. 3.38. Illustration of basic terms describing the pavement surface texture (ISO, 2002).

3.3.7.1.1. Sand patch test or Volumetric patch method

The most universal and most employed rest worldwide is the sand patch or volumetric patch test. It is so universally extended that it is covered in several national and international specifications, including ASTM E965-15 (ASTM, 2015d) or British specifications (BSI, 2000a).

In the sand patch test, a known volume of standardized sand or small glass spheres is distributed to form a circular patch on the pavement surface with a flat disk. A circle as big as possible must be made with the available material (Fig. 3.39). Once the material is spread out to its limits, the average diameter of this patch is measured. The result, known as the Mean Texture Depth, *MTD*, is obtained dividing the volume of sand (or glass spheres), which is known in advanced, by the surface area covered by the sand, measured on site (Eq. 3.9) (CEN, 2010; ASTM, 2015d).

$$MTD = \frac{4 \cdot V}{\pi \cdot D^2} \qquad [3.9]$$

When V is the sample volume employed and D is the average diameter of the circle.

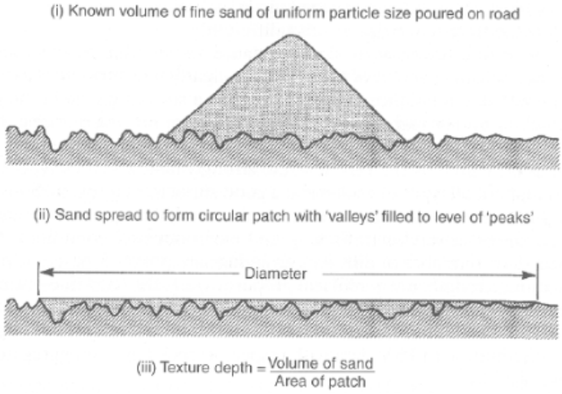


Fig. 3.39. Sand patch test (Wilson 2006)

Initially, sand from Ottawa (Canada) was use and, hence, the *sand patch* denomination comes from it. At present, glasses spheres are preferred as a constant size and shape can be achieved easier. At present, the test is usually referred as *Volumetric patch method.* When spreading the material (sand or glass spheres), the surface voids are completely filled to the tips of the surroundings aggregate particles. The plane from which the texture depth is measured is effectively defined by the bottom surface of the spreading disk passing over the tips of the aggregate particles.

The main advantage of this method is the simplicity. The material and the equipment are relatively low-cost, calculations are fast and results are immediately obtained. Hence, this method is widely used. However, it has some disadvantages. The test requires lane closure and is only representative of a small area. It is slow and time-consuming; there is variability on the results depending on the technician ability to carry it out. Some estimates state that the standard deviation of repeated measurements by the same operator on the same surface can be as low as 1 %, whereas when carried out by different technicians in the same surface, the standard deviation can be around 2 %. Some authors point out that the greatest dispersion is achieved when inexperienced operators perform the test (Feighan 2006). However, the ASTM E965 (ASTM, 2015d) standards indicates that the standard deviation of individual measurements within a nominally homogeneous pavement section can be as high as 27 % of the average texture depth, even with high standard of operator repeatability. Therefore, a big amount of tests are necessary to estimate the average texture depth reliably where the texture is very variable.

Furthermore, despite of its simplicity and low cost equipment, it requires a stationary operator and a road closed to traffic. Hence, it is not conducted for large scale measurements, as in a network level assessment.

3.3.7.1.2. Outflow Meter (OFM)

The Outflow Meter is a outflow volumetric test aims to capture the texture and drainage capabilities of materials that have a negative macrotexture. It is universally recognized that volumetric patch test or lased-based texture measurement devices (presented in subsection 3.4.6.2.1.3) are not able to fully capture the negative-textured surfacing surface offered by some materials such as porous asphalts and some materials used in thin layers. Therefore, this test can measure the drainage capabilities of these materials. Moreover, if using traditional test for texture measurement in these materials, great variability is obtained.

The Outflow Meter consists of a cylinder with a known volume of water, which is placed on the pavement surface. It has a rubber seal between the edge of the cylinder and the pavement surface to ensure that all water can only drain vertically from the cylinder through the pavement surface, and then laterally bellow the pavement surface through interconnected voids (Fig. 3.40). It is measured the time employed for a known volume of water (between two marks in the cylinder) to drain out. The faster the water drains away, the greater negative and effective texture depth the material has. The texture determines the outflow rate. It is developed for pavement surfaces with a high drainage potential and not for impervious surfaces.

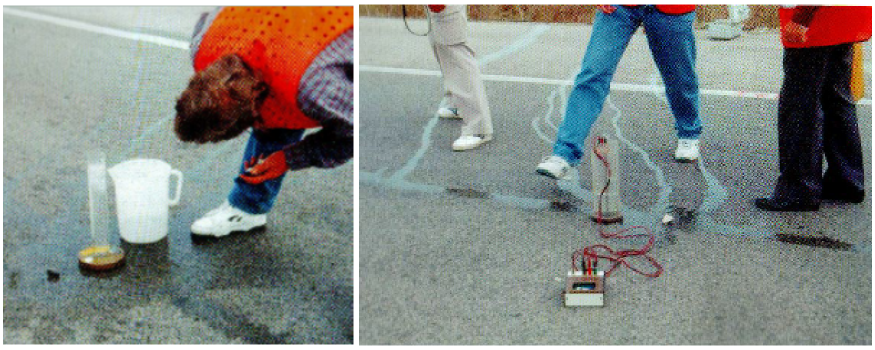


Fig. 3.40. Outflow volumetric test. a) with manual chronometer, b) with automated chronometer Achutegi Viada 2005).

It is a simple method and relatively inexpensive equipment. It gives an idea of the hydroplaning potential in wet weather. On the contrary, it also needs lane closure and only represent a small portion of the pavement.

3.3.7.1.3. Circular Texture Meter (CTM)

The Circular Texture Meter (CTM) is a non-contact lased device, which measures the surface profile along a 286 mm (11 inches) path of the pavement surface at intervals of 0.868 mm. The device rotates at 6 m/min and generates profile traces of the pavement surfaces, registered on a portable computer (Fig. 3.41). Its standards are specified in ASTM E 2157 (ASTM, 2015e). It provides the Mean Profile Depth (MPD) and the Root Mean Square (RMS), described in 3.3.9.3.2.2. However, the main disadvantage is that the Circular Texture Meter also needs to close the lane to traffic.

Fig. 3.41. Circular Texture Meter (CTM). (Hall et al., 2009.)

3.3.7.1.3. Profilometer method. Laser-based texture depth measurement

The main disadvantages of the test explained previously, the sand patch and outflow volumetric tests, is that they are slow and localized. They can be performed on closed road like new or maintained areas, but it is not viable to carry them out on a routine basis in a network. Consequently, the profilometer method is conducted. The profilometer method is an automated method in which the profile of the surface is measured by scanning the road surface by means of a certain type of sensor (PIARC, 2016). They are capable of measuring the texture depth, the wavelength or amplitude at macrotexture scale. Thus, high-speed measurement equipments are obtained for use in a whole road network.

Present designs of profilometers include sensors based on laser, light, sectioning, needle tracer and ultrasonic technologies. The most employed technique is the laser sensors (PIARC, 2016). The projection of light from the laser onto the pavement surface, and measurement of the intensity of the reflected light by an array of light sensors after the reflected light is first focused onto the array by a receiving lens. The sensors are placed at different heights, perpendicular to pavement surface or not, and there is a correspondence between the height of the sensor measuring the highest intensity of reflected light and the height of the pavement surface at the point reflecting the light (Fig. 3.42). The laser produced pulsed light with a frequency of 4 to 64 kHz, depending on the equipment.

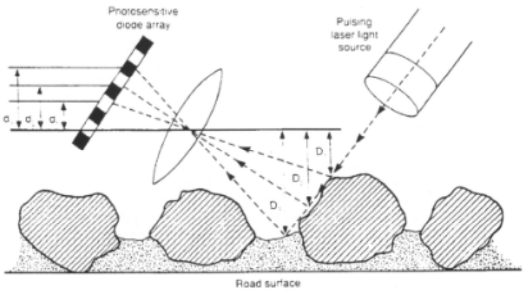


Fig. 3.42. Scheme of a laser-based device (Hall et al., 2009).

Laser profilometers can be vehicle mounted or stationary. Stationary profilometers are slow and need closure of traffic lanes, whereas vehicle mounted devices work at usual highway speeds without disturbing traffic. The variance of the individual displacement measurements over a given length can be calculated, and representative values are recorded for intervals specified in advanced.

One example of high-speed devices for measuring surface texture is the ROad Surface ANalyzer ($ROSAN_V$) developed by the Federal Highway Administration (FHWA). It incorporates a laser sensor mounted on the vehicle from bumper and it operates at speeds up to 113 km/h. Automated measurement devices provide a huge amount of texture data at low cost and improve safety during the tests. Some of the results that are usually reported by laser-based devices are Mean Profile Depth, *MPD*, the Root Mean Square, *RMS*, which allows obtaining the Sensor-Measured Texture Depth, *SMTD*. Further discussion on these values and other are provided in section 3.3.9.3.2.2.

3.3.7.2. Microtexture measurements

As previously explained, due to the microscopic nature of the microtexture it is not possible to measure directly. Normally employed devices are the British Pendulum Tester.

Very sophisticated image analysis is needed to directly measure the microtexture. As some examples of this technique are an image-profiling method developed by Forster (1989) and the automatic image analysis technique developed by Perry (1996). The basis of these techniques lies in the calculation of the spatial texture depth, i.e. the average height of peaks, by means of a microscope and image processing software. Although these methods are promising, they are only employed in research area at present (Austroad, 2011b).

Generally, a skid resistance measurement is employed as a representative index of the microtexture depth. Before the skid resistance measurement, the aggregate sample undergoes a polishing stage in order to be close to the equilibrium stage (section 5.3), and hence, the measurement is more representative of the in situ conditions. Normally, skid resistance is measured by means of the British Pendulum Tester (BPT) or the Dynamic Friction Tester. Polishing methods, as the Polished Stone Value (PSV), are used to achieve that equilibrium stage. Further description is provided in 5.3.

3.3.8. Friction measurement models in relation with the speed

3.3.8.1. Introduction

As discussed, the skid resistance of a pavement surface for the same tyre depends on the texture (macrotexture and microtexture) of the pavement and on speed. The shape of the curve is shown in Fig. 3.5.

In order to characterize the skid resistance reduction as a function of speed, some models have been proposed obtained in different experiments. Although it is feasible to represent by means of linear regressions small variances of slip speed, if the range is larger, it is necessary to introduce other kind of regressions. Second-degree parabolas have represented quite well the descending part, but it could be confusing to think that from a minimum friction value at a specified speed, the friction starts to increase, which is not based on the reality. Therefore, the best fitting models are the exponential ones. On this basis, the PIARC model was developed in the International Experiment of PIARC (1995), which has a precedent in the Pennsylvania State University model, and later, the Rado model was proposed with a better adjustment.

3.3.8.2. Pennsylvania State University model (PSU model)

The Pennsylvania State University model (USA) expresses the relationship between the friction measure and the slip speed, *S*, by means of the exponential equation shown in Eq. 3.10 (Leu and Henry, 1978):

$$F = F_0 \cdot e^{-\frac{PNG}{100} \cdot S} \qquad [3.10]$$

Where F_0 is the ordinate in the origin and *PNG*, *Percent Normalized Gradient*, the slope expressed in percentage defined by Eq. 3.11:

$$PNG = \frac{100}{F} \cdot \frac{dF}{dS} \qquad [3.11]$$

It was demonstrated that *PNG* had a constant value, and hence, Eq. 3.10 can be obtained integrating Eq. 3.11 from S = 0 to S. Moreover, PNG is correlated with the pavement surface macrotexture and F_0 can be predicted by means of the microtexture value. In later versions of the model, 100/PNG was substituted by a constant, S_0, resulting Eq. 3.12:

$$F = F_0 \cdot e^{-\frac{S}{S_0}} \qquad [3.12]$$

In other publications, Eq. 3.12 is expressed with other notation, such as *SN* (Skid Number) and SN_0; or μ (friction coefficient) and μ_0 instead of F and F_0 respectively. After the International PIARC Experiment (PIARC, 1995) S_0 is denominated S_p. The sub index indicates that S_p has been obtained following the procedures of the PIARC experiment. The ordinate in the origin, F_0 can be obtained experimentally because although sideway force and variable slip speed device measures at low slip speeds, no one of them measures at zero speed. Consequently, the Pennsylvania State University model was modified, taking as constant F_{10}, i.e., the friction at 10 km/h slip speed (Eq. 3.13).

$$F = F_{10} \cdot e^{-\frac{10-S}{S_0}} \qquad [3.13]$$

3.3.8.3. PIARC Model

In the International PIARC Experiment (PIARC, 1995), (section 3.3.9.4.1.1), all the result of the friction measurements obtained by the different devices that took part were analyzed, starting from the Pennsylvania State University Model (Eq. 3.14).

After a hard statistical adjustment, (PIARC, 1995), it was concluded that it was better to utilise F60 as a friction constant, which is the friction at a slip speed of 60 km/h, because this is a reference value in the locked-wheel devices. Furthermore, statistical analysis showed that F60 has a lower dispersion than F10. Moreover, the slip speed of 60 km/h was chosen as a representative median value for road vehicles when braking in an emergency. Eq. 3.14 represents the simplified model.

$$F(S) = F60 \cdot e^{\frac{60-S}{S_p}} \qquad [3.14]$$

Where *F(S)* is the friction at the S slip speed, S is the slip speed (calculated as explained in section 3.3.2), *F60* is the friction at 60 km/h after the application of the harmonization procedure described in 3.3.9.4.1.1, and S_p is a constant, with speed units, which depends on the macrotexture. Typically, S_p values range from 1 to 500 km/h (Hall *et al*., 2009). The procedure for determining S_p is exposed in 3.3.9.4. If the IFI parameters for a surface are known, the available friction value can be obtained for all the slip speeds (Fig. 3.43).

3.3.8.4. Rado Model

As discussed in section 3.3.2, when a tyre in free rotation is partially braked, the friction between the pneumatic and the pavement increases until a maximum, the peak friction, approximately 15 % of the slip ratio, and if the tyre continues being braked, it decreases (Fig. 3.4). It was mentioned that the ascending part of the graph depends on the tyre properties and the descending part is dependent on the pavement properties.

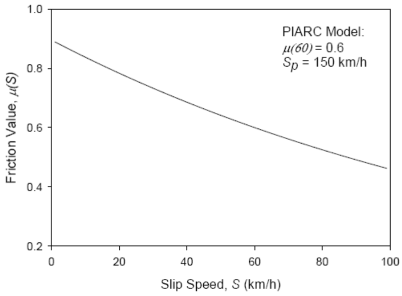


Fig. 3.43. Example of graph of the friction model proposed for the IFI. Source: Author

Z. Rado, a researcher of Pennsylvania State Univ., who participated in the PIARC International Experiment, analysed the experiment result and proposed a good fit for a new friction model, Eq. 3.15 (Rado, 1994).

$$\mu(S) = \mu_{peak} \cdot e^{\frac{-\left(\ln \frac{S}{S_C}\right)^2}{C^2}} \qquad [3.15]$$

where $\mu(S)$ is the friction coefficient at a S slip speed; μ_{peak} is the maximum or peak friction coefficient value measured during a controlled, linearly ramped braking from free rolling to locked wheel at a constant measuring speed; S_c is the slip speed at which the maximum friction occurred, i.e. the critical slip speed; and C^2 is a shape factor related to the texture measurements in a slightly different way that the speed constant S_P of the IFI. Fig. 3.44 shows an example of a Rado model.

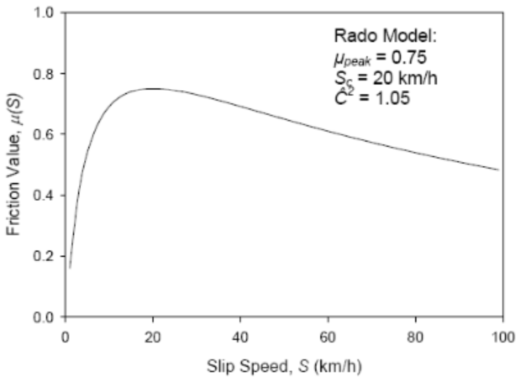


Fig. 3.44. Example of Rado model (Rado, 2000).

As stated previously on subsection 3.3.2.1, the ascending part of the model depends on the properties of the pneumatic/tyre. Once reached the critical slip speed where the maximum friction is achieved (around a slip ratio of 15 %), the descending part starts, which depends on the pavement properties.

In an emergency stopping, if brakes are applied until wheels are blocked, the friction between the tyres and the pavement follows the curve modelled by the Rado model until the slip speed reaches the vehicles speed, and then, if the tyres continue braked, the friction increases followinh the PIARC model until the vehicle stops (Fig. 3.45)

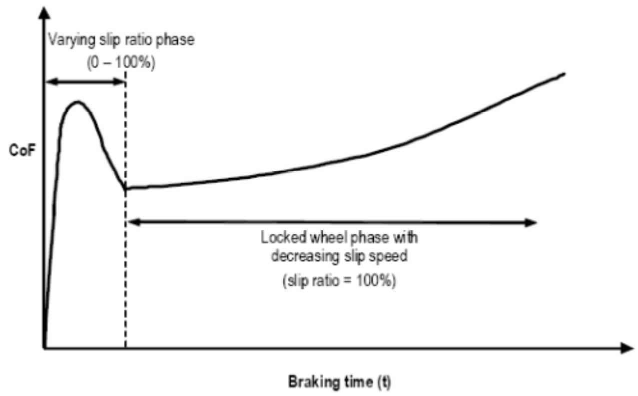


Fig. 3.45. Varying coefficient of friction over braking time. (Rado, 2000)

If brakes are not applied until the wheel is totally locked, and brakes are used again, friction follows again a curve defined by the Rado model but at a slower vehicle speed. This is the main objective of the Antilock Braking Systems (ABS), which reproduce these braking cycles in order to ensure the maximum friction available by only operating in the initial raising part of the curve.

The main difference between the PIARC and Rado models is observed at low slip speeds (Fig. 3.46). The Rado model represents the transient phase when brakes are first applied, up to some slip. Then, the PIARC model is followed as the vehicle slows downs. The PIARC model represents the steady-state value of friction. When braking, the transient phase happens so rapidly that only the PIARC model is needed to simulate the braking process. Nevertheless, if ABS systems are employed, both models must be used to calculate the stopping distance.

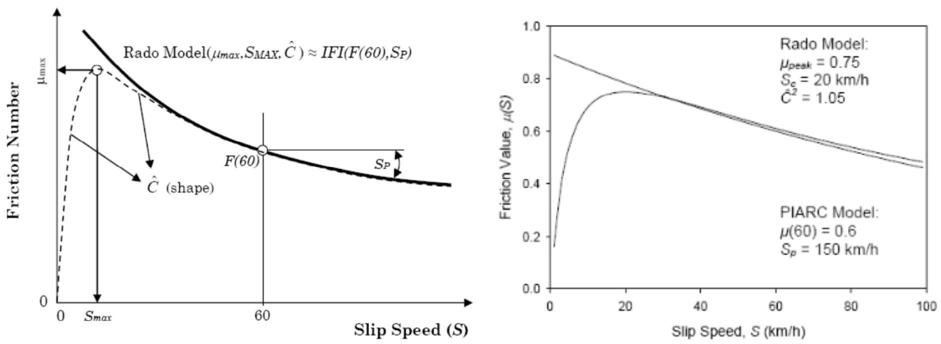


Fig. 3.46. Comparison between Rado and PIARC models, a) ideally, and b) with values (Hall et al., 2009)

While the PIARC model and the IFI value are used to evaluate the skid resistance of the pavement and to better allocate available funds in sections with unacceptable levels, the Rado model is used to predict the braking performance (Hall *et al*., 2009). The Rado model is needed because the majority of cars include ABS brake systems. Variable slip measurement devices are required to characterize Rado model, being to provide the three parameters that describe the process, μ_{max}, S_{max}, C. With hundreds of test values at known slip speeds, a friction number curve can be fitted to available data.

As a conclusion, the PIARC models is best for use with fixed-slip devices and varying measuring speed and the Rado model must be used with fixed measuring speed and varying slip speed (Andresen and Wambold 1999; Achútegi Viada 2005; Hall *et al*., 2009). When the two models are combined, three-dimensional models can be obtained, which are described in the following subsection.

3.3.8.5. Three-dimensional modelling of tyre-surface friction

If vehicle speed, slip speed and friction are combined, it is possible to create three-dimensional models (Fig. 3.5). Usually, vehicle speed and slip speed have been considered as separately as two independent variables by friction models. Thus, both vehicle and slip speed could be combined in a unique three-dimensional model. Bachmann (1998) performed many test with variable-slip devices and obtained data to develop these 3-D models, obtaining a surface plot of friction (Fig. 3.47, Fig. 3.48).

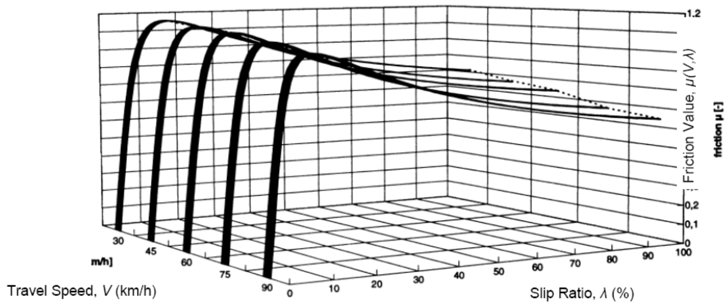


Fig. 3.47. Series of variable-slip measurements with an automotive tyre at different measuring speed on dry concrete pavements. (Bachmann, 1998).

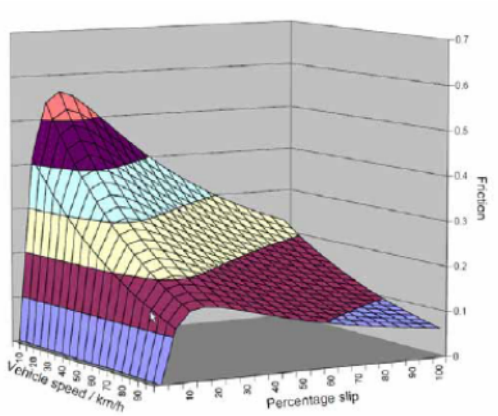


Fig. 3.48. Variation of friction as a function of vehicle speed and slip ratio (Do and Roe, 2008).

As observed in Fig. 3.47 and Fig. 3.48, it is more useful if slip ratio, *SR*, than slip speed, as the last one could never exceed the travel or vehicle speed (Hall *et al.*, 2009). Despite the fact that braking slip friction could be presented in a three-dimensional way, no documented universal three-dimensional mathematical models has been developed without the combination of the PIARC and Rado models (Hall *et al.*, 2009).

3.3.8.6. Other models that incorporate other parameters

PSU, IRI and Rado model take into account speed but not other parameters like the applied pressure and rubber temperature. Consequently, other models that consider these inputs have been developed.

The Wriggers model defines the friction evolution as a function of the normal pressure, the maximum friction coefficient and the corresponding speed (Wriggers, 2006). An alternative model, only dependent of speed and pressure is also available (Nackenhorst, 2004).

3.3.8.7. LuGre Model

Despite the efforts for the harmonisation of an international standard for friction, there are criticisms on the inconsistency of the harmonisation method. There are examples where the predicted friction (IFI) at 60 km/h slip speed from the same device from friction measurements at two different speeds on the same pavement showed considerable variations. Furthermore, the major assumption of linearity between the friction index at 60 km/h slip speed (F60) and the measured value at a different slip speed adjusted to 60 km/h slip speed (FR60) does not seem to be valid in some situation, especially in the calibration at 96 km/h (Fig. 3.49).

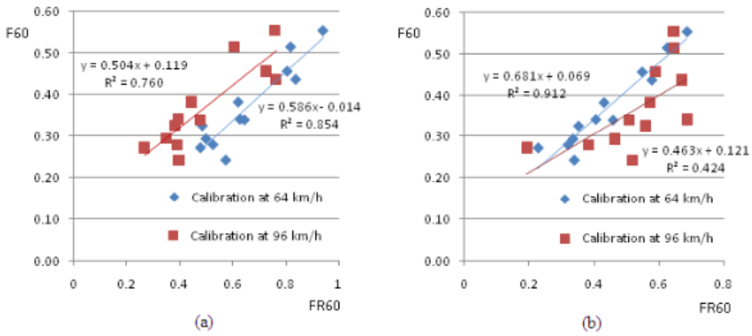


Fig. 3.49. Quality of IFI calibration for two different measuring equipments, a)Runway Friction Tester and b) Locked wheel skid tester, at two different slip speeds (Rajapakshe, 2011).

Other deficiencies of the IFI model were pointed out. The IFI method fails to compensate for the disparity in the measurements due to differences in properties of the measuring mechanisms between two devices of similar characteristics, such as tyre pressures. Moreover, an extra set of spot texture measurements on the surface is required by the IFI procedure, which can be impractical in places like busy runways. As a consequence, the Lund-Grenoble (LuGre) model has been proposed as a better alternative to the model described in the ASTM E1960 (ASTM, 2015f). The LuGre friction model (Canudas-de-Wit *et al.*, 1995), which derives its name from the “Lund Institute of Technology” of Sweden and the “Laboratoire d’Automatiquede Grenoble” in France, where the initial work to develop the model was conducted. The LuGre model was originally developed to model general sliding friction for control purposes and it was later applied to longitudinal tyre dynamics (Canudas-de-Wit *et al.*, 2003).

Rajapakshe (2011) developed friction measurement predictions by different equipment by means of non-linear optimization of LuGre parameters. Specifically, Rajapakshe (2011) developed a practical method to calibrate the LuGre tyre model to capture the effect of the amount of water at the tyre-pavement interface on friction.

3.3.9. Friction and texture indices

3.3.9.1. Index classes

The skid resistance determined by a device is usually expressed by means of a value. This figure, normally corrected to standard conditions, gives an index or indicator of the skid resistance. Although there may be so many indices as measurement devices, it is usually attempted to establish a common index that evaluates the skid resistance in a unique way. According to the pavement characteristic that evaluate, there are three index groups:

- Friction indices
- Texture indices
- Friction and texture double indices

3.3.9.2. Friction indices

3.3.9.2.1. British Pendulum Number

The height achieved in the British Pendulum Tester, described in section 3.3.6.1.1, is expressed as British Pendulum Number, BPN. The pendulum is calibrated in the part of the readings, and hence, the value is directly obtained. Moreover, due to the low test speed, around 10 km/h, it is said to be an indicator of the microtexture. It is used worldwide.

3.3.9.2.2. Dynamic Friction Tester Number

The Dynamic Friction Tester provides friction measures as a function of speed, from 0 to 90 km/h. Results are usually collected at 20, 40, 60 and 80 km/h and it can provide a speed-friction plot (Fig. 3.20). The value is expressed as DFT(S), where S is the test speed. At present, the DFT at 20 km/h, DFT(20), is replacing the BPN around the world as an indicator for friction at low slip speeds. It can be correlated with IFI.

Tests carried out at the NASA Wallops Friction Workshops indicated that DFT(20) is more reproducible than the BPN (Henry 2000)

3.3.9.2.3. Friction Number or Skid Number

The Friction Number, *FN*, previously called Skid Number, *SN*, has been used for a long time, mainly in the USA. It is obtained by means of locked-wheel testing devices, following the ASTM E 274 standard (ASTM, 2006) It is calculated following Eq. 3.16:

$$FN(V) = 100 \cdot \mu = 100 \cdot \frac{F}{W} \qquad [3.16]$$

Where V is the speed of the test tyre (usually in mi/h), μ is the coefficient of friction, F is the tractive horizontal force applied to the tyre and W is the vertical load applied to the tyre. Both forces must be expressed in the same units. It is a similar expression to Eq. 3.1, expressed in percentage. It represents the average coefficient of friction along the test stretch. The figures range from 0 to 100. A value of 0 represents no friction available and 100 means the complete friction.

Generally, *FN* values are accompanied by a figure representing the speed at which the test was carried out and by a letter indicating the type of tyre that was employed in the test. For example, *FN50R* = 38 means that a friction value of 38 have been obtained when measured at a test speed of 50 mi/hour (81 km/h) with a rubber (*R*) tyre. Similarly, *FN40S* = 27 means that a friction value of 27 was obtained when measuring at a test speed of 40 mi/hour (64 km/h) with a smooth (*S*) tyre.

3.3.9.2.5. Other indices obtained by Longitudinal Force Coefficient measuring devices

Fixed-slip devices measure both the resistive drag force and the wheel load applied to the pavement, and, hence, the coefficient of friction is provided, μ. Usually, this friction is reported as *FN*. They collect continuous friction data. However, these equipments provide values at a specific slip rate, which is not always the critical slip value. For example, the GripNumber, obtained from the GripTester is calculated in a similar way to the Friction Number for locked wheel devices, as a ratio of the horizontal drag forces to vertical load force (Eq. 3.1).

On the contrary, the variable slip devices are able to report a big amount of data, including longitudinal slip friction number, Peak slip friction, critical slip ratio (Fig. 3.6) and Rado Shape factor (Fig. 3.44). When used as locked-wheel equipment, they provide *FN* values, too.

3.3.9.2.6. Mu Number

The Mu-Meter provides the Mu Number as side-force Coefficient, SFC

3.3.9.2.7. Sideway-force coefficient (SFC)

The Side-force Coefficient, SFC, obtained from the SCRIM is the ratio of the sideway force to vertical reaction between the tyre and the pavement surface, with a value from 0 to 1. A SCRIM reading, SR, is the output for each subsection of the tested road, usually 5, 10 or 20 m. Usually, SR values are calculated at 10 m intervals on rural roads and at 5 m intervals on urban roads. The SR value is the average SFC value over the entire subsection length, expressed as an integer value, multiplied by 100. These SR values come directly from the SCRIM machine, they are raw values and they must be corrected for speed.

In the United Kingdom, readings for each 10 m sub-section collected within the speed range 25 to 85 km/h is corrected to a speed of 50 km/h using Eq. 3.17 (Highway Agency, 2015):

$$SR(50) = SR(s) \cdot \left(-0{,}0152 \cdot s^2 + 4{,}77 \cdot s + 799\right)/1000 \qquad [3.17]$$

Where SR(50) is the value of SR(s) corrected to 50 km/h, SR(s) is the Sideway Force Coefficient, measured at speed *s*, multiplied by 100. This term is defined in British Standard BS7941-1 (BSI, 2006).

When the truck-mounted style of SCRIMs were introduced, leaving the SCRIM motorbike, an “index of SFC” factor was introduced in order to correlate existing historical records with present measures, because consistency was aimed to be maintained. Currently, the “index of SFC” value in use in the UK is 0,78, and is applicable to all UK SCRIMs in use (Highways Agencies 2015). Consequently, SCRIM Coefficient, *SC*, is calculated for each 10 m sub-section for which a valid SR(s) value is available with Eq. 3.18:

$$SC = \left(SR(50)/\ 100\right) \cdot 0{,}78 \qquad [3.18]$$

AS seen, the SC is an SFC value corrected for speed and machine variability. It is expressed as a decimal fraction, with two decimal places. The index of SFC is just a factor applied to relate the values given by SCRIM machines to historic values. Thus, this relation that relies present SC values and past SR values must be taken into account when interpreting SFC, SR, SC values and standards about SCRIM data from the United Kingdom. (Feighan 2006; Achútegi Viada 2005) In New Zealand, this index factor is also applied (Austroad, 2011b).

In Spain, in precedent standards, SCRIM Coefficients, SC values had to be expressed as a decimal fraction, from 0 to 1 (Ministerio de Obras Públicas y Urbanismo, 1989b). Since 2001, SC is expressed from 0 to 100, i.e. multiplied by 100 (Ministerio de Fomento, 2001). SCRIM Coefficient is adjusted for temperature and speed, but no "index of SFC" is applied.

3.3.9.3. Texture indices

As previously stated, friction is dependent on microtexture and macrotexture, which were defined by the PIARC (1987) according to the wavelength and amplitude and are independent of any particular test method. Hence, it is to know test equipment measuring range when conducting a measure. Present test methods are surrogate measurement rather that actual measurements.

3.3.9.3.1. Microtexture measurement

As exposed, direct measurement of microtexture is not feasible at present because of the microscopic nature. Test methods employed to measure microtexture measure frictional resistance in the microtexture range, as the British Pendulum Tester. Furthermore, if polishing susceptibility of a particular stone, under scrubbing action of vehicle tyres is wanted to be measured, tests like PSV (Section 5.3) are conducted.

3.3.9.3.2. Macrotexture measurement

Traditionally, texture depth measurements have been conducted as a measure of pavement macrotexture. Texture depth may be defined as the distance between the textured road surface and a plane produced by the peak of the three highest particles in that road surface area of the size as the tyre-pavement contact area (Fig.3.TI-1). As seen, the texture depth is the distance between an arbitrary point of the plane down to the surface perpendicular to the plane.

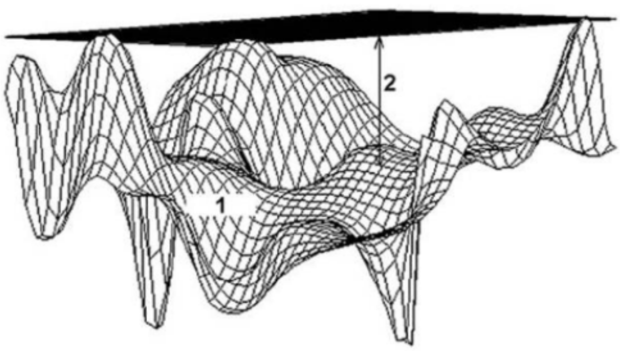


Fig. 3.50.Illustration of the surface and texture depth (1: surface, 2: texture depth) (PIARC, 2016)

3.3.9.3.2.1. Mean Texture Depth (MTD)

For many years, road surface texture has been evaluated by the sand or volumetric patch test. As the material used in the test, sand or glass spheres, is spread with a spreading tool until the sand (or spheres) is levelled with the tops of the aggregates, the average texture in the area of filled surface voids can be obtained. It is calculated using Eq. 3.9, and the value is called Mean Texture Depth, MTD. The index is expressed in millimetres (CEN, 2010; ASTM 2015d).

As explained previously, this test method needs traffic interruptions, depends on the worker's experience and it is slow and time consuming. Therefore, laser based tools were developed.

3.3.9.3.2.2. Indices obtained by laser-based devices: Mean Profile Depth (MPD) and others

Profile depth can be defined as the difference between the profile and a horizontal line through the highest peak (the peak level) within a certain distance along the surface. The distance along the surface is of the same order of length as the length of the tyre/road interface (PIARC, 2016)

When a laser-based system is used to measure the road surface profile, this is divided into segments of length of 100 mm, called baselines (Fig. 3.51). Then, each baseline is divided into two equal parts of 50 mm and the highest peak in each subpart is determined. The two highest points are arithmetically averaged to calculate the average peak profile level. The average profile of the entire segment of 100 mm is the calculated and substracted from the average peak profile level and the Mean Segment Depth (MSD) is obtained. The Mean Profile Depth (MPD) is the average of the values of the Mean Segment Depth of the tested section. MPD is also expressed in millimetres (ISO 2004). This method is a two dimensional simulation of the volumetric sand path method, which is three dimensional (Wilson 2006; Hall *et al.*, 2009).

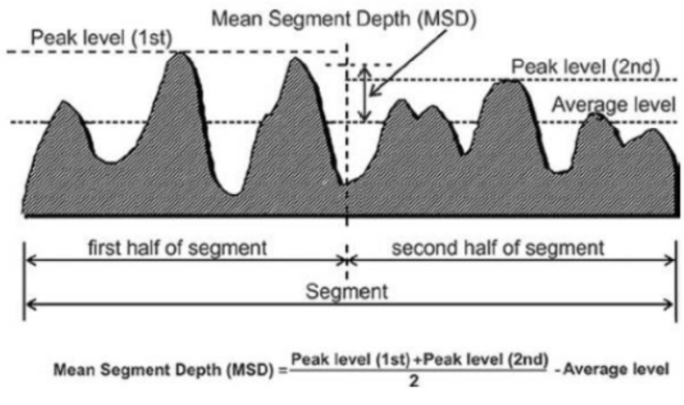


Fig. 3.51. Definition of Mean Segment Depth (MSD) (PIARC, 2016).

A similar index is the Sensor-Measured Texture Depth (SMTD), used for macrotexture measures. The difference between SMTD and MPD is the way that the height of the texture is estimated. SMTD measurement is essentially a root mean square measure of the texture both above and below the mean level, while MPD measures the height of the highest peaks above the mean level (Fig. 3.51). The Root Mean Square (RMS) is the root mean square deviation of the profile height within the evaluation length. The RMS offers a measure of how much the data (actual profile) deviates from a best-fit (modelled profile) of the data (McGGhee and Flintsch, 2003).

Finally, Estimated Texture Depth (ETD), developed by PIARC (1995) is an estimate of the MTD that would be obtained by the sand patch test; by means of a measurement of MPD, using a transformation equation (Eq. 3.19) (ISO, 2004):

$$ETD = 0{,}2 + 0{,}8 \cdot MPD \qquad [3.19]$$

ETD has been also related to MPD measurement derived from CTM (ASTM E 2157) (ASTM, 2015e):

$$ETD = 0{,}{,}947 \cdot MPD + 0{,}069 \qquad [3.20]$$

Where MPD is obtained from CTM and both ETD and MPD are expressed in mm.

3.3.9.3.2.3. Outflow time (OFT)

The index obtained from the Outflow Meter is called Outflow Time (OFT) and is the time in milliseconds for outflow of specified volume of water. Low OFTs implies a rough macrotexture, while high OFTs indicates smooth macrotexture.

ETD can also be calculated from OFT by Eq. 3.21 (PIARC, 1995, ASTM, 2015g):

$$ETD = \frac{3{,}114}{OFT} + 0{,}656 \qquad [3.21]$$

Where OFT is the Outflow time, as described in section vbg, expressed in milliseconds and ETD in mm.

3.3.9.3.3. Correlation between texture indices

The International Experiment of the PIARC (1995) compared 14 texture methods and concluded that very good linear correlations exist between MPD and the Sand Patch Method, resulting in the main finding , Eq. 3.19. Other relationships can be obtained in PIARC (1995).

3.3.9.4. Friction and texture double indices

Friction and texture double indices need two values to be determined, in this case, a friction value and a texture value. The main index in this group is the International Friction Index, IFI, developed by the World Road Association (PIARC) as a result of the International PIARC experiment carried out in 1992 (PIARC, 1995). Other double indices have arisen, such as the European Friction Index, (EFI), developed under the project HERMES (Harmonisation of European Routine and Research Measuring Equipment.

3.3.9.4.1. International Friction Index

As it has been exposed, there is a wide range of friction or texture indices. Road agencies employ generally one for friction and another one for texture according to the device they usually deploy. Data are expressed in a different scale, according to the test performed, which has different type of tyre (rubber or smooth) and was carried out at different slip ratios, at different temperatures, etc. As a result, every road agency develops its own skid resistance management based on their data and experience recorded. However, as other administration could use another devices, the interchange of knowledge was laborious and sometimes, impossible. In order to compare the different measuring methods employed through the world and to develop formulae in order to convert results obtained by different devices to a common scale, called the International Friction Index, the World Road Association (PIARC) carried an experiment, commonly referred to as International PIARC Experiment (PIARC, 1995).

3.3.9.4.1.1. International PIARC Experiment

Previous to the International PIARC Experiment, many attempts to correlate data from different pavement devices were performed. Some of them provided good correlations, when the test was carried out in similar surfaces, but, a general correlation was not achieved. From these experiences, it was known that for correlating data from different devices, surface texture had to be included in the model (PIARC, 1991, Shah and Henry 1978). The main goal of the experiment was to harmonize the large variety of pavement friction measuring devices employed through the world.

To this end a large amount of data were needed. The experiment was carried out in September and October of 1992. 54 sites in Europe were selected: 28 in Belgium (2 at an airfield, 4 at a racetrack and 22 on public roads) and 26 in Spain (8 at airfields and 18 on public roads). 51 different measurement devices from 16 countries took part.

Each friction tester measured at three different speeds: 30, 60 and 90 km/h, making two runs at each speed. If the equipment works at a different speed, this one was substituted for the speed nearest to that standard one. All texture measurements were performed on dry surfaces before water was applied to the pavement. A microtexture test by means of the British Pendulum Tester was carried out before and after the friction evaluation and it made possible to verify that no statistically significant changes occurred during the experiment. As observed, the four basic types of friction measuring devices were employed: locked-wheel, fixed slip, variable slip and sideway force. As some equipment is able to vary the slip, it is possible to operate around the friction peak. Among the texture measuring devices, groups described in section 3.3.7 were deployed: volumetric tests, outflow test and profilometers or laser-based devices. No microtexture measuring devices were included, but low speed friction tests, like the British Pendulum Tester, are considered surrogates for microtexture.

The main achievement of the experiment was a well defined universal friction scale by means of the International Friction Index, IFI, which needs two values, one derived from friction and the other one from texture. In next section, the procedure to calculate the IFI is provided.

3.3.9.4.1.2. Procedure to obtain the IFI values

As no device can be stated as universal one, no true value can be determined for friction. Consequently, a composite of the measurements made by the systems measured over a wide range of sliding speeds was employed to develop a friction-speed curve for each site (PIARC, 1995). These curves are called the "*Golden curves*" and they represent the true values for the experiment. As commented before, the Golden Curves follow the PIARC model (section 3.3.8.3) and are composed of two numbers, GF60 and GS, related by Eq. 3.22:

$$GF(S) = GF60 \cdot e^{\left(\frac{60-S}{GS}\right)} \qquad [3.22]$$

Where S is the slip speed (in km/h), $GF(S)$ is the “true” fiction-speed relationship for a pavement, *GF60* is the Golden Value Friction Number and *GS* is the Golden Value Speed Number in km/h.

It was observed that the Golden Value Speed Number, *GS*, was related to the macrotexture and could be easily predicted by the texture measurements carried out in the experiment. It was only necessary a linear regression to adjust the texture measurement data to a predicted value, S_p, of the Golden Value Speed Number, by means of Eq. 3.23.

$$S_p = a + b \cdot Tx \qquad [3.23]$$

Where *T*x is the texture measurement value obtained by a determined devices and a and b are regression coefficient, available for each device used in the experiment.

The slip speed, *S*, of each friction measurement device must be determined, which depends on the tester speed, V, and can be calculated according to the group of the friction devices it belongs (Eq. 3.24 - Eq. 3.26):

- For locked-wheel testers:

$$S = V \qquad [3.24]$$

- For fixed slip testers:

$$S = V \cdot SR \qquad [3.25]$$

- For sideway force testers:

$$S = V \cdot \sin(\alpha) \qquad [3.26]$$

Where *SR* is the slip ratio, as defined in Eq. 3.6, and α is the yaw angle, described in Fig. 3.9.

Next step is to determine the Golden Value Friction Number, *GF60*, at a common slip speed of 60 km/h, by means of a measured value at a specific slip speed. To this end, the exponential PIARC friction model is employed (Eq. 3.27):

$$FRS = FR60 \cdot e^{\left(\frac{60-S}{S_p}\right)} \qquad [3.27]$$

Where *FR60* is the adjusted value of friction for the system, at a slip speed of 60 km/h, *FRS* is the friction value obtained at specific *S* slip speed. As *FR60* is the unknown variable, it can be determined by Eq. 3.28:

$$FR60 = FRS \cdot e^{\left(\frac{S-60}{S_p}\right)} \qquad [3.28]$$

Finally, the last step is to calibrate the system by regression of the adjusted measurement, *FR60* with the Golden Value Friction Number, *F60*, obtained by means of Eq. 3.29.

$$F60 = A + B \cdot FR60 + C \cdot Tx \qquad [3.29]$$

Where *F60* is the prediction of the Golden Value Friction Number, *GF*; and *A*, *B* and *C* are regression constants for a particular equipment. *C* is only employed when using ribbed tyre. If a smooth tyre is employed, *C* is equal to zero. Their values are dependent on the employed devices. The correlations for the devices deployed in the experiment are provided in PIARC (1995). If combined the results described above, *F60* can be directly expressed by a unique equation with the terms of the friction and texture measurement of the device employed (Eq. 3.30).

$$F60 = A + B \cdot FRS \cdot e^{\left(\frac{-(60-S)}{a+b\cdot Tx}\right)} + C \cdot Tx \qquad [3.30]$$

Both values, *F60* and S_p, are the IFI, the International Friction Index, the prediction of the Golden International Friction Index, GIFI. The *F*(*S*) curve obtained in the process, represented in Fig. 3.52, is a very close approximation of the pavement reference curve or Golden Curve *GF*(60). The Golden Curve was developed for each test stretch in the PIARC experiment by all the average values of the *F*(*S*) curves obtained by means of a pair of values of *FRS* and *Tx*, determined by a friction test device (*FRS*) and a texture measurement device (*Tx*) (Achútegi Viada, 2005). The average absolute difference is a direct measure of the error in predicting the *GF60* using the *F60* from particular texture and friction pair of devices (PIARC, 1995).

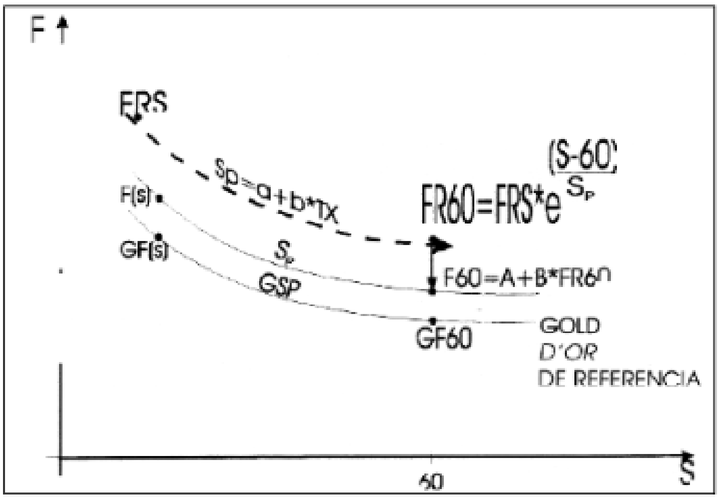


Fig. 3.52. PIARC model (PIARC, 1995)

As a conclusion of the experiment, it was determined that the IFI can be used with a "*acceptable small error, typically in the range of the ± 0,03 friction number*" (PIARC, 1995), when using a measure of the Mean profile depth (MPD) and whatever friction measuring device that took part in the experiment.

Both figures of IFI are also proposed for all kind of applications, like pavement management systems, airfield condition evaluation, accident investigations, etc. Moreover, researches that express their results in IFI values could be understood all over the world.

For highway administrations, decision makers could state interventions levels for *F60* and S_p, designated *F60** and S_p*, in order to establish the adequate strategy for maintenance and rehabilitation works.

Comparing data from their devices and transformed to IFI values (*F60* and S_p), with the intervention levels they have proposed, *F60** and S_p * figures similar to Fig. 3.18 could be made the factor that needs improving is observed. Those intervention levels are determined by road agencies and generally differ from one road type to another.

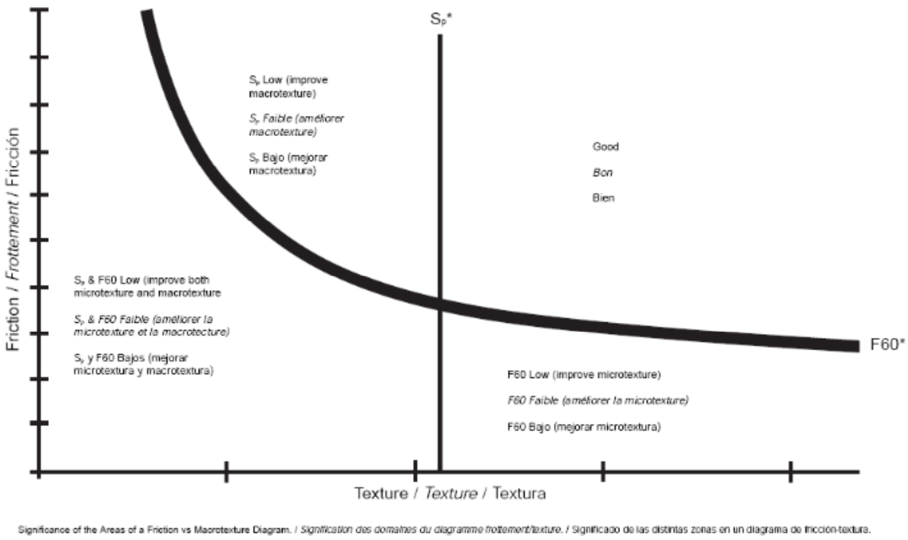


Fig. 3.53. Significance of the areas of a Friction vs. Macrotexture diagram (PIARC, 1995).

3.3.9.4.2. HERMES project

Although the IFI was proposed as a friction index to be used worldwide, European countries have shown that they are not fully confident in the precision of IRI result (Roe and Sinhal, 2005). Consequently, it led to propose a development of the European Friction Index (EFI), also based on a similar principle to IFI, but focused on the friction measurement devices that are usually in Europe (Descornet *et al.*, 2006). As a result, the Harmonisation of European Routine and Research Measuring Equipment (HERMES) arose in 2001-2002.

The HERMES project was performed by means of the Forum of European National Highway Research Laboratories (FEHRL) and included 15 skid resistance measurement devices, 7 texture measurement testers and 61 test in 5 countries (Belgium, the Netherlands, Spain, Great Britain and France). The sites were selected as a representation of the materials and texture that could be found normally in Europe. The Mean Profile Depth (MPD) was established as texture depth measurement. The HERMES project found that (Descornet *et al.*, 2006):

- A power law for correlating the speed constant (Sp) and MPD fitted better than the linear correlation suggested in the PIARC IFI model.
- A procedure was prepared, for a common and stable scale of friction for the devices that took part in the calibration tests, in real conditions.
- The reproducibility of EFI was evaluated as poor, making impossible to produce a compulsory application.
- The project has established alternative solutions for future specifications.

3.3.9.5. Remarks about friction and texture indices

As stated previously, one of the goals of the International PIARC Experiment was to determinate an international friction scale, where all the devices could be referred to PIARC (1995). There is no doubt that the International Friction Index has met this objective, but there is still a long way for IFI to became an universal index in which all the researches or pavement regulations about skid resistance are expressed in developed countries (Achútegi Viada 2005, Feighan 2006). There were important forward steps in that direction, but the process is long and difficult.

Complementary researches have been carried out to complete IFI experiment, as the one in New Zealand (Cenek and Jamieson, 2000). They attempted to apply the ICI concepts to coarse textured surfaces. The Speed Number was to be derived from Sand Patch test measurements and the Friction Number from Griptester measurements. The New Zealand experiment derived PIARC transformation constants.

Undoubtedly, as more research is performed worldwide, transformation equations and methodology will be improved to reduce variability on IFI calculation.

3.3.10. Skid resistance policy

Previous sections have underlined the importance of adequate microtexture and macrotexture in the road network to provide acceptable levels of skid resistance to road users. Laboratory and field tests characterize the performance of the pavement surface over a whole range of vehicle speeds. Although different tests are deployed over the world, the same key aim underlies in all of them: ensure adequate micro and macrotexture. A highway administration can use the information in different ways. Essentially, these are the main areas:

- Control the frictional resistance in the road network by means of microtexture and macrotexture specifications
- Compare frictional resistance values with accidents and identify high-risk accident locations
- Manage in situ frictional resistance through systematic network surveys.

Consequently, developing a performance model for friction or skid resistance evolution as a function of different factors would be of great interest.

3.4. Longitudinal profile and roughness

3.4.1. Introduction

Pavement roughness, or roughness, is probably the attribute of most interest to the travelling public, because it is the pavement characteristic that drivers and users only perceive, mainly by means of a better or worse riding experience. (Achutegi Viada 2005, COTO 2007; Smith and Ram, 2016, Robbins and Tran, 2016). Moreover, studies have shown roughness is the primary criteria for users to judge pavement performance, and hence, the condition of a road network (Budras, 2001).

Roughness describes the irregularities in the pavement surface, originated from variation in surface elevation, which induces vibrations in the vehicle, which are transmitted to the passengers, impacting negatively in their comfort. If roughness is important, it could also affect the safety since the contact conditions between tyre and pavement change. At the same time, roughness may have influence the vehicle operating costs due to its incidence on the rolling resistance, increasing fuel consumption and vehicle maintenance costs (Smith *et al.*, 1997; FHWA, 2006; Chatti and Zaabar 2012; Van Dam *et al*., 2015).

Another issue, which is usually not noted by users, is that the service life of the pavements is reduced. Increasing dynamic loads produced by the heavy vehicle action on roads with a deficient roughness damage pavement structure, and therefore, dynamic forces accelerate pavement deterioration, especially when the amplitude of the wavelengths are high (FHWA, 1991; Bennett *et al.*, 2007; Smith and Ram, 2016). Hence, maintenance costs increase (Swanlund, 2000).

Consequently, roughness is one of the most important aspects to monitor from a highway due to direct relation to users' experience evaluation (COTO, 2007). It also serves as a collective measure of several aspects of road state, including rutting, cracking, potholes, local failures and undulations.

Apart from the term roughness, it is also called riding quality or smoothness, and all the names can be used interchangeably (COTO, 2007; Smith and Ram, 2016). Road agencies also use roughness evaluation to evaluate initial pavement smoothness during construction and as a key performance indicator in the maintenance and rehabilitation process. (Smith and Ram, 2016; Robbins and Tran, 2016).

3.4.2. Definition

Pavement roughness is defined as "*the deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, dynamic loads and pavement dr*ainage" (ASTM, 2012).

As exposed in section 3.3. Skid resistance, the deviation of the pavement surface from a true planar surface appear at different levels of scale, which were classified by the Permanent International Association of Road Congresses (PIARC) in the World Road Congress in 1987 in Brussels, defining 4 levels according to the wavelength, and the peak-to-peak amplitude, *A*, as shown in Table 3.1 (PIARC, 1987). It has been deeply commented that deviations with a wavelength below 50 mm are related to skid resistance and friction.

Irregularities with a wavelength between 50 and 500 mm and vertical amplitudes ranging between 0,1 mm to 50 mm are called megatexture, and have a similar size as the pavement-tyre interface. This level of texture generally results from poor construction practices, local settlements, or surface deterioration. Megatexture can cause vibration in tyre walls, resulting in in-vehicle noise and some external noise as well. It also adversely affects pavement ride quality and can produce premature wear of the vehicle suspension (i.e., tyres, shock absorbers and struts). It is rarely measured or considered directly; it is defined primarily to provide a continuum between macrotexture and unevenness (roughness).

Roughness or unevenness are the deviations with a wavelength between 0,5 and 50 m. Wavelengths in this range impact vehicle dynamics, ride quality, and surface drainage. Unevenness is generally attributed to the environment (i.e., temperature and moisture effects) and/or poor construction practices and deformations in any pavement layer that have been induced by vehicle loads. Unevenness does not significantly affect tire-pavement noise. As seen, each type of deviation affects different pavement properties, resumed in Fig. 3.15.

Whereas microtexture and macrotexture are necessary for providing frictional resistance, the megatexture and roughness must be avoided as they have a negative impact on riding quality, safety, vehicle operating costs and pavement life due to dynamic loads.

Wavelengths over 50 m or 100 m are due to the longitudinal profile configuration of the road and are considered neither in frictional properties nor in roughness evaluation.

The analysis of irregularities in pavement surface is a three-dimensional problem, and hence, it is usual to consider separately the longitudinal and transverse profiles (Fig. 3.54). The transverse profile is measured in a direction perpendicular to the direction of vehicle movement, while the longitudinal profile is measured in the direction of movement. Roughness is primarily concerned with the longitudinal profile while transverse profiles are mainly used to assess rutting.

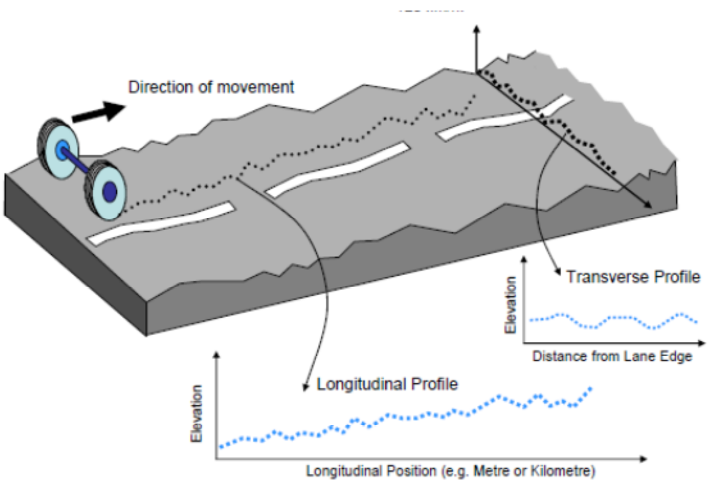


Fig. 3.54. Road profile measurement (COTO, 2007)

Some concepts related to the measurement of a road profile must be included.

- Along any line on the road, there exists a “true profile” (Fig. 3.55a). The true profile is approximated by the measured profile, which is a profile measured at a predetermined sampling interval (Fig. 3.55b). This sampling interval is the spacing between two consecutive points along the line of measurement on the road. At present, modern profiling equipment can measure elevations at intervals less than 250 at speeds of up to 120 km/h.
- Any road profile consists of a number of sinusoidal curves, each with a different length and hence, the profile can be subdivided into various sinusoidal curves. Fig. 3.55b shows that the true profile consists of curves with a long wavelength as well as curves with shorter wavelengths.

As previously stated, deviations with different wavelengths can be found in the road profile, but not all of them are important for road roughness measurements. The effects depend on the frequency of the deviations, which are a function of the vehicle speed and the range of the wavelength of the deviations.

$$f = \frac{v}{\lambda} \qquad [3.31]$$

Where f is the frequency (Hz), v is the vehicle speed (m/s) and λ is the wavelength (m).

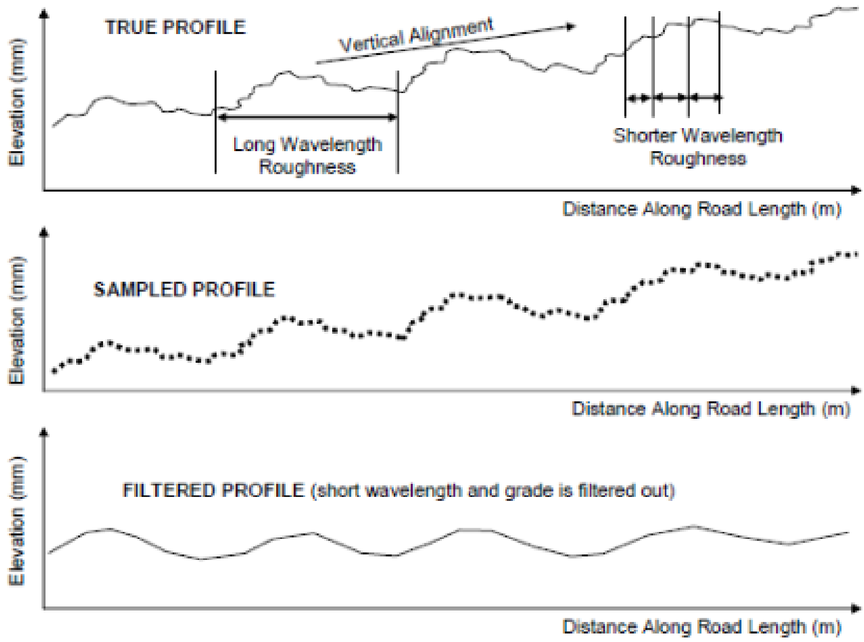


Fig. 3.55. Profile measurement concepts (COTO, 2007).

The users' comfort decreases when pending masses (chassis, seats) are in resonance, which occur with frequencies from 1 to 4 Hz, and this happen with wavelengths between 1 m and 30 m. For speeds over 40 km/h, discomfort occurs with wavelengths over 3 m. Road safety can also be affected with unevenness, and it happens with wavelengths between 0,5 and 8 m, when moving at speeds from 40 to 120 km/h (Achutegi Viada, 2005).Very long wavelengths related to vertical alignment or slope are typically not important, as are very short wavelengths related to surface texture.

When a road profile is processed to compute roughness, the wavelengths outside of the critical range are typically filtered out. There are many types of filters that can be applied to a measured profile. These filters can be a mathematical function (like a moving average) or a mechanical filter consisting of the suspension of a measurement vehicle. Fig. 3.55c shows the measured profile after the very short wavelengths and grades have been filtered out. Further discussion about filters is provided in section 3.4.5.3.

3.4.3. Measuring techniques

There are two basic types for measuring road roughness:

- Profilometric Type Measurement. It implies the measurement of the road profile, and then, the profile is filtered and then further processed to determine the road roughness parameter over segments of the measured profile. The filtering and processing of the road profile is designed to simulate and represent the response of standard vehicles to the measured profile.
- Response Type Measurement. They are deployed to directly measure the response of a measurement

vehicle to the travelled section of road. Instead of a profile of the road, they measure and quantify the measurement vehicle's response to the profile. Essentially, the measurement vehicle's suspension is filtering out the unimportant wavelengths and is quantifying the effect of the important wavelengths. The parameter measured by this type of devices is the Average Rectified Slope (ARS), which is the total up and down movement of the suspension normalized by the distance travelled. ARS is normally expressed in m/km.

The main difference between both techniques is that Response Type devices apply a physical filter to the actual road profile, whereas profilometric methods apply a mathematical filter to a measured profile (Fig. 3.56).

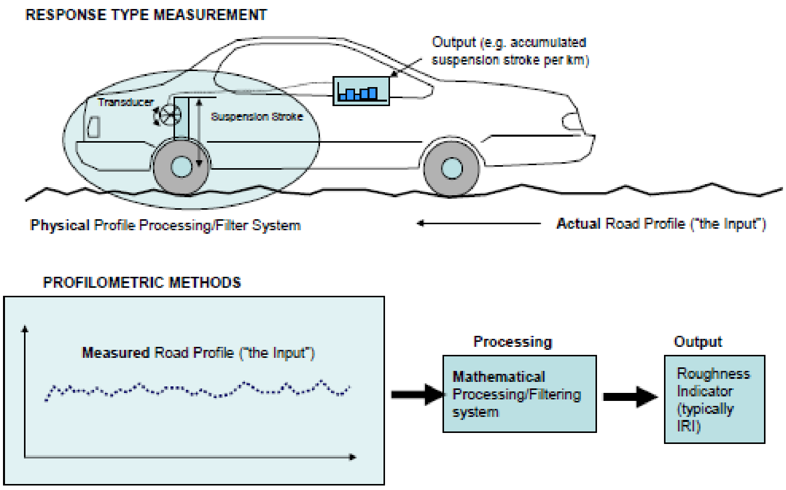


Fig. 3.56. Roughness measurement types (Response and profilometric types) (COTO, 2007)

3.4.4. Longitudinal roughness measuring devices

A wide range of devices and tools have been employed over decades, from simple straightedges, which indicate localized deviations in the surface, to inertial profilers equipped with laser sensors that record elevation measurements along the pavement. Table 3.8 shows a list of the usual types of roughness-measuring devices, with a brief summary of their respective advantages and disadvantages.

3.4.4.1. Precision profile measuring devices

As it will be commented in section 3.4.6.2, the response type devices need to be calibrated by precision profile measuring devices. In order to determine the real road profile, the classic topography principles can be employed, by means of the precision road and level (Fig. 3.57). The operation is very similar to that employed for surveying, etc.

It must follow the ASTM Standard E1364-95 (ASTM, 2017b), which requires at least two persons and is time consuming and labour intensive. A usual profile reading implies around 260 readings, and experienced team`s performance is approximately 600 m/day. Thence, this method is only suited for measuring profiles on calibration sections, for research or construction control purposes.

Table 3.8. Summary of roughness measurement equipment (Smith et al,. 1997; Grogg and Smith, 2002; Sayers and Karamihas, 1998; Perera et al., 2008)

Equipment / Device	Advantages	Disadvantages
Rod and level	• Easy to use • Accurate data measurement	• Not suitable for network level data collection
Straightedge	• Easy to use • Useful to analyse localized roughness issues	• Not suitable for network level data collection
Dipstick	• Easy to use • Accurate data measurement • Suitable for calibrating other devices	• Not suitable for network level data collection
Walking profilers	• Easy to use • Accurate data measurement • Suitable for calibrating other devices • Results provided instantaneously on equipment's display	• Not suitable for network level data collection
Profilographs	• Easy to use, lightweight • Able to collect continuous profile data • Analog trace of pavement deviations • Locates bumps and most grinds	• Slow operating speeds • Lack of precision • It does not provide true profile • May not relate to user response • Not suitable for network level data collection
Response-Type devices	• Data is collected near or at highway speeds • Reasonably accurate and reproducible	• Results depend on the mechanical system and the speed of travel • It does not provide true profile • Calibration over a range of speed and pavement roughness levels is required • Poor comparability of roughness results between devices • Measurement are not stable with time
High-Speed Inertial Profilers	• Data collection at highway speeds, does not need lane closure • High repeatability and accuracy • Measures true road profile	• Higher initial cost • Unable to collect accurate data at low speed (urban areas) • Not useful to quality control during paving operations
Lightweight inertial profilers	• On concrete pavements, can be used for testing before concrete has gained sufficient strength for opening to regular traffic • Measures true profile • Identifies areas of bumps and dips	• Not suitable for network level data collection • Requires lane closure

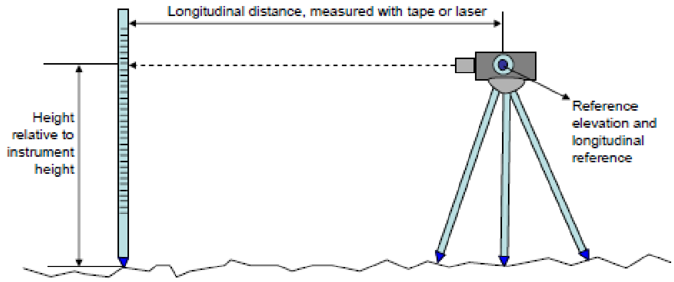


Fig. 3.57. Operation of the precision road and level (adapted from Sayers and Karamihas, 1998).

In order to simply the operation, other devices were developed. The most employed are the Dipstick and the Walking Profiler devices. The Dipstick, or pivoting Dipstick, "walks" along the line that is profiled (Fig. 3.58). The device contains a precision inclinometer that measures the height between the two support feet at the base of the instrument (Sayers and Karamihas, 1998). These feet can be spaced 20 to 500 mm apart, being 250 mm the usual spacing (Achutegi Viada 2005). The Dipstick is moved by leaning all of the device weight onto the front foot, and then pivoting the rear foot around the front foot by 180 degrees. When the instrument has stabilized, the change in elevation is automatically recorded and a beep is sounded. The longitudinal distance is determined by multiplying the number of measures made with the known spacing between the contact feet and it can also calculate the IRI value directly by means of the software installed in the device (Sayers and Karamihas, 1998, Achutegi Viada 2005). Usual performance of the Dipstick is approximately 150 - 200 m per hour.

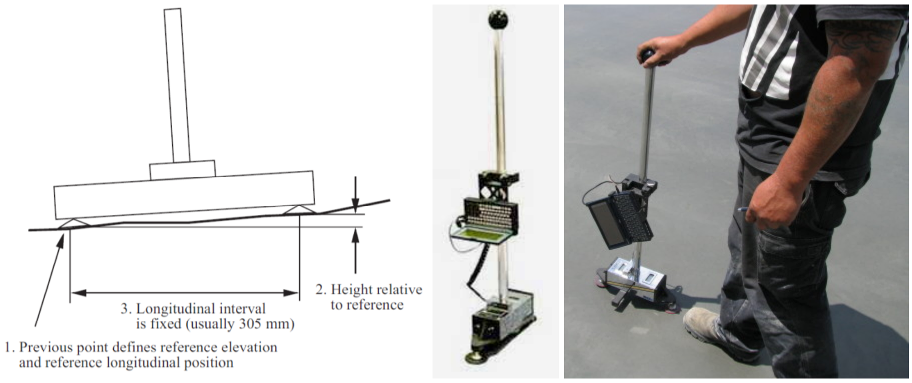


Fig. 3.58. Dipstick and Dipstick in operation (Sayers and Karamihas, 1998).

There are other prototypes of walking devices that aim to substitute the dipstick with a higher performance (1 or 2 km/h). Among other, the ARRB Walking Profiler can be cited, developed by the Australian Road Research Board Ltd (Fig. 3.59).

Fig. 3.59. ARRB Walking Profiler (COTO, 2007)

3.4.4.2. Geometric reference devices

3.4.4.2.1. Straightedges and rolling straightedges

Among the geometric reference devices, the straightedges have been used in almost all the countries to controlling initial pavement smoothness. The most employed length is 3 m, used in France, Belgium, United Kingdom, Spain and some states from the USA. Longer lengths can be found in Germany 4 m), in other states of the USA (from 3,60 to 4,80 m) and Sweden (5 m).

It is easy to use and it serves for controlling initial pavement smoothness, in a longitudinal and transverse direction, although it cannot indicate some deficiencies with certain wavelengths (Achutegi Viada, 2005). An improvement from the fixed straightedges is the rolling straightedge. They measure the vertical deviation at the centre of the straightedge to the profile Fig. 3.60. Rolling straightedges have a better performance than the fixed ones and allow a continuous data collection of the longitudinal profile. Nevertheless, they only detect some wavelengths (Fig. 3.61). As seen, a 3-m rolling straightedge does not detect wavelength of 1,5 m and duplicates the amplitude of wave of 3 m. Old and current rolling straightedges are shown in Fig. 3.62.

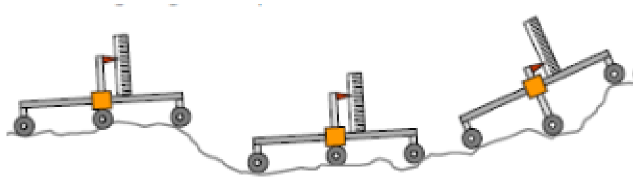


Fig. 3.60. Measuring principle of the rolling straightedge.

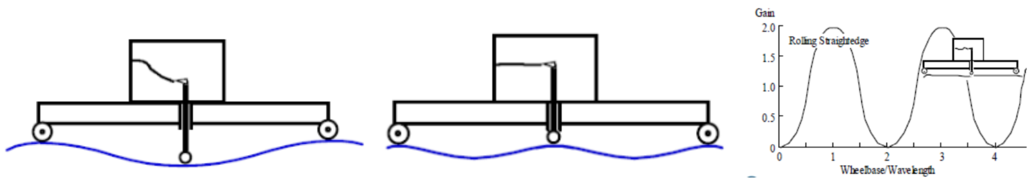


Fig. 3.61. Performance of the rolling straightedge with different wavelengths.

Fig. 3.62. Rolling straigthedges. A) And old rolling straightedge, b) an ODOT rolling straightedge circa 1940s – 1950s, and c) at present.

3.4.4.2.2. Profilographs

In order to improve the previous devices, some profilographs were developed. Profilographs are low-speed profile measurement systems (3 to 8 km/hr) that consist of a rigid frame, a centre profiling wheel, and a system of support wheels to provide a datum. Originally, profilographs mechanically recorded data on a strip chart recorder that was linked to the profile wheel, but today most profilographs are computerized and record data electronically.

Two basic profilograph models are in use at present: the California profilograph (Fig. 3.64) and the Rainhart profilograph ( ). The California profilograph uses between four and twelve wheels mounted on a 7.6 m frame, while the Rainhart device uses twelve support wheels evenly spaced along a 7.5 m frame at offsets up to 560 mm so that no wheel follows the same path (Smith *et al*., 1997). Consequently, the datum for the Rainhart device is established over the entire length of the unit and over a width of 1,118 mm. On the other hand, the datum for the California type is established near the end of the 7.6 m beam.

The profilograph measures wavelengths between 0.3 and 23 m, amplifying or attenuating wavelengths that are factors of the profilograph length. Thus, the device can introduce some bias into the results, depending on the wavelengths of the pavement profile. The output of the profilograph is a smoothness statistic referred to as the profile index (PI) (Smith and Ram, 2016).

3.4.4.3. Response-type devices

Generally, these devices consist of an instrumental installed in a vehicle or trailer, which record the up-and-down movement of the suspension, called the suspension stroke. They aim to reproduce and record the bump sensation perceived by the user when moving through an irregular pavement. Response type devices are also called Response-Type Road Roughness Measuring Systems (RTRRMS), or Road Meter systems (Sayers and Karamihas, 1998). The vehicle used in these devices may be a passenger car, a van, a light truck, or a special trailer. A road meter is a transducer that accumulates suspension motions. There are some popular brands were the Mays Ride Meter (Fig.3.78), the PCA meter, the Cox meter, etc. Response type devices have been used for decades to measure road roughness. They first appeared in the 1920s, with the Via-Log in the state of New York. Almost all of the road meter designs are based on the concept of the Bureau of Public Roads (BPR) Roughmeter. A RTRRMS consists of the following main components (Fig. 3.66):

- The measurement vehicle
- A transducer that detect the relative movement of the suspension
- A recording system and display, connected electronically to the transducer
- Automatic speed control and accurate distance measuring instruments.

Fig. 3.64. California profilograph (Smith and Ram, 2016).

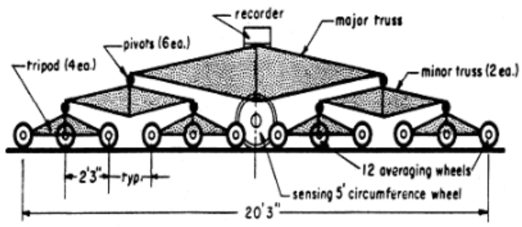


Fig. 3.63. Rainhart profilograph (Budras, 2001)

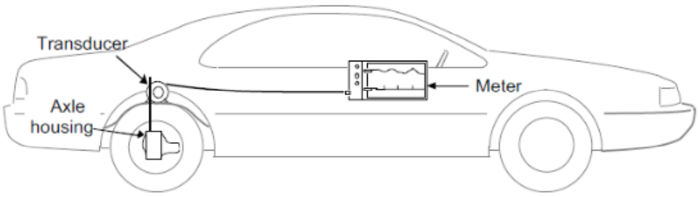


Fig. 3.65. A car with a Mays meter (Sayers and Karamihas, 1998).

Fig. 3.66. Components of a RTRRMS devices. Left: vertical measurement transducer, odometer and data capturing components. Right: Attachment of suspension monitoring device to rear axle (COTO, 2007)

The transducer, recording system and display are normally manufactured and sold as a single system (often called a Roadmeter), measuring the response of the vehicle to the road profile at the measurement speed.

There are accelerometer based systems, like the Roadmaster, the ARRB Roughometer and devices that employs a bump integrator, like the CSIR the LDI, the ROMDAS, TRL Bump Integrator.

RTRRMS can measure pavement roughness of up to 300 km per day. Engineers generally agree that measures obtained with response type systems match their experience of pavement roughness and overall condition (Sayers and Karamihas, 1998). Even when the vehicle is standardized, differences appear between similar vehicles. Moreover, the response properties of the vehicle change with time and hence, the results depend on the dynamics of the host vehicle. Consequently, they require calibration before been used in order to provide IRI values. Response-type devices have some advantages:

- They were employed worldwide for decades. Operation and output of these devices are well known.
- Generally, their output are known to agree with engineer's evaluation of roughness and pavement condition
- They are relatively inexpensive.
- Although they require frequent maintenance and care of operation to ensure that the calibration remains valid, maintenance and operation care are relatively simple and inexpensive
- The calibration process is relatively easy and inexpensive to perform once calibration sections have been set out and measured.

And they also have some disadvantages.

- Their precision (and hence, their repeatability) is significantly lower than other profiling devices (Class 1, section 3.4.6.2.1). Moreover, the annual deterioration or roughness is often smaller than the measurement error of these devices (The error appears because of the lack of high precision and due to errors inherent in the calibration to correlate with IRI). This implies that this equipment cannot assess the deterioration of a road network annually.
- The transformation of the road profile to an IRI value totally depends on the properties of the vehicle suspension system, which are known to change over time, and also differs from one vehicle to another. Consequently, response-type devices require annual calibration
- They only measure road roughness. Other modern equipments can carry out other functions as high resolution photographs or videos of the road surface simultaneously.

3.4.4.4. High speed profiling devices or Inertial Profilers

The profilographs, apart from the disadvantage of not being capable of measuring long wavelengths, related to users' comfort, they perform at low speeds. To overcome these deficiencies, the high speed profiling devices were developed. They are able of measuring a precision road profile of one or more wheelpaths while moving at speeds higher than 100 km/h.

The first high speed profiling device appeared in the 1960s (Sayers and Karamihas, 1998). From then, significant advances in precision measurement technologies have conducted, aiding the design of these devices, which are widely used for road data collection.

High speed profilers are also referred to as Inertial Profilers, due to the accelerometers to determine an inertial reference which provides the instantaneous height of the measurement base continuously when the vehicle is moving. They consist of the following main components (Fig. 3.67):

- The measurement vehicle
- A height sensor (called a transducer), which measures the distance from the measurement base to the road surface. There are four types of height sensor commonly in use: laser, optical, infrared and ultrasonic sensors.
- Accelerometers to measure vertical acceleration of the measurement base. The accelerometer reading is employed in conjunction with the height sensor output to determine the elevation of the road surface.
- A distance measuring system to measure the longitudinal distance along the road profile.
- A computer and data storage system to process the output from the height sensor, accelerometer and distance measuring system, compute the surface profile (or profiles, if both wheelpaths are measured), and store the produced profile with other parameters like vehicle speed, position coordinates, etc.

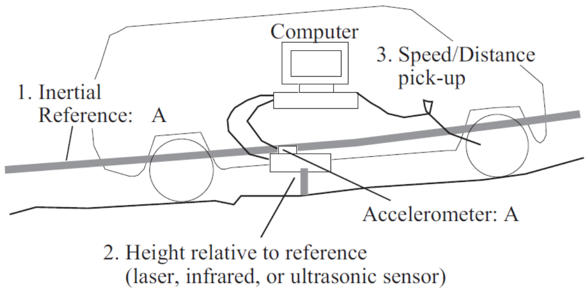


Fig. 3.67. Inertial profiler (Sayers and Karamihas, 1998)

The height sensors, accelerometers and distance measuring equipment are calibrated in the factory and remain calibrated for a long time. Hence, and unlike response-type devices, precision of inertial profilers does not need to be calibrated as part of a measurement process. Instead, the output of the device is validated by measurement of test section with known IRI values to determine if all of the components of the system work correctly and that the device is able to measure the road profile to the required level of precision. There are two types of inertial profilers:

- **High Speed Inertial Profilers** (Fig. 3.68). These devices are considered to be highly accurate, are employed widely to collect roughness data at the network level. The measurement equipment is mounted on the front or rear of the data collection vehicle and the measurements are collected at posted speeds. Additionally, the modern devices can collect other data such as right-of-way video, downward imagery of pavement surface (to enable pavement condition surveys), sign and signal inventory, highway grade, cross-slope, etc.
- •**Lightweight Inertial Profilers**. These types of profilers (Fig. 3.69) use the same technologies used in high-speed systems, but in smaller, lightweight vehicle, making them ideal for testing new pavement construction, and particularly newly constructed concrete pavements that have not yet achieved sufficient strength to support regular traffic loading.

Some examples of high speed profilers are the K. J. Law Model 3300 A, which employs ultrasonic height sensors, the ARAN, from Canada, which employs ultrasonic or laser-based sensors, and the British HRM, the Swedish RST and the Danish Greenwood, which employs laser sensors.

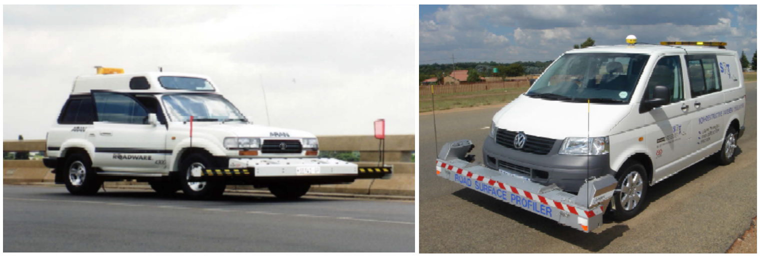


Fig. 3.68. High Speed Inertial Profilers (COTO, 2007).

Fig. 3.69. Fig. Lightweight Inertial Profilers (COTO, 2007).

The High Speed Profiling devices imply some advantages:

- They are able to measure the pavement surface profile very precisely. Moreover, as the IRI transform employs a fixed computer algorithm, the processing constants remain invariable and hence, the measured IRI remains constant.
- Due to the stability and precision of IRI values obtained by means of a validated high speed profiler, those IRI values can be compared with other from other years, and therefore, deterioration of the network can be assessed.
- Modern devices can measure longitudinal and transverse profile simultaneously and hence, providing roughness and rutting data at the same time. Additionally, some devices can also obtain a high resolution video of the pavement surface, which can be used for distress evaluation.

However, there are also some disadvantages:

- Devices that uses laser height sensor are not adequate for profiling gravel roads.
- Modern equipment is relatively expensive, and not all the road agencies can afford and maintain it.
- Despite the fact that the components of these profilers are calibrated by the manufacturer, extensive validation testing and control procedures are still needed to ensure the accurateness of the measured profile. If rigorous validation procedures are not followed, the results are only a little more accurate than those of calibrated response type device. The validation procedures are costly.
- The operation and control procedures of these devices are relatively complex, and if used by road agency employees, they need time to understand and participate in the validation and control procedures.

3.4.5. Roughness indices

In this section, the most used roughness indices are presented. They can be classified in the following types:

- Opinion-based indices
- Indices obtained from a response-type device.
- Indices based on the measurement and analysis of the longitudinal profile

3.4.5.1. Opinion-based indices

The opinion of the experts in charge of the maintenance has been a traditional base for the pavement management. As some road agencies from the USA wanted to satisfy users, they collect users' opinion by means of queries, which became a roughness index. From then, the first approach employed to assess roughness was a qualitative rating system, Present Serviceability Rating (PSR), which later led to an objective quantitative index, the Present Serviceability Index, PSI.

The Pavement Serviceability Rating (PSR) was developed in the late 1950s in the AASHO Road test, finished in 1961, and it is based on some users' opinion, contrasted with engineers' opinion. It is a subjective 0 to 5 rating scale, where 0 means Very Poor and 5 Very Good, and serve to express the condition of the pavement in terms of the ride quality (AASHO, 1962; Carey and Irick, 1960). According to the AASHO Terminology, each rating is Individual Present Serviceability Rating, and the mean of the individual ratings is the Present Serviceability Rating (PSR).

Additionally, the Pavement Design and Evaluation Committee of the Canadian Good Roads Association (CGRA) (currently the Transportation Association of Canada (TAC/ATC) developed a similar and widely employed technique in the late 1950s and early 1960s, the Riding Comfort Index (CGRA, 1959; 1965; 1967; 1971). It was originally called the Present Performance Rating but it was changed to Riding Comfort Index (RCI) to express more explicitly the assessment of pavement riding quality only (Wilkins, 1968).

The main difference between the two approaches is the construction of the scales. There are five descriptive clues in each, however, the Canadian scale has 10 numerical categories while the AASHO scale only uses 5 categories. Both approaches emphasize that only the descriptive words are employed by raters when evaluating pavements. The numerical rating is scaled from the forms during data processing.

In addition to the rankings obtained from the panel of raters, in the AASHO tests, several measures with response type devices were made, and through statistical analyses, results were used to estimate the PSR. The estimate of the PSR by means of a multiple regression was called Present Serviceability Index (PSI).

The PSI was the first to relate the objective measures of surface condition to the users' perception of serviceability. In fact, it gathers the surface roughness (longitudinal and transverse profiles (ruttings) and other aspects of the pavement distress condition (patching and cracking). It has been a very used index, especially in the USA for pavement management. For flexible pavement, the equation is:

$$PSI = 5{,}03 - 1{,}91 \cdot \log(1 + S_V) - 0{,}01\sqrt{C + P} - 1{,}38 \cdot R_d^2 \qquad [3.32]$$

Where *SV* is the slope variance, the mean slope variance of the two wheelpaths obtained from the CHLOE profilometer, R_d is the mean rut depth as measured by simple rut depth indicator, and *C* and *P* are the amount of cracking and patching area, determined by procedures developed at the AASHO Road Test.

For rigid pavements:

$$PSI = 5{,}41 - 1{,}8 \cdot \log(1 + S_V) - 0{,}09\sqrt{C+P} \qquad [3.33]$$

It must be noted that any PSI model, represented by equations 3.32 and 3.332 is not an end in itself. It is only an attempt to predict PSR to satisfactory approximation (Carey and Irick 1960). As previously said, the PSI equation was developed by multiple regression techniques, i.e. a set of physical measurements were correlated to the subjective user evaluations (PSR). Although condition and distress data are present among variables, roughness provides the major correlation, and sometimes, it is the only variable employed (Achutegi Viada 2005). Nevertheless, in any case, if PSI is calculated from physical measurement data, it is only an estimate of PSR (Eq. 3.34):

$$PSI = PSR \pm e \qquad [3.34]$$

where e is an error term. Briefly, PSI and PSR are not two different ways of obtaining pavement serviceability; PSI is only a way to use objective data for estimating a subjective parameter (Carey and Irick, 1962; Haas and Hudson, 1971).

Although PSR and PSI are falling out of use, some agencies still make correlations between smoothness statistics and PSI values, and PSI is the basis for the AASHTO 1986/1993 pavement design procedure (Smith and Ram, 2016).

Another indicator based on some peoples' opinion is the Mean Panel Rating (MPR). It is a subjective indicator that is affected by the instructions given to the panel members and the way to calculate the mean. Usually, instead of calculating the average rating, ratings are re-scaled before being averaged. Thus, the MPR is not necessarily the mean of the original ratings of the panel members. However, there are two problems with using MPR data directly for a network level evaluation. First, it is expensive to obtain panel rating due to the number or people needed and the need to transport them to the roads to be evaluated. Secondly, the rating scale is not a measure of road condition that is stable with time. The perception of the necessary quality can change in the long-term.

Subjective panel ratings depend enormously on the instruction given to the members, and they must be trained. Consequently, the NCHRP sponsored two research projects in the 1980s to develop a methodology in order to obtain valid ratings, resulting in the concept of Ride Number. These projects investigated the effect of road surface roughness on ride comfort (Janoff *et al*., 1985; Janoff 1988). Thus, the Ride Number (RN) was defined, as an estimate of the MPR, based on the exponential transformation of the Profile Index (PI) (described in section 3.4.6.3.2).

$$RN = 5 \cdot e^{-160 \cdot PI} \qquad [3.35]$$

The RN employs the same 0 to 5 scale as PSI, as it is familiar to the highway community, mainly in the USA.

3.4.5.2. Indices obtained from response-type devices

The transducer of a response-type device measures the movement of the suspension in "counts" or millimetres according to the specific dynamic characteristics of the vehicle. When the counts or total mm are summed, a parameter is obtained which gives an indication of the total suspension stroke that occurred over the length of road travelled. When the total count of summed mm of travel is divided by the length of the test section, the Average Rectified Slope (ARS) is obtained. It has units of slope (mm/m) or (in/mi) ,although sometimes arbitrary units are also utilized based on the instrumentation hardware, for example, “counts/mi”

As exposed in section 3.4.6., the most preferred parameter for quantifying pavement roughness is IRI and, hence the output from response type devices must be converted to IRI. This is carried out by means of a procedure called “correlation by calibration”. There, the output from a RTRRMS is correlated with the known IRI values of several selected sections of road. These sections are referred as calibration sections and the IRI values of these sections need to be determined beforehand using one of the high precision profiling devices discussed in section 3.4.4.1.

This process of calibrating a response type device provides an equation which can be used to calculate an estimated IRI from the response type device output. It is critical to note that the calibration of a response type device is valid only as long as key aspects of the measurement vehicle (shock absorbers, tyres, loading, etc.) remain unchanged. The calibration is also performed only for a specific speed, which should be 80 km/h since this is the speed for which the IRI transform is designed.

3.4.5.3. Indices based on the measurement and analysis of the longitudinal profile

3.4.5.3.1. Introduction

The longitudinal profile contains the information that can be used to assess the level of comfort or discomfort that a road user experiments when travelling at a certain speed. The variations in the road profile causes vibration in the vehicle body, which are transmitted to the road user.

As previously exposed, not all the wavelengths in a road profile are important for users’ comfort. Vehicle suspension systems are design to remove or dampen the effect of many of the wavelengths in a road profile. Hence, some wavelengths of the profile will have a greater impact on perceived roughness than others. To quantify the degree of comfort or discomfort, the measured profile first needs to be processed or filtered to isolate and “add-up” the amplitudes and variations of the most important wavelengths in the profile. The processing of the road profile typically results in a number or parameter which is used as an indication of road roughness

Fig 3.83 shows an idealized road surface that varies following a sinusoidal pattern It represents a pavement surface where speed bumps are very close. The figure has the following mathematical equation

$$Y = A \cdot \sin\left[\frac{2\pi(X - X_0)}{\lambda}\right] \qquad [3.36]$$

Where X is the distance from the initial reference, Y is the elevation in point X, A is the amplitude of the wavelength, λ is the wavelength and X_0 is the phase shift.

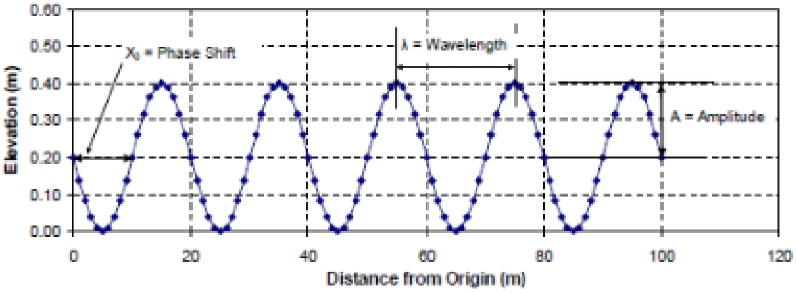


Fig. 3.70. Road profile showing a sinusoidal variation (COTO, 2007)

The curve representing the road surface consists of three main components:

- Amplitude. It is the absolute deviation from the neutral line. According to Fig. 3.70, road roughness varies 200 mm up and down from the neutral axis.
- Wavelength: It determines the length of the full cycle or wave, i.e, the distance from one crest to the next after a full cycle. In Fig. 3.70, the wavelength is 20 m,
- Phase shift. It represents the point where the first full cycle starts. It is basically determined by the reference point of the distance measurement, relative to the value of the sinusoid.

Road roughness is dependent on the amplitude and the wavelength. If the amplitude is very small (1 or 2 mm), the tyres will absorb the roughness and it will not be transmitted to the suspension system. If the wavelength is very long (for example, 70 m) and the car speed is 80 km/h, the driver will experience the bumps as slight undulations, because the vehicle will float over the bump. In this case, the suspension system will absorb (or "filter out") the long wavelength almost completely. With a shorter wavelength, for example 5 or 10 m, the crest of the bumps are closer and although vehicle suspension will dampen some of the roughness, much of it will be transmitted to the road user. The influence of the wavelength can also expressed by the inverse of the wavelength expressed in metre. This parameter ($1/\lambda$) is the wave number and defines how many cycles occur in a unit of length. Its units are cycle/m. The higher the wave number, the more waves per unit length and, hence, the shorter the wavelength

Although Fig. 3.70 represents a highly idealized profile and no road would have such a profile, it is interesting to note that a road profile can be constructed by adding sinusoids like the Fig. 3.71, but each of them with varying properties. 4 sinusoids are shown in Fig. 3.71, each one with a different wavelength (or wave numbers) and amplitude. If we sum the 4 curves we get the profile shown in Fig. 3.72. This curve is a more realistic representation of a road profile. It looks almost "random" and any cyclic sinusoidal pattern is hard to discern.

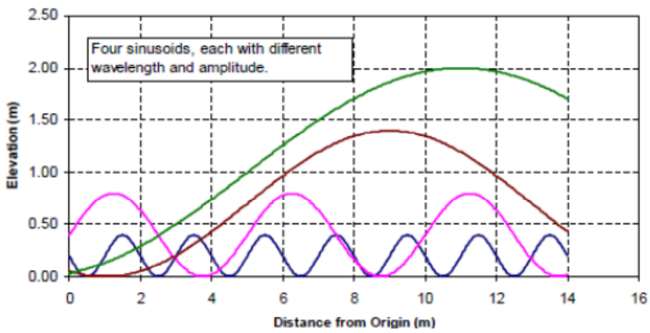


Fig. 3.71. Four different sinusoids (COTO, 2007).

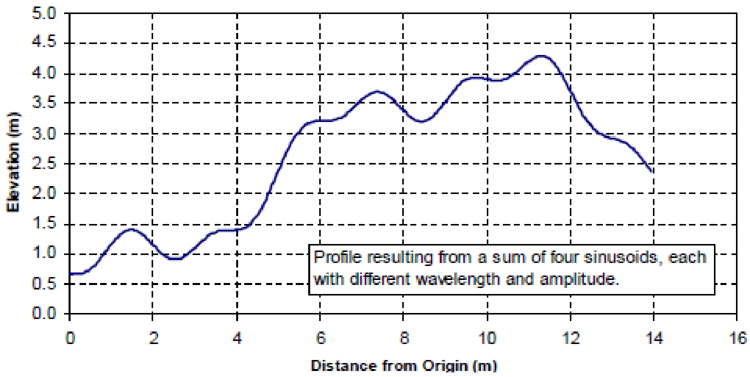


Fig. 3.72. Profile resulting from a sum of four sinusoids (COTO, 2007)

If more sinusoids are added, a virtually random profile can be obtained. The opposite approach can also be carried out, in which, from a random profile (as the ones that can be measured on a road), it can be split into separate sinusoids, each one with a different wavelength and amplitude, as in Fig. 3.71. The information obtained from this splitting up of the road profile into sinusoids can be very useful. A profile consisting of sinusoids with short wavelengths (high wave numbers) and large amplitudes will result in a bone jarring ride. On the other hand, a profile consisting of long wavelengths (low wave numbers) results in a more nauseating wave-like motion of the vehicle. The process of splitting up the profile into different sinusoids and the analysis of the resulting data is generally called Power Spectral Density (PSD) analysis. The term power comes from the first use of this approach in electronics, related to voltage and its variations. Spectral Density refers to the analysis of the density or composition of the spectrum of sinusoids which make up the measured road profile. The analysis of sinusoids is summarized by means of a PSD plot (Fig. 3.73). Generally, a characteristic PSD from road profiles has the curve slopes down toward the right, as the amplitude decreases as the wave number increases (or the wavelength decreases). This happens because shorter wavelengths (higher wave numbers) generally have lower amplitude than those of longer undulations-type roughness.

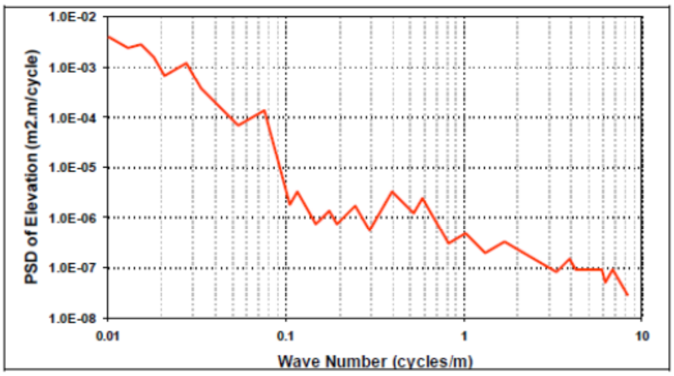


Fig. 3.73. Example of a PSD Plot (COTO, 2007)

It must be underlined that the splitting up of a random road profile into its constituent sinusoids is not a straightforward exercise, as there can be an almost infinite number of sinusoids that makeup the profile, and their wavelength and amplitudes are unknown. In fact, PSD analysis is what determines. PSD analyses are usually carried out by means of Fourier Transforms or similar techniques.

Another important use of the PSD is to troubleshoot and analyze the accuracy of profilers. Some profilers, due to the way in which they record elevations, effectively “lose” or filter out some wavelengths. The characteristics of a specific profiler could thus be evaluated at a fundamental level by comparing the PSD of a profile measured with one device with the power spectral density measured with another benchmark device. This technique has advanced to the point where some network managers are using power spectral density as part of the validation of a profiler before a network survey (Prem, 1998; Fong and Brown, 1997).

Fig. 3.74 shows how a PSD analysis can be employed to understand a roughness profile. While curve A (red curve) indicates a profile made of many short wavelengths (high wave number) and low amplitudes, curve B is made of longer wavelengths (low wave number) and higher amplitudes. The PDS plot clearly separates the two profiles and shows that curve B has higher amplitudes for long wavelengths (low wave numbers). On the other hand, curve A has higher amplitude for short wavelengths, or higher wave numbers.

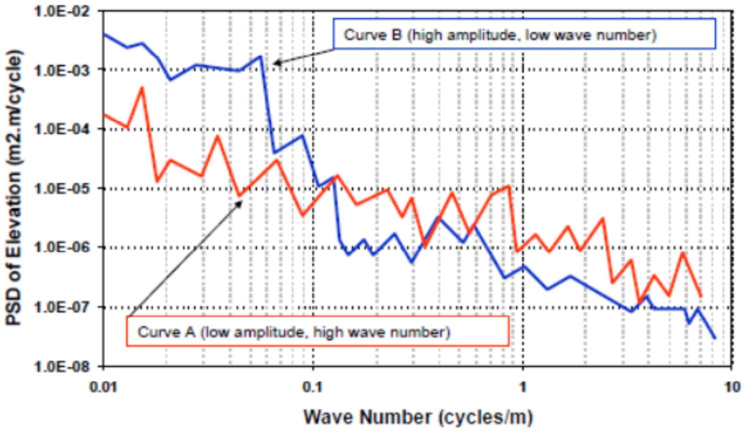


Fig. 3.74. PDS Analysis of two profiles (COTO, 2007)

A filter is a transform that is applied to a measured series of data to filter out or remove some of the information. The filter, or transform, can be a mathematical function or a physical filter, such as the suspension of a road profile, which filters the profile elevation into a series of counts.

For instance, the profile in Fig. 3.76 is considered, which is identical to the one showed in Fig. 3.72, and it is constituted of four sinusoids shown in Fig. 3.71, but extended over a longer length of the road. If the analysis does not require the sinusoids with the shorter wave numbers (long wavelengths), it is possible to filter the influence of this sinusoids, and other with similar wave numbers. There are a wide variety of filters that can be used to achieve it. One simple way to achieve such filtering is by taking the moving average over a length that is roughly equal to the wavelengths tried to be filter out (Fig. 3.75). Fig. 3.76 shows the original profile, the one from Fig. 3.72, with a filtered profile consisting of the moving average over 6 metres. This moving average is called a smoothing or low-pass filter (the red line in Fig. 3.76) and can be expressed by Eq. 3.37.

$$p_{fL}(i) = \frac{1}{N} \sum_{j=i-\frac{B}{2\Delta X}}^{i+\frac{B}{2\Delta X}} p(j) \qquad [3.37]$$

Where p_{fL} is the smoothed profile (also called a low-pass filtered profile, B is the base length of the moving average, and N is the number of samples included in the summation. The effect of the moving average filter is to smooth the profile by averaging out the point-by-point fluctuations (Fig. 3.75).

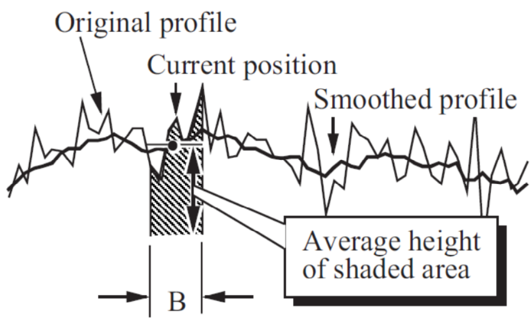


Fig. 3.75. The moving average filter (Sayers and Karamihas, 1998).

If compared the moving average line in Fig. 3.76 to the original, it can be observed that the smoothed profile mainly gives the indication of slope changes (large elevation changes). Hence, it is not very useful for roughness evaluation purposes. Nevertheless, it is possible to apply a second filter in which the original profile is subtracted to the moving average value at that point. This new profile gives the indication of the how much the profile deviates from the smoothed profile at each point because the main objective is to know the deviation from the smoothed profile, which really degrade vehicle ride and annoy the travelling users. This filter is called an anti-smoothing (or high-pass) filter (Eq. 3.38), denoted in blue in Fig. 3.76.

$$p_{fH}(i) = p(i) - \frac{1}{N} \sum_{j=i-\frac{B}{2\Delta X}}^{i+\frac{B}{2\Delta X}} p(j) \qquad [3.38]$$

As observed, the final filtered profile has removed much of the larger up-down movements in the profile, and now highlights the roughness with higher wave numbers (shorter wavelengths).

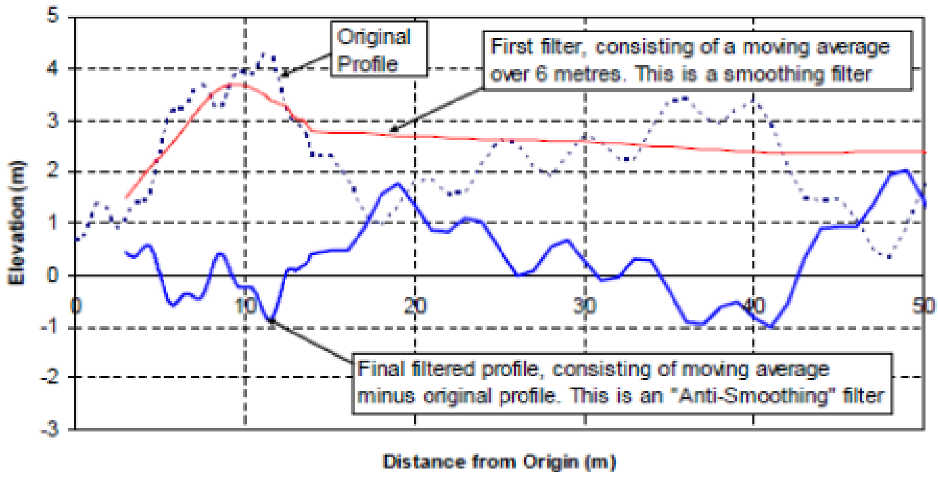


Fig. 3.76. Filtered profiles (COTO, 2007)

Both the smoothing (low-pass) and the opposite (high-pass, anti-smoothing) forms of the filter are useful. Both versions can be used on the same profile, although it only makes sense to do it if the base length B is longer for the anti-smoothing version than the base for the smoothing version.

There is no single best base length for profile interpretation; the best setting depends on the use to be made of the data (Sayers and Karamihas, 1998).

With regard to the roughness analysis, it is also necessary to talk about frequency response, which describes the input/output behaviour of any of commented devices. It can be defined as the transfer function or response curve, *H(λ)*, the factor (which is a function of the wavelength λ) that multiplies the real elevation (the input) of a sinusoidal profile of λ wavelength to obtain the value (the output) measured by a device. If represented graphically, an amplitude frequency response plot is obtained, and it shows the ratio of the output amplitude to the input amplitude. The amplitude ratio is called the gain.

For instance, in the straightedge of L meters, the *H(λ)* function is:

$$H(\lambda) = 1 - \cos\frac{\pi L}{\lambda} \qquad [3.39]$$

Where *H(λ)* is the transfer function of the straightedge, λ is the wavelength and L is the length of the straightedge. The possible values are between 0 and 2. Fig. 3.77 represents the transfer function of a usual 3 m straightedge.

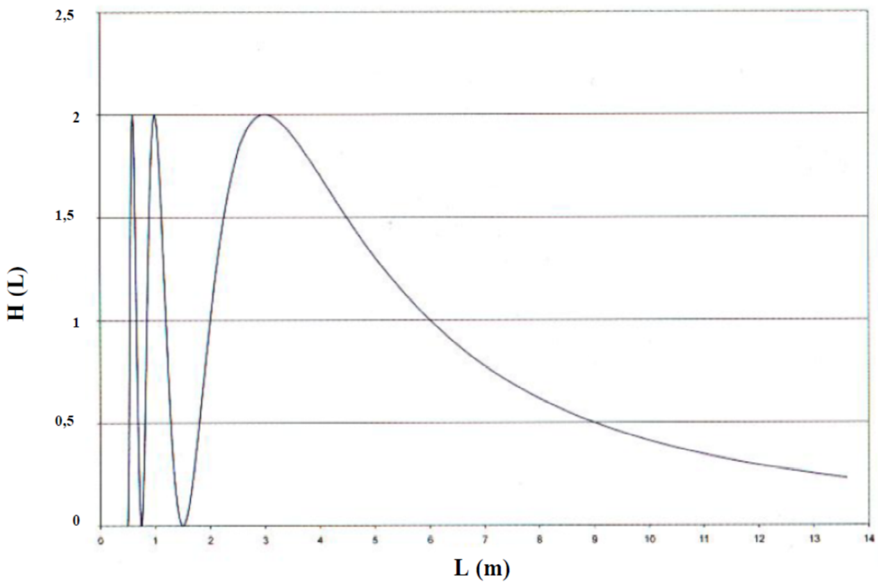


Fig. 3.77. Transfer function of the straightedge with a length of 3 m (Achutegi Viada,. 2005).

As observed, for wavelengths over 12 m, the response is attenuated, $H < 0{,}3$, and therefore, although it can be used for controlling initial pavement defects, it is not useful for assessing travelling public's comfort.

There are different indices that analyse the longitudinal profile.

With regard to the high-pass filters, the variance of the high-pass filtered profile with moving average of different wavelengths are used, being the bases 3 m, 10 and 30. They are referred as C3, V10 and V30, respectively. They are commonly used in the UK. Sometimes the standard deviation (SD) from moving average is calculated, with the same wavelengths as before, and SD3, SD10 and SD30 are obtained, which are more used in Germany (Achutegi Viada, 2005).

However, the most important and most used roughness index is the International Roughness Index, which is based on the quarter-car model, and it is deeply described and commented in next section.

3.4.6. International Roughness Index (IRI)

Due to the several roughness measuring devices, the World Bank initiated in 1982 a correlation experiment in Brazil called the International Road Roughness Experiment (IRRE) to establish correlation and a calibration standard for roughness measurements (Sayers *et al*., 1986a). When data was being processing, it was notices that nearly all the roughness-measuring devices in use around the world could produce measures on the same scale, as long as that scale had been selected suitably. Therefore, an additional objective was included in the experiment: the development of the International Roughness Index. For designing the IRI, some criteria were established: It had to be relevant, transportable and stable with time.

- To be transportability, the index needed to be measurable with a wide variety of devices, including response-type systems.
- To be stable with time, it had to be defined as a mathematical transform of a measured profile.

After applying many roughness definitions to the data obtained in the IRRE, many candidates were checked, included the Golden Car simulation from the NCHRP project (Gillespie *et al*., 1980), but the best correlations were found with two vehicle simulations based on the Golden Car parameters: a quarter-car and a half-car, both with the same level of correlations. Finally, the quarter-car was selected for the IRI because it could be employed with all profiling methods that were used at that time. The standard speed was agreed to be 80 km/h since at that speed the IRI is sensitive to the same wavelengths that cause vehicle vibrations in normal road use.

Due to the encouraging research findings, the World Bank decided to publish a guideline for conducting and calibrating roughness measurements, where instructions for using different types of IRI measuring devices are included (Sayers *et al*., 1986b). This guideline also provides has computer code to calculate IRI from profile. The International Road Roughness Experiment, many analytical comparisons of algorithms and some sensitivity analyses are available in a companion report (Sayers *et al*., 1986a). In 1990 the IRI was required as the standard reference for reporting roughness in the Highway Performance Monitoring system (HPMS) by the FHWA (1990) and since then, the IRI has become the most international index for pavement roughness measurement, it has been related to vehicle operation costs and helps prioritizing maintenance and rehabilitation works (Bernett *et al*., 2007; Smith and Ram, 2016; Nassiri *et al*., 2013; Prozzi and Madanat, 2004; Wu *et al*., 2013a, Zhou *et al*., 2013; Pérez-Acebo *et al*., 2017b; 2018a; El-Assaly *et al*., 2002; Pantha *et al*., 2010; Souliman *et al*., 2010; Abaza and Mullin, 2013; Khattak *et al*., 2014; Jannat *et al*., 2016).

Technically, the IRI is a mathematical representation of the accumulated suspension stroke of a vehicle, divided by the distance travelled by the vehicle during a test. Hence, it has units of slope (Sayers and Karamihas, 1996b). The IRI is characteristic of the longitudinal profile, equivalent to the average rectified slope, obtained from the mathematical simulation of the quarter-car, at an average speed of 80 km/h (Sayers *et al*., 1986a; 1986b). The main points that fully define the IRI concepts are (Sayers, 1995):

- IRI is computed from a single longitudinal profile.
- The profile is assumed to have a constant slope between sampled elevation points
- The profiles is smoothed with a moving average whose base length is 250 mm

- The smoothed profile is then filtered using a quarter-car simulation, with specific parameter values at a simulated speed of 80 km/h.
- The simulated suspension motion is linearly accumulated and divided by the length of the profile to yield IRI. Hence, IRI has units of slope, such as m/km.

As indicated, the IRI is defined as characteristic of a single wheel-track profile. If a device calculated some profiles at the same time, it is calculated independently for each. Sometimes, an alternative analysis carried out, measuring 2 profiles at the same time. The profiles are averaged point by point, and then processed using the IRI algorithm, resulting in the half-car simulation (HRI), which is not the same as IRI.

There were many reasons for defining IRI for a single wheel-track: (a) many profiler devices were only able to measure one profile at a time, (b) for methods like road and level or Dipstick the cost is doubled, (c) there was a high correlation between IRI and HRI.

Profile analysis is almost always conducted numerically. The profile is sampled to obtain a sequence of elevation numbers, where each one corresponds to a different location along the profile. The longitudinal separation between samples is a constant, Δ, which is dependent of the device used. The filter for the IRI quarter car is defined for a continuous profile, which implies an assumption about what the profile does between samples. Some methods are (Fig. 3.78):

1) Zero slope between points, meaning a discontinuity in elevation at each sample location.
2) Linear interpolation between points, meaning a constant slope
3) Quadratic interpolation maintains continuity in both elevation and slope through the sample values.

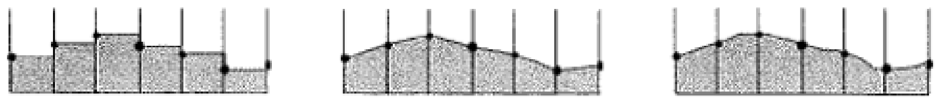


Fig. 3.78. Methods for interpolating between profile samples: left, zero slope, middle, linear interpolation; right, quadratic interpolation (Sayers, 1995).

For low values of Δ, on the order of 50 mm, the three methods five the same IRI values. For larger Δ-values, with option 1, results were high; with option 3 too low; and with 2 results were reasonably accurate. For larger sample intervals, results were two low even with option 2. This led to define the assumption that profile between sampled measures is a straight line connecting the points, but the sample interval was limited to 300 mm for accurate measures.

The IRI uses two filters; a moving average and a quarter-car model. The moving average was included to simulate the enveloping behaviour of tyres and to reduce the sensitivity of the IRI algorithm to the sample interval, Δ. For a profile sampled at Δ, a moving average smoothing filter is defined by the summation (Eq. 3.40 and Eq. 3.41)

$$h_{pr}(i) = \frac{1}{k} \sum_{j=1}^{i+k-1} h_p(j) \qquad [3.40]$$

$$k = \max[1, \mathrm{nint}(L_B/\Delta)] \qquad [3.41]$$

where h_p is the profile height, h_{pr} is the smoothed profile height, max is the maximum of two arguments, nint is the nearest integer and L_B is the moving average base length, 250m.

For example, if the sample interval is Δ = 150 mm, the ratio L_B/Δ is 1,67, which rounded to 1. The number 2 is larger than 1, so k is set to 2.

The quarter-car model of the IRI includes the major dynamic effects that determine how roughness causes vibration in the vehicle (Fig. 3.IRI2). The masses, springs and dampers are defined by the following parameters:

c_s = suspension damping rate

k_s = suspension spring rate

k_t = tyre spring rate

m_s = sprung mass (portion of vehicle body mass supported by one wheel)

m_u = unsprung mass (mass of wheel, tyre and half of axle/suspension)

To simplify the equations, the parameters are normalized by the sprung mass, m_s. The following values for the normalized parameters define the Golden Car data set:

$$c = c_s / m_s = 6{,}0\ s^{-1} \quad [3.42]$$

$$k_1 = k_t / m_s = 653\ s^{-2} \quad [3.43]$$

$$k_2 = k_s / m_s = 63{,}3\ s^{-2} \quad [3.44]$$

$$\mu = m_u / m_s = 0{,}15 \quad [3.45]$$

The behaviour of quarter-car is described by the following two second-order differential equations:

$$m_s \cdot \ddot{z}_s + c_s \cdot (\dot{z}_s - \dot{z}_u) + k_s \cdot (z_s - z_u) = 0 \quad [3.46]$$

$$m_s \cdot \ddot{z}_s + m_u \cdot \ddot{z}_u + k_t \cdot (z_u - z) = 0 \quad [3.47]$$

Where z is the road profile elevation points, z_u is the elevation of unsprung mass (axle), z_s is the elevation of sprung mass (body).

The quarter-car model can be also described by four first-order ordinary differential equations that can be written in matrix form (Eq. 3.48)

$$\dot{x} = Ax + Bh_{ps} \quad [3.\ 48]$$

Where:

$$x = [z_s, \dot{z}_s, z_u, \dot{z}_u]^T \quad [3.49]$$

$$A = \begin{bmatrix} 1 & 0 & 0 & 0 \\ -k_2 & -c & -k_2 & -c \\ 0 & 0 & 1 & 0 \\ \frac{k_2}{\mu} & \frac{c}{\mu} & -\frac{k_1+k_2}{\mu} & -\frac{c}{\mu} \end{bmatrix} \quad [3.50]$$

$$B = \left[0, 0, 0, \frac{k_1}{\mu} \right]^T \quad [3.51]$$

Where h_{ps} is smoothed profile elevation, x is the array of state variables (variables that, together, completed describe state of simulated system).

Time is related to longitudinal distance by the simulated speed of the vehicle

$$t = x/V \quad [3.52]$$

Where x is longitudinal distance and V is the simulated forward speed, and for the IRI is defined as 80 km/h. The units of V should be length/second, where the units of length match those of x.

The IRI is an accumulation of the simulated motion between the sprung and unsprung masses in the quarter-car model, normalized by the length L of the profile:

$$IRI = \frac{1}{L}\int_0^{L/V} |\dot{z}_s - \dot{z}_u| dt \quad [3.53]$$

The vehicle response variables oscillate about and have 0 as an average value. Hence, the absolute value in Eq. 3.53 is necessary to obtain a non-zero average.

Equations 3.46 to 3.53 define the IRI. The quarter-car dynamics and the initialization method together make up a type of calculation whose generic name is the “initial value problem”, or “integration or ordinary differential equations” (Sayers, 1995). There are several methods for solving these equations. Sayers (1995) commented that, although the numerical integration is the most common way to solve ordinary differential equations, such as the Euler integration, the simplest one, is not recommendable unless the interval between profiles is sufficiently small. It showed that the output of the algorithm is in error even for samples spaced 10 mm. On the contrary, Sayers (1995) recommends the state transition matrix method. The response of the recommended algorithm for various sample intervals is shown in Fig 3.92., which is less sensitive to changes in sample intervals than the Euler integration method (Sayers, 1995).

3.4.6.1. Remarks about IRI

At present, the International Roughness Index is the most used index for road roughness evaluation around the world. It the calculation, the measured profile is processed by means of a mathematical transform which filters and cumulates the wavelengths encountered in the profile. This transform was produced and calibrated to mensures that the IRI value is closely related with the users’ perception of roughness and the tyre load dynamics, which have impact on vehicle control and safety. It needs a profile and then processed with an algorithm, which simulates the physical properties and displacement of a vehicle wheel and suspension system, when moving at 80 km/h. Therefore, essentially, IRI is calculated by a mathematical simulation of the physical response of a typical vehicle to a road profile. The IRI calculation procedure reproduces the physical processing and filtering of a measurement vehicle to produce a simulated Average Rectified Slope (ARS) value. Nevertheless, as the IRI calculation uses a computer algorithm that remains constant over time, unlike to RTRRMS devices, it does not depend on the damping and stiffness properties of the measurement vehicle, which are sure to change over time, and also from on vehicle to another. Consequently, the IRI value is stable with time, a very valuable advantage.

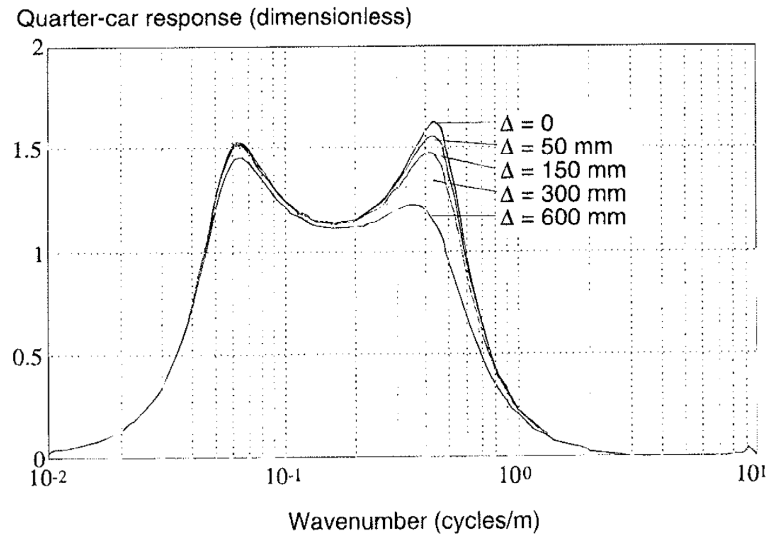


Fig. 3.79. Response of recommended algorithm (Sayers, 1995).

Fig. 3.80 shows the approximate and usual ranges of IRI roughness on different road types.

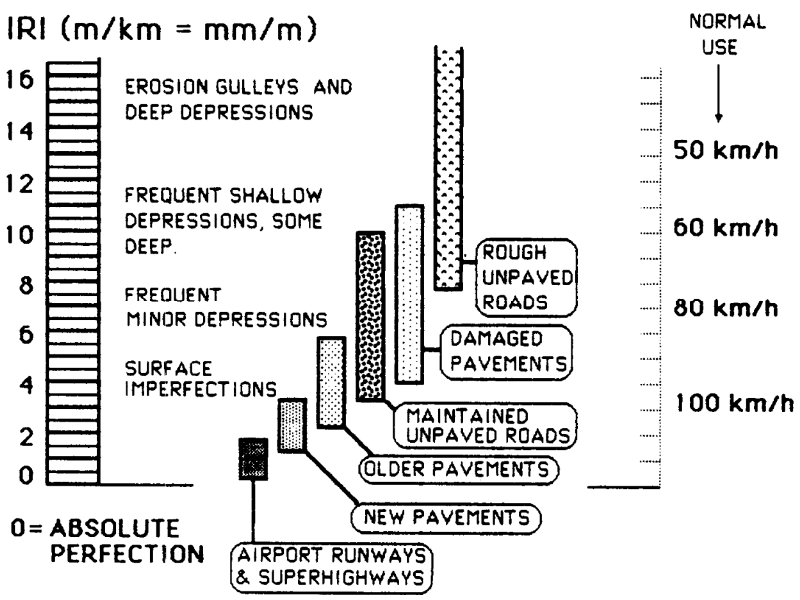


Fig. 3.80. The IRI roughness scale (Sayers et al., 1986b).

Another key advantage is that IRI is reproducible, i.e., it can be measured by means of different types of profiling devices, as far as the devices measures the profile accurately. Since the IRI calculation simulates the displacement of one wheel, a quarter of a typical car, the computation model is called “quarter car model”.

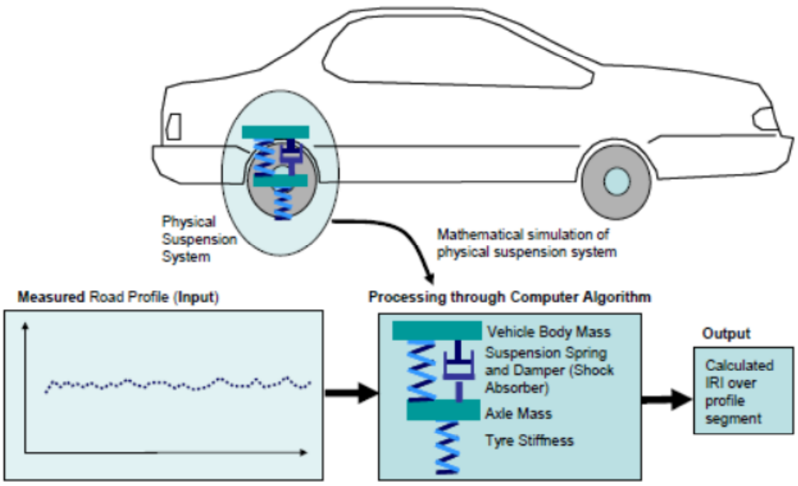


Fig. 3.81. Aspects of the IRI calculation (Sayers and Karamihas, 1998)

It must be underlined that the IRI algorithm effectively filters the raw roughness data and in the process highlights the roughness elements that impact most on the users’ perception of the roughness. With that aim, the IRI algorithm eliminates all wavelengths components that do not contribute to the roughness experienced by road users at speeds close to 80 km/h., filtering those wavelengths that are fall outside the 1,3 m to 30 m wavelength band (wave numbers between 0,03 and 0,77), as seen in Fig. 3.79

Since the IRI algorithm filters out wavelength components outside 1,3 to 30, IRI is not recommended to be interpreted for section shorter than 30 m. IRI is recommended to be averaged over 100 m sections.

3.4.6.2. Classification of Roughness measuring devices to calculate IRI.

As stated, one of the most important properties of the IRI is that it can be calculated by any roughness measuring devices, which have given its international recognition. Therefore, Sayers *et al.* (1986b) grouped the roughness measuring devices into four generic classes, according to the accuracy associated with them and the calibration requirements to determine the IRI. This classification is also established by the ASTM E 950 standard (ASTM, 2018b).

3.4.6.2.1. Class I: Precision profiles

This class is the highest accuracy standards for measurement of IRI. The longitudinal profile of the wheel track is required to be measured as a series of accurate elevation points closely-spaced along the travelled wheelpath. The distance is suggested to be not more than 0,25 m, i.e, 4 measures per metre and the precision in the elevation must be 0,5 mm for very smooth pavements. Devices in this class is grouped in two groups:

- Manually operated equipment: such rod and level, Dipstick and walking profiler
- Laser based technology, such as inertial profilers.

Methods in this class are those than can conduct measures of such high quality that reproducibility of the IRI numeric could not be improved.

3.4.6.2.2. Class II: Other profilometric methods

This class include dynamic profile measuring methods that measure the profile elevations as the basis for direct computation of the IRI. The profile of one or both wheelpaths is measured with contact or non contact profilometers, but the accuracy required for Class I is not achieved, being less accurate than Class I devices. Highspeed profilers are included in this group.

High-speed profilers can be classified as Class I and II according to the two primary parameters involved in profile measurement: sample interval and precision of the elevation measurement (Table 3.9)

Table 3.9. Accuracy requirements for Class I and II profilometric measurement of IRI (Sayers et al. 1986b).

Roughness range IRI (m/km)	**Maximum convenient sample intervals between points (mm)**		**Precision of elevation measures (mm)**	
	Class I	**Class II**	**Class I**	**Class II**
1,0 – 3,0	250	500	0,5	1,0
3,0 – 5,0	250	500	1,0	1,5
5,0 – 7,0	250	500	1,5	2,5
7,0 – 10,0	250	500	2,0	4,0
10,0 – 20,0	250	500	3,0	6,0

3.4.6.2.3 Class III: IRI estimates from correlation equations

Class III equipment include mechanical or electronic devices that indirectly evaluate pavement profiles. It includes the Response-type road roughness measuring systems (RTRRMS), which measure the dynamic response of the vehicle to the road, ether mechanically or by using accelerometers. The RTRRMS measure is dependent on the dynamics of a vehicle, and it need to be scaled to yield roughness properties comparable to IRI. The dynamic properties are unique for each RTRRMS vehicle, and, additionally, change with time, Therefore, the “raw” measures must be corrected to the IRI scale using a calibration equation that is obtained experimentally for each devices. Furthermore, as the dynamics of the vehicle change easily, very rigorous maintenance and operating procedures must be followed. A control testing has to be carried before normal operations. If changes appear, the device must be recalibrated. Accelerometer based systems (e.g., Roadmaster, ARRB Roughometer) are easier to calibrate, but they do not give as accurate results as a well calibrated bump integrator (e.g., CSIR LDI, ROMDAS, TRL Bump Integrator)

In this class other devices that can generate a roughness numeric reasonably correlated to the IRI are included, such as rolling straightedge. Obtained measuring data can be employed to estimate IRI by means of regression equations, after a correlation experiment.

3.4.6.2.4. Class IV: Subjective ratings and uncalibrated measures

This group is the least accurate. It included subjective evaluations of the roughness, which are conducted by either driving over the section or carrying out a visual inspection. Another possibility is to use the measurements from an uncalibrated instrument. Conversion of these observations to the IRI scale is limited to an approximate equivalence. Essentially, the estimates of equivalence can be considered as a “calibration by description". These types of subjective evaluations are conducted when higher accuracy is not necessary or is not affordable.

3.4.6.3. Other roughness indices based on the longitudinal profile

3.4.6.3.1. Half Car Index (HRI)

The Half Car Index (HRI) is an index that uses the same processing algorithm as the IRI, but instead of employun the left or right wheelpath profile, the HRI uses both.

As exposed, the IRI calculation represents the motion of one quarter of a normal passenger car and is measured from a profile of a single wheelpath. Modern profilometers can measure the longitudinal profile in both wheelpaths simultaneously. Theoretically, a more accurate evaluation of road roughness can be achieved if the roughness index is obtained from both wheelpaths, if compared to calculate from a single wheelpath, because the overall vehicle response is determined by the profile input obtained from both left and right wheelpaths at the same time. The calculation consists of taking point-by-point average of the two profiles first (one from the left wheelpath and one from the right wheelpath), and the process the averaged profile with a quarter-car filter (Fig. 3.82).

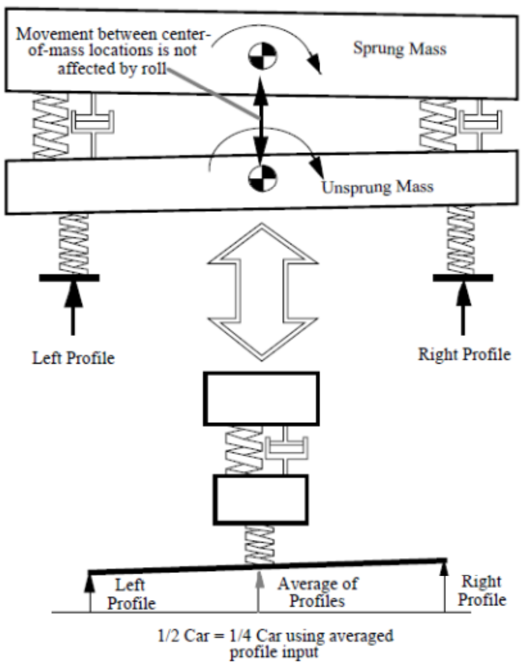


Fig. 3.82. Half-car model (Sayers and Karamihas, 1998)

The advantage of the half-car analysis is that it matches the way road meters are installed in passenger cars more closely (Sayers and Karamihas, 1996a). If the right and left profile receive the same sinusoid, in phase, the whole vehicle bounces in response. It does not roll at all. Nevertheless, if they are out of phase, the vehicle rolls but does not bounce. With the road meter installed at the centre of the axle, it sense bounces by not roll. In real road, there is a mixture of bounce and roll. The bounce part gets through, but the roll part does not. Therefore, the roughness as calculated with an HRI analysis is less than or equal to the result obtained from the IRI analysis. A disadvantage of the HRI is that it requires the two profiles to be perfectly synchronized before averaging.

Sayers (1989) analyzed both approaches, half-car and quarter-car models and showed that correlation between HRI and IRI were very high (Fig. 3.83) and, hence, no additional information is provided by HRI. As not all the devices can measure both profiles simultaneously, IRI was selected for international index .

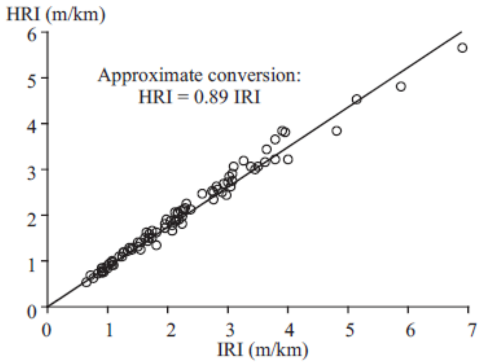


Fig. 3.83. Correlation between IRI and HRI (Sayers, 1989).

3.4.6.3.2. Profile Index

As commented previously, the Ride Number (RN) was defined as an exponential transform of an RMS slope statistic called PI with dimensionless units of slope (m/m), according to equation 3.35. If two profiles are processed, PI is the RMS value of the PI for the left and right profile:

$$PI = \sqrt{\frac{PI_L^2 + PI_R^2}{2}} \qquad [3.54]$$

However, if only a single profile is measured, its PI is used in equation 3.35. PI for an individual profiles is calculated using a modified version of the IRI algorithm. The differences between the calculation of PI and IRI are (Sayers and Karamihas, 1996b):

- The coefficients of the IRI are replaced with the following: k_1 = 5120, k_2 = 390, c = 17, μ = 0,036
- The initialization length is changed from 11,0 m for IRI to 19,0 m for PI
- The accumulation is done by RMS, rather than mean absolute.

The sensitivity of the PI filter to wave number can be observed in Fig. 3.84. The profile index (PI) was defined for several combinations of RMS (root-mean-square) values.

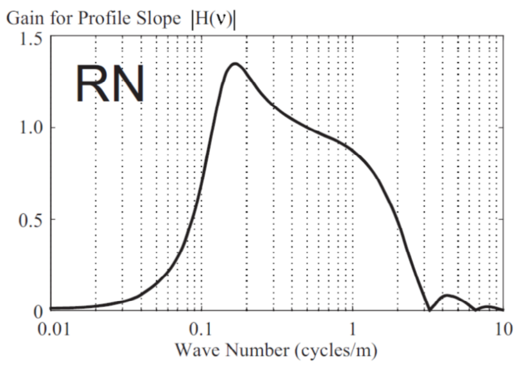


Fig. 3.84. Wave-number response of the quarter-car filter for PI (Sayers and Karamihas, 1998).

3.5. Structural condition

3.5.1. Introduction

The structural evaluation of a pavement is employed to estimate the structural adequacy of an existing pavement to precise if the pavement is able to meet projected traffic loadings. Moreover, when the pavement is tested not to be adequate to meet future loadings, analysis results can give an idea of the appropriate treatment (AASHTO, 2012).

The structural evaluation is normally performed with a high level of detail and it is an usual part of investigations at project-level, where it can be combined with more extensive pavement condition surveys, including destructive (coring and boring) and non-destructive testing (deflection). Despite the fact that structural condition evaluation is generally performed at project-level, some road agencies also employs structural evaluation obtained from non-destructive tests, generally, deflection testing, at network level to evaluate the road condition and to establish the required maintenance or rehabilitation work.

The most commonly used indicator of the strength of a pavement structure is the pavement deflection, where a load is applied to the pavement surface and the resulting deflection is measured. In other words, pavement deflection can be defined as the vertical measure due to an applied load on the pavement surface, measured at the surface (PIARC, 2016). Deflection measurement techniques, which are non-destructive testing, are more employed than destructive methods for assessing strength, because of the following reasons:

- Lower costs of the non-destructive deflection method
- The necessity of lane closure and traffic control during testing is reduced
- It does not suppose any damage to the pavements, as it not disturb the underlying pavement structure
- It does not require removing any pavement materials for laboratory testing.

Consequently, this method allows carrying out more tests than by means of destructive testing.

3.5.2. Historical overview

There have been a great variety of pavement deflection measurement methods in the last decades. In the 1930s, a loaded plate was employed for static measurements of bearing capacity for the design of building foundations, applying basic soil mechanics principles. This idea was carried to the pavement engineering, aiming to develop a rationale for the carrying (bearing) capacity of the pavement structure. These initial methods can be regarded as first generation techniques.

The next progression was the very slow-moving load deflection measurement, initiated with the development of the Benkelman Beam (Fig. 3.85a), used at the WASHO (Western Association of State Highway Organizations) Road Test in the early 1950s in the USA. It represents the second generation.

Later, the next development of the Benkelman Beam was the Lacroix Deflectograph (Fig. 3.85b), manufactured in 1956 at the Laboratoire Central des Pont et Chaussées (LCPC) in France, and the British Deflectograph, by the TRRL in England in 1970. It is able to measure the amplitude of the deflection basin at a relatively low speed. They are referred as the third generation.

With the aim of measuring the profile of the deflection basin, the fourth generation used a dynamic vibratory load, and the Dynaflect (Fig. 3.85c, d) and Road Rater were produced in the early 1970s in the USA. These techniques were more mobile than static plate loading test and allow to measure deflection as a routine task.

The subsequent step is the Falling Weight Deflectometer (FWD) (Fig. 3.85e), first demonstrated at the LCPC in the mid 1960s, and commercially available from 1975, which applies an impulse loading, with closely replicates the load of a moving truck tyre. By means of series of geophones spaced a different distances from the load, the exact shape of the deflection bowl is possible to obtain. It is categorized as the fifth generation.

However, the main disadvantage is that the FWD needs to be stationary to measure deflection, which implies lane closure. Therefore, the Rolling Wheel Deflectometer (RWD) (Fig. 3.85f) has been developed, which allow measuring the deflection basin at traffic speed, making possible to employ at network-level. It is regarded as the sixth generation. The Curviameter, commonly deployed in Europe, is classified as an intermediate point between FWD and RWD, measuring deflection bowls at 8 km/h. Further description of mentioned devices can be found in PIARC (2016), Pearson (2011).

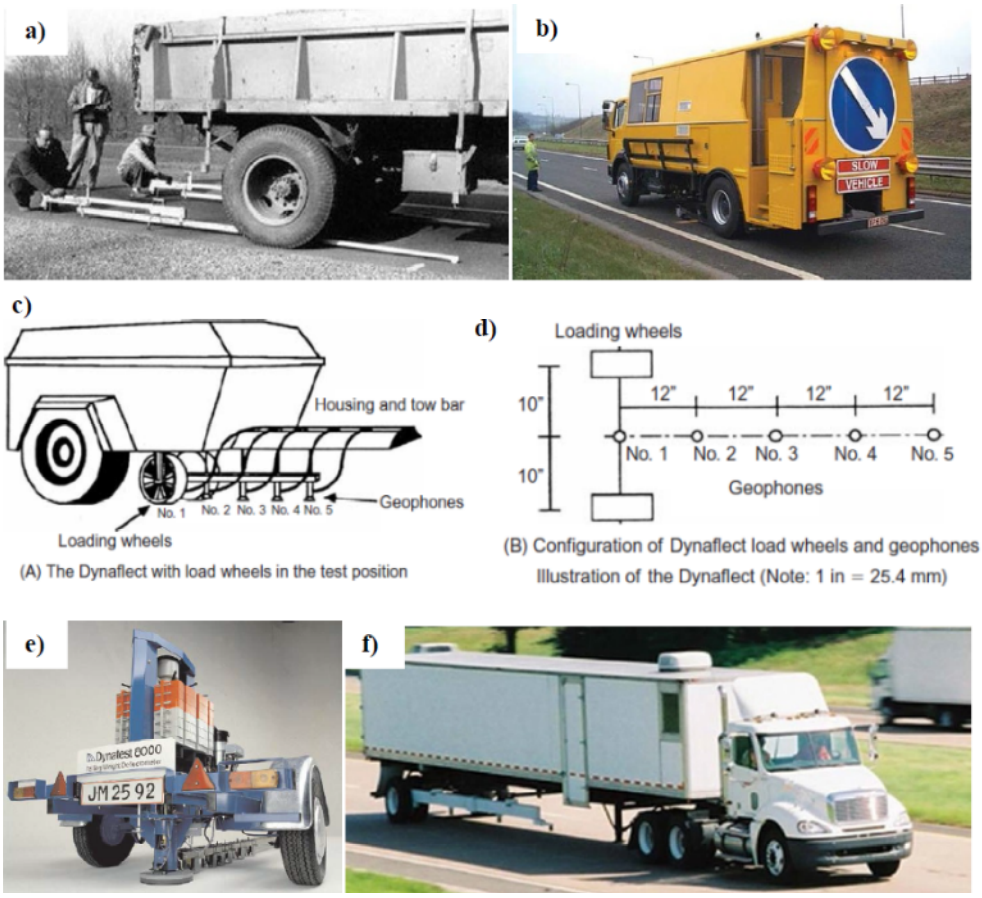


Fig. 3.85. Pavement deflection measuring devices a) Benkelman beam (PIARC, 2016), b) Deflectograph in use (PIARC, 2016), c) The Dynaflect with load wheels in the test position (Haas et al., 1994), d) Configuration of Dynaflect load wheels and geophones (Haas et al., 1994), e) Falling Weight Deflectometer (Pearson, 2011), e) Rolling Wheel Deflectometer of the FHWA (PIARC, 2016).

3.5.3. Structural capacity indicators

The most used pavement deflection indicator, and also the one that has been traditionally employed, is the maximum deflection response to the applied load. It can be utilized in several forms (PIARC, 2016):

- As an indicator of variability in construction quality
- As a specification limit in construction works
- As a parameter to suggest the selection of adequate rehabilitation treatments
- As an input into structural overlay design procedures
- As an estimator of change of structural condition with time.

Other parameters, generally related to the deflection bowl shape, are employed for additional information. The curvature parameter, $D_0 – D_{300}$, commonly referred as SCI300, is employed. In some countries, where thin asphalt layers are extended, the curvature function is defined as D0 – D200. The radius of curvature of the deflection bowl is also employed.

Full deflections basin data can be deployed for back-calculation processes to estimate the properties of the existing materials in the section, as far as thickness is a known parameter. With this aim, response-to-load (usually layered linear-elastic) pavement calculations are carried out using the modulus properties of the existing pavement layer adjusted, in an iterative way, searching to calculate a deflection bowl that best matches obtained data.

Deflection data is also employed as predictor of other pavement structural indices. Among them, the structural number of the pavement is the most commonly deployed. A variety of correlations between structural number and deflection results have been proposed for different types of pavements and structural capacity measuring devices (Morosiuk *et al*., 2004). Moreover, COST 354 document (Litzka *et al.*, 2008) suggests that the residual service life may be employed as an indicator of the pavement structural condition.

3.6. Pavement distress measurement

3.6.1. Introduction

The signs of that deterioration can be seen in the form of pavement distresses. Visible distresses can be observed and quantified by visual examination of the pavement surface. Generally, these distresses are measured and catalogued as a part of the pavement condition data collection in PMS. Results are usually classified as a function of the distress type, severity and extent. Visible distresses have been recorded for decades in most of the countries, being habitual in any pavement management practice as they can be carried out with a minimum investment in tools and technology.

Every country and even every road administration has its own procedure for cataloguing and measuring the different distress types and, hence, it can be said that there is not a universal procedure for distress identification and characterization. Two of the most employed documents for distress classification come from the USA and are the following:

- The *Distress Identification Manual for the Long-term Pavement Performance Program* (FHWA, 2003). It was first published in the late 1980s. It was aimed to provide assistance for researchers about collecting pavement performance data in a consistent, repeatable way that was independent of the collector and even of the collection method. Every distress type is defined in the manual and illustrations with graphics and figures were included to “foster more uniform and consistent definitions of pavement distress” (FHWA, 2003). Pavement distresses identified in this document are shown in Table 3.10.
- The *ASTM D6433 Standard Practice for Roads and Parking Lots Pavement Condition Index Survey* (ASTM, 2016). Six versions of this document have been published, in 1999, 2003, 2007, 2009 and 2011, and the last one in 2016. It defines the distresses as a part of a standardized pavement evaluation process to calculate the Pavement Condition Index (PCI), which further described in subsection 3.6.4. Distresses that are introduced in the PCI calculation are summarized in Table 3.11.

Table 3.10. Pavement distress in bituminous layers identified in FHWA (2003)

Bituminous roads				
Cracking	**Patching and potholes**	**Surface deformation**	**Surface defects**	**Miscellaneous distresses**
1. Fatigue cracking	7. Patch/patch deterioration	9. Rutting	11. Bleeding	14. Lane-to-shoulder dropoff
2. Block cracking	8. Potholes	10. Shoving	12. Polished aggregate	15. Water bleeding and pumping
3. Edge cracking			13. Raveling	
4. Longitudinal cracking				
5. Reflection cracking at joints				
6. Transverse cracking				

Table 3.11. Pavement distresses in bituminous layers identified in ASTM D6433 (ASTM, 2016).

Distresses in bituminous roads	
Alligator cracking (fatigue)	Polished aggregate
Bleeding	Potholes
Block cracking	Railroad crossing
Bumps and sags	Rutting
Corrugation	Shoving
Edge cracking	Slippage cracking
Joint reflection cracking	Swell
Lane/shoulder drop-off	Weathering and ravelling
Longitudinal and transverse cracking	Ride quality, a separate “distress”, is actually an input in determining the severity level of bumps, corrugation, railroad crossing, shoving and swells
Patching and utility cut patching	

With regard to Spain, the Spanish Ministry of Public Work Spanish (previously *Ministerio de Obras Públicas*, at present Ministerio de Fomento) has also published guides to identify and classify visible distresses on pavement. The first one was the “*Catalogo de deterioros en firmes*” [Pavement distress catalogue] (Ministerio de Obras Públicas y Urbanismo, 1989c), where all the possible distresses that could be found in roads were listed and described individually, showing a picture example, how to quantify them and giving an example and the possible causes for this failure. Later, in another document, called Guía para la actualización del inventario de firme de la Red de Carreteras del Estado" [Guide for updating the data inventory of the pavements of the Road Network of the State] (Ministerio de Fomento, 2011a), a distress classification list is provided (Table 3.12), which is totally based on the distress list provided in the previous document.

Table 3.12. Distress classification according to Ministerio de Obras Públicas y Urbanismo (1989c) and Ministerio de Fomento (2011a)

Distress in bituminous pavements			
Deformations	**Cracking**	**Loss of surface integrity**	**Bleeding**
Rutting [*rodera*]	Central longitudinal cracking [*fisura long central*]	[*firme brillante*]	Flushing [*exudación*]
Depressions [*hundimiento*]	Lateral longitudinal cracking [*fisura long. lateral*]	Raveling [*descarnadura*]	Bitumen bleeding [*flujo de ligante*]
Localized depression [*blandón*]	Transverse cracking [*fisura transversal*]	Polished aggregates / polishing [*áridos pulimentados*]	Spring under pavement [*mancha de humedad*]
Longitudinal shoving [*cordon longitudinal*]	Random cracking [*fisura errática*]	Delamination [*peladura*]	Mud pumping [*ascensión de finos*]
Transverse shoving [*arrollamiento transversal*]	Crescent shaped cracking [*fisura parabolica*]	Pothole [*bache*]	
Corrugations [*firme ondulado*]	Crocodile cracking [*piel de cocodrilo*]	Coating failure [*fallo envuelta*]	
Individual corrugation [*ondulación*]	Block cracking [*cuarteo de malla gorda*]	Stripping [*desintegración*]	
Pavement print / non-structural rutting [*huella*]	Cracking in wheelpath [*fisuras en rodada*]	Stripping in lines[*estriado*]	
Localized creep [*protuberancia*]	Edge cracking [*fisuras en borde de calzada*]		
Blistering (Bubbling) [*burbuja*]	Reflection cracking [*fisura reflejada*]		
	Meandering crack [*fisura curva*]		
	Roller cracks [*fisuras finas*]		

Note: Spanish denomination of distresses are also displayed

As seen, there is no universally employed or unique approach to distress data collection. As an example, the highway Agencies of the states of the USA reported the distress data that each one collected (Flintsch and McGhee, 2009). Rutting was the only distress that was collected by all agencies (Fig. 3.86).

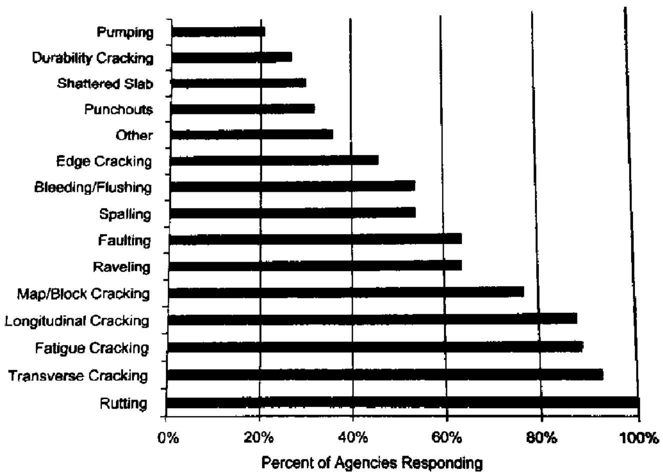


Fig. 3.86. Distress data collection. Percentage of Highway Agencies of the states of the USA that collect each distress data from their road network (Flintsch and McGhee, 2009).

The distresses whose data are the most collected worldwide are defined. The following distress classification groups are considered: cracking, deformation, surface defects. Fig. 3.87 shows the asphalt pavement generic distress pattern (Pearson, 2011).

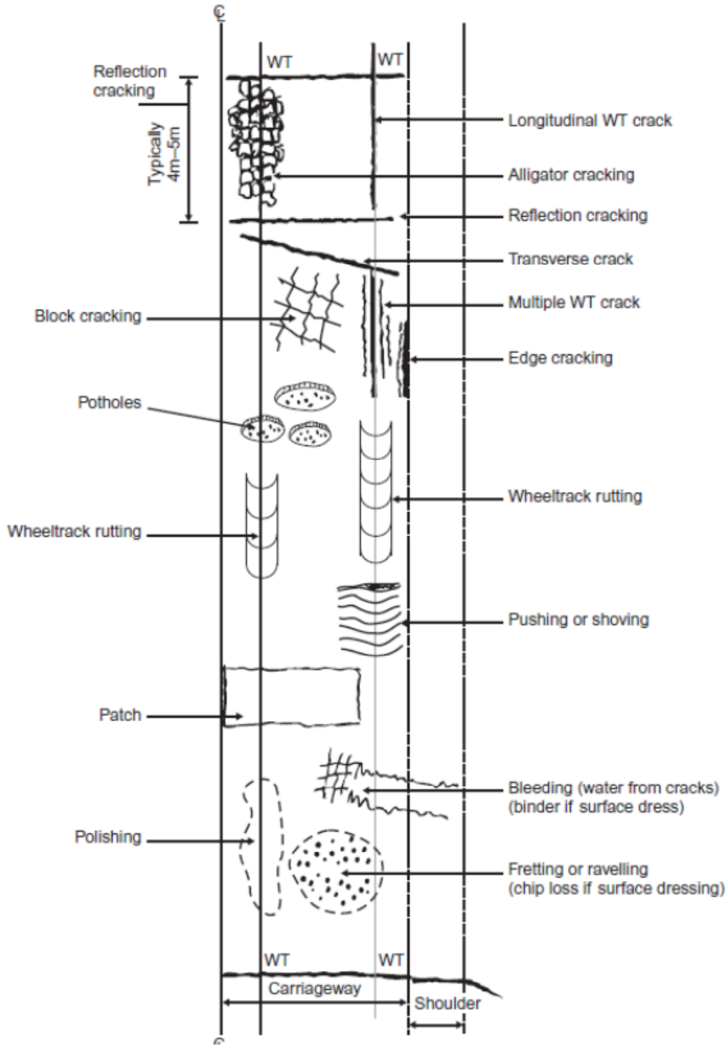


Fig. 3.87. Asphalt pavement generic distress patterns (Pearson, 2011).

3.6.2. Pavement distresses

3.6.2.1. Cracking

A surface crack is a narrow width fracture or discontinuity in the pavement surface material (PIARC, 2016). In flexible pavements it can result from a number of causes and has many detrimental effects. There are several types of cracking (Fig. 3.88):

- Longitudinal cracks usually appear parallel to the pavement centerline or in the direction of traffic, and also in mid paving lane
- Transverse cracks usually appear perpendicular to the pavement centerline and in the direction of traffic.
- Block cracking is characterised by interconnected cracks that forms a block pattern, approximately rectangular in shape, generally extended over the full pavement. Usually, the block range is from

0,25 m by 0,5 to 3 m by 3 m.

- Crocodile cracking is characterized by interconnected cracks that form a series of small, approximately straight-side polygons.
- Crescent shaped cracks generally appear on pavements as a set of closely spaced parallel cracks along with shoving, normally in the direction of traffic.
- Diagonal cracks are single cracks that form diagonally across the pavement.
- Meandering cracks are unconnected irregular single cracks that run in any direction.

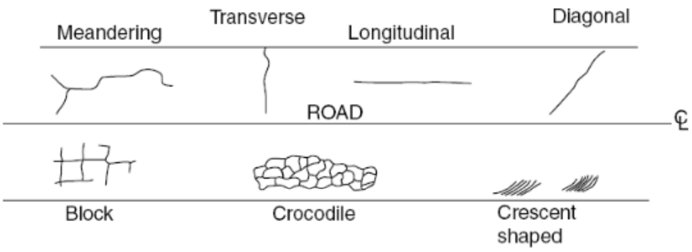


Fig. 3.88. Sketch of common types of cracks in flexible pavements (Austroads, 1987).

Crack detection is usually carried out by trained personnel using the image rating method, or by using automatic software processing of recorded images. At present, laser and line-scan cameras are employed instead of traditional photographic or video images as source data to automatic crack detection processing software. There is a wide variety of indicator of cracking distresses. One index is the ratio of the total cumulated lengths/areas of all the cracks measured and the corresponding measurement surface area. Another indicator that is commonly employed is the type/severity/extent approach, shown in Table 3.13.

Table 3.13. Example of type/severity/extent cracking indicators

Cracking type	**Severity categories**		**Extent categories**
	Name	**Width range**	
Longitudinal	Fine	≤ 1 mm	< 1 %
Transverse	Medium	1 mm – 3 mm	1 % - 5 %
Block	Wide	> 3 mm	5 % - 10 %
Crocodile	Spalled	> 3 mm and spalled	10 % - 25 %
Irregular			> 25 %

3.6.2.2. Deformation

Deformation of a pavement is the change in the pavement surface profile. It can affect roughness condition and skid resistance with water pounds. Deformation can accelerate crack initiation. The most common flexible pavement deformations are (Fig. 3.89):

- Rutting is the longitudinal depression occurring in the wheel path as a consequence of inadequate surface thickness and lack of compaction or stability in the surface or base course
- Corrugations in flexible pavements are closely and regularly spaced transverse undulations
- Depressions are localized bowl shaped settlement in the pavement.
- Shoving or creep is the horizontal displacement of surfacing material occurring generally in the

direction of traffic where braking or acceleration takes place

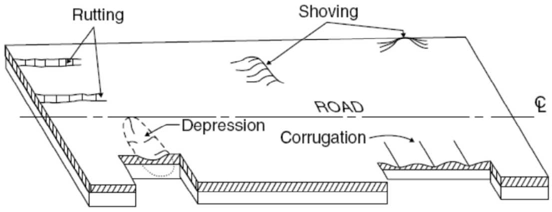


Fig. 3.89. Sketch of deformation in flexible pavements (Austroads, 1987)

3.6.2.3. Surface defects

Surface defects appear in the surface layer and they are not related to structural layers in the pavement layers, but they cause important effect on pavement serviceability and skid resistance. Common types are described in the following subsections (Fig. 3.90):

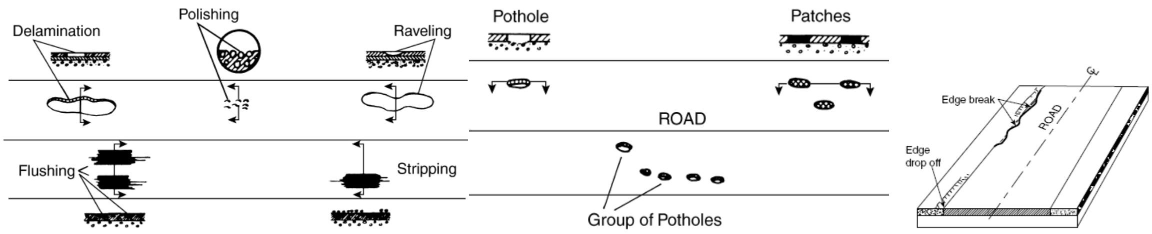


Fig. 3.90. Sketch of surface defects in flexible pavement. (Austroad, 1987).

- Delamination occurs when surface layer debonds from the underlying layers and, hence part of the surface layer, of uniform thickness, is lost.
- Bleeding is the consequence of the movement of bituminous binder to the surface resulting in a reduction in surface macrotexture (
- Polishing is the decrease of the microtexture of surface aggregates due to traffic (
- Raveling occurs when both aggregate and binder are progressively lost from pavement surface.
- Stripping is related to sprayed seal pavements. It occurs where only coarse aggregate is lost leaving the binder in good condition. It
- A pothole is a localised deterioration of the pavement surface which results from breakdown and loss of the surface material, in a relatively short period of time, resulting in a steep depression.
- Patching, the localised repair of previously damaged pavement, can be also considered as a distress on a pavement. It corresponds to an area where a local repairing work has been performed on the existing pavement
- Edge break appears when the edge of surface material is fretted, broken or irregular
- The edge drop-off is the difference between the elevation of pavement surface and shoulder.

3.6.3. Surface distress data collection

Surface data has traditionally been collected manually, by trained official who carried a visual assessment.

The surveys can be carried out in three ways (PIARC, 2016):

- Manual surveys. Rating is carried out manually on foot. A technician records surface distresses in detail for a small length of pavement. This technique is very accurate, but, at the same time, is slow and requires closing of the road to provide a safe access to the observer (Fig. Manual survey). Observed defects are rated by a pre-defined set of criteria using paper forms or a computer.
- Windscreen rating. Pavement distresses are observed while travelling over the entire length of pavement, or an entire network of pavement; generally at a lower speed than normal traffic. Similar to manual rating, defects are rated by means of a pre-defined set of criteria using paper forms or a computer.
- Automated surveys. Surveys are conducted using vans fitted with specialized equipment.. Distress observations are made using images or videos that are collected by high resolution cameras mounted on a special vehicle moving at traffic speeds, over the entire length of pavement or network. Analysis can be made while travelling on the vehicle or at the office visualizing previously recorded images. Rating of observed defects using a pre-defined set of criteria are recorded by a trained technician using software, generally the same computer software that is employed to display the images/videos to the technician who rates the defects. This software generally allows the evaluator to detect the defects and to estimate the extent and the severity of them. Compared to the previously exposed techniques, this one is safer for the manual observer and is less disruptive to the traffic than manual and windscreen surveys, which are important advantages. The technician can carry this task in an office. The software allows pausing images and examining them in detail, providing a more optimised environment for this task than a moving vehicle. The quality of assessment depends on the quality of the collected images, the software and the experience of the technician evaluating.

Generally, surveys conducted as technicians walk along the pavement and those carried out viewing through the windshield of a vehicle are considered manual surveys. These types of surveys are still in use; road agencies combine manual and automated data collection techniques, since some information (roughness and surface texture information) can be collected by means of automated equipment. Manual surveys are usually employed on low-volume roads, where traffic volumes do not represent a substantial hazard to the survey crew (AASHTO, 2012). Manual surveys imply some advantages: very detailed distress type, severity and quantity information can be collected and assets outside the mainline of the roadway can be evaluated. If distresses are measured, the survey becomes objective. Additionally, manual surveys do not require any specialized device and hence, it can be conducted in undeveloped or in developing countries. On the contrary, there are disadvantages: they are slow and fairly labour intensive, hazardous if carried out walking along the road and training programmes for technicians are necessary for avoiding variability in the data.

In order to reduce the potential safety hazard of manual survey, many road agencies began to use automated survey, allowing capturing pavement images and sensor data. At present, automated data collection devices are available with fully integrated systems that enable a variety of data to be collected in a single pass of pavement. The accuracy and reliability of the distress data from automated surveys depends on the quality of the images captured in the field, with high improvements in camera resolution and elimination of distortions associated with lighting and shadows (AASHTO, 2012).

3.6.4. Distress condition indices

As seen in previous parts of the section 3.7. Pavement distress measurement, distress surveys are conducted to evaluate the degree of physical pavement deterioration, which is usually a function of the type of distress, the severity of distress and the extent of distress (reflected by the amount or density of distress) . Each of these three aforementioned characteristics of pavement distress have an important influence on the determination of the global pavement deterioration. As there are many types of distresses (some of them presented in section 3.6.2., the most important ones), and a wide variety of ways to describe severity levels and extent measurement, it is essential to employ standard procedures for distress identification and measurement of extent at each severity level. In order to get a practical and meaningful performance assessment of a network, most distress data are usually combined into a global condition index. The first approach was proposed in the Washington State PMS. Firstly, it implied to deduct values for each measured distress from a perfect score of 100, which represents a pavement in excellent condition with no distress. Secondly, the riding quality was calculated in a similar way, and finally, combining both values the Pavement Condition Rating (PCR) was obtained (LeClerc and Nelson, 1982).

The approach of PCR was further developed by the U.S. Army Corps of Engineers in PAVER distress survey procedure (Shahin and Kohn, 1979) and documented in ASTM standard D6433 (with several revisions) (ASTM, 2016). It combines the effect of various distress types and measurements of distress severity and extent into a single index, PCI, which assess the global pavement condition of the surveyed section. PCI also varies from 0 to 100. A value of 100 means that the pavement is in excellent condition and zero implies a failed pavement (Shahin and Walther, 1990).

3.7. Composite indices

In this chapter, the main characteristics of the pavements are commented and the indicators and indices developed to quantity those characteristics were commented. Additionally, apart from discussed indices, there is another type of indices to rate pavement condition: the composite indices or combined indices. The composite indices can be defined as the aggregation of multiple types of condition data into a single index that is representative of the overall condition of a pavement (AASHTO, 2012). The main reason for composite indices is that they are useful to communicate information about a road to senior administrators, elected officials, and to general public (Haas *et al.*, 1994). Although civil engineers and people who deal with pavement management are used to the variety of indices available, combined indices are employed at a level where administrative and political decisions are taken. There is no single or mechanistic formula to establish a combined pavement quality index. A first classification is to divide the proposed composite indices in subjective and objective.

The simplest approach to develop a combined index is to define a numerical scale and to assign descriptors to each of the scores. One example is the Pavement Surface Evaluation Ration (PASER) condition survey developed by the University of Wisconsin (Wisconsin Transportation Information Center, 2002). It is a subjective indicator due to the degree of interpretation that is needed to assign a score to a pavement section.

A usual methodology for objective composite indices consists of subtracting points from the index associated with a perfect score, according to the distress type, severity and extension observed in the field. This type of approach usually subtracts more points for what is considered more substantial distress than for less important distress. Hence, the calculation depends on the types of distress and the relative importance that each road agency gives to the combination of distress type and severity. The most known and used index within this category is the Pavement Condition Index (PCI) procedure, standardized in ASTM D6433-18 (ASTM, 2018a). Generally, combined indices consider two or more pavement characteristics and represent an overall aggregation of the different measures of pavement condition, following the form of the Eq. 3.59.

$$Combined\ Index = W_1 \cdot C_1 + W_2 \cdot C_2 + W_3 \cdot C_3 + W_4 \cdot C_4 \qquad [3.55]$$

Where *General Index* is the proposed global index, W_i is the weighting factor for pavement condition measure i, and C_i is the value for pavement condition measure i.

With regard to composite indices, the COST Action 354 must be commented. After an extensive literature research and the analysis of questionnaires sent to road agencies in the participating countries, a set of performance indicators was developed (Litzka *et al*., 2008). COST Action 354 defines a performance indicator as a superior term of a technical road pavement characteristic, which can classify its condition, and classified indicators in 4 categories: single, pre-combined, combined or general.

A “single” indicator relates to one technical characteristic of the road pavement; a “pre-combined” indicator relates to two or more similar characteristics (e.g. alligator cracking and isolated cracking”), a “combined” indicator relates to two or more different characteristics of the road pavement, indicating the condition of the characteristics involved. Finally, a “general” indicator” results from the combination of indicators that describe the pavement condition concerning several aspects, such as, safety, comfort and structure. A complete overview of the performance indicators in the COST Action 354 for the standard level of application can be found in Marcelino *et al.*, 2018.

3.8. Conclusions of the chapter

As it has shown, there are several devices and indices for each pavement characteristic and property. With regard to a specific property, the International Roughness Index (IRI) is the only index that is widely employed around the world. Due to its time-stability, portability and reproducibility has extended to all the countries and all the road agencies.

The International Friction Index IFI was also an attempt to create an international indicator, which apart from friction, also introduces texture values. However, it has not achieved the same level of internationality as the IRI.

With regard to the other properties, the great variety of devices and indices has led not to use any of them universally.

Chapter 4. Pavement performance prediction models

4.1. Introduction

A pavement performance prediction models can be defined as "*a mathematical description of the expected value that a pavement attribute will take during a specified period of analysis*" (Hudson *et al*., 1979). As discussed in chapter 2, the pavement deterioration models play an essential role in the pavement management (Abaza *et al*., 2004; Jorge and Ferreira, 2012; Abaza, 2017a). First of all, they are used to predict future pavement conditions. Secondly, using the information about future state of the pavement, they can be employed to identify the most cost-effective treatment strategy for the pavement in the network, and even being able to determine the appropriate time (moment) to carry out the maintenance or rehabilitation works. Moreover, performance models can demonstrate the consequences of different pavement investment strategies. Lastly, as it is possible to forecast future condition it can estimate statewide pavement needs required to fulfil goals, objectives and constrains specified by the road agency. They also help to establish the performance criteria (thresholds) for performance specifications and warranty contracts (AASHTO, 2012).

Additionally, deterioration models provide feedback on the pavement design and on the effectiveness of different maintenance strategies. Due to the great importance of these models, their accuracy is a key factor for establishing the correct moment when the action is required, the level that is needed (total reconstruction, rehabilitation, maintenance), and regarded from a global point of view, for prediction the future condition of the entire road network. Hence, it is aimed to develop a very accurate performance model, reflecting the real evolution of assessed pavements in order not to mislead road agencies into estimating future condition or treatment needs. This accuracy depends on the quality of the pavement condition data used for the models.

Fig. 4.1 shows the deterioration evolution trends of different pavement performance indicators or indices, and the associated decisions criteria when a minimum acceptable limit or "failure" status is achieved, which indicates that an intervention is needed.

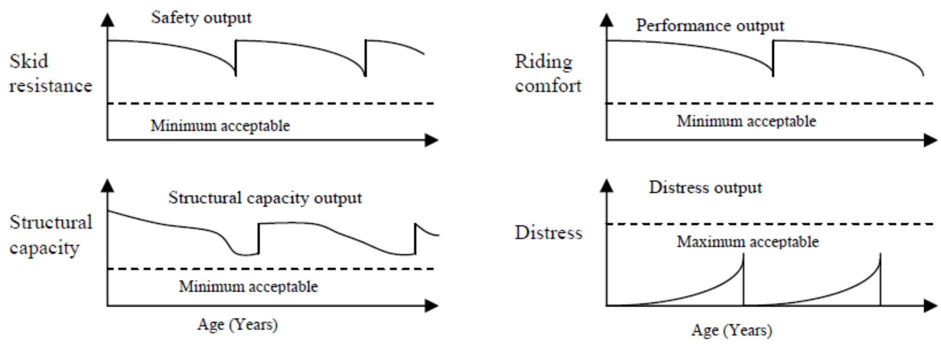


Fig. 4.1. Usual evolution of different pavement indices (adapted from Haas et al., 1994).

In the area of pavement management, there are various expressions to refer to the pavement performance prediction models, like deterioration models, prediction curves, pavement condition evolution models, pavement progression models, etc. These expressions are used in the present PhD thesis as synonymous.

This chapter presents the main categories of pavement deterioration models that are employed around the world. As there is a wide variety, they are classified according to different criteria. Furthermore, there is no universally employed approach to model pavement deterioration. Therefore, advantages and disadvantages of each model are discussed, underlining the circumstances that recommend the employment of each model. A model, which fits available data in a pavement management system, could be not applicable in another pavement management system under different circumstances.

4.2. Data requirements for pavement performance models

First of all, it must be reminded that performance modelling requires data to reliably develop the evolution models (Darter, 1980; Lytton, 1987). According to a literature review, the following factors must be taken into consideration in the process of choosing the model approach and when determining the availability of sufficient data for the development process (Darter, 1980):

a) Adequate source of data. Every performance model requires its own type of data and, hence, required data must be available before starting to develop the model. The availability of all of the variables that are introduced in the model must be checked for all the pavement sections included in the pavement management system. Moreover, these data must be continuous over time in order to produce models that forecast reasonable values.
b) The most significant variables that really have influence on the pavement performance must be considered. There are several variables than can influence the pavement performance over time, such as traffic volumes, climate factors, layer thickness and material properties. Although road agencies would like to introduce all the possible variables for modelling the deterioration, it is impractical to do so due to the lack of adequate records in the PMS database to support the use of multiple variables. If multiple variables are decided to be introduced in the modelling, statistics analysis can provide clues about the impact degree of each variable on the pavement deterioration.
c) A functional form that fits the data. Pavement deterioration models describe the pavement condition evolution over time, by means of some equations. The change ratio of the condition can be reflected with different forms (or shapes), according to the kind of equation deployed. The selected model should fir the data conveniently and should reflect the usual performance pattern of that kind of pavement.
d) Satisfaction of criteria for precision and accuracy. As stated, performance models are key items in PMS because forecasted future condition is based on their estimation. Consequently, models should predict condition changes over time reasonably. By means of statistical analysis, like the coefficient of determination (R^2), the accuracy and reliability of different models is assessed.

Lytton (1987) indicates the limitations of each model must be considered not to employ models out of their intended goal.

4.3. Predicted values of the models

The first step in pavement modelling is the selection of what the model is going to predict. Generally, pavement performance models employed in PMS forecast evolution of one of the following indices that represent pavement condition over time:

- Distress severity and extent. It usually includes evolution of the amount and severity of a particular distress, like fatigue cracking, rutting or faulting.
- Individual pavement condition indices. Models predict the changes in indices related to each of the characteristics usually measured in pavements. They most usual indices for each characteristic (surface properties, structural capacity, roughness, distresses, etc.) were exposed in Chapter 3.
- Composite indices. Evolution of composite indices, such as the PCI or PSI is forecasted.

There is not any universal rule to determine which type of index must be deployed to better know the pavement deterioration evolution. All indices, those explained above, have their benefits and handicaps (Table 4.1) (AASHTO, 2012).

Table 4.1. Advantages and disadvantages of predicting different types of pavement condition variables (adapted from AASHTO, 2012).

Predicted variable	Advantage	Disadvantage
Distress severity and extent	- provides specific information about future distress severity - express in a format closely related to the way that the data are collected	- must determine the moment when the distress first appears in conjunction with the propagation of the distress since it appeared. - distress severities are combined for a particular distress to avoid modelling the progression of distress - difficult to be introduced in the PMS software
Individual indices	- are stated to be easier to develop - forecasted states are related to the factor that select treatments	- a specific model must be calculated for every index. - Updating several models with the new data can be tedious
Composite index	- considered as the simplest approach, resulting in the fewest number of models.	- same index value can be obtained by different pavement sections with different distress and the same age. The composite index will not indicate different deterioration rate for the two sections. Different distress types could mask the real evolution. - may not satisfy the needs of stakeholders who want more detailed models

4.4. Performance modelling approaches

After the index for modelling is determined, the type of model must be selected. They are several models for predicting pavement performance, and, moreover, there are some forms for classifying them. The Pavement Management Guide (AASHTO, 2012) classifies the models in the following types:

- **Deterministic models**. A single variable (dependent variable) is predicted from one or more variables (independent variables), usually factors that influence the forecasted value, by means of a regression analysis.
- **Probabilistic models**. A range of values for the dependent variable is forecasted. For example, the probability of a pavement section in one condition state to remain in the same level or change to another one after a cycle.
- **Bayesian models**. Objective and subjective data are combined in these models, where each of the variables introduced is described in terms of a probability function.
- **Subjective (or expert-based) models**. The relationship between the dependent and independent variables are stated after consulting different experts.

Moreover, it indicates that another classification approach can be considered according to the variables included (AASHTO, 2012), stating that models can also be classified as mechanistic, mechanistic-empirical or empirical. Empirical models are based or the results obtained in experiments, whereas the mechanistic ones use the fundamental principles of pavement behaviour. Mechanistic-empirical modelling combines both approaches and links forecasted condition to measured deterioration, by means of equations (FHWA, 1998).

Another classification is suggested by Haas *et al*., (1994) and Lytton (1987) (Table 4.2), distinguishing between project- and network level models and the two basic performance models: deterministic and probabilistic. The deterministic models are split according to the variable forecasted, and the probabilistic ones according to used approach.

Finally, Haas *et al.* (1994) summarized all these models into four basic types: purely mechanistic, mechanistic-empirical, regression (or deterministic) and subjective (indicating that probabilistic models are generally employed for subjective or expert based models).

Table 4.2. Classification of performance models (Haas et al., 1994).

Levels of Pavement Management	**Types of models**						
	Deterministic				Transition Process models		
	Primary response: - Deflection - Stress - Strain	Structural - Distress - Pavement - Condition	Functional - PSI - Safety	Damage - Load equiv.	Survivor curves	Markov	Semi-Markov
National Network				X	X	X	X
State Network		X	X	X	X	X	X
District Network		X	X	X	X	X	X
Project	X	X	X	X			

Moreover, Uddin (2006) proposes the following methodologies for performance modelling:

- Regression analysis techniques, which are based on multiple linear regression analysis techniques.
- Artificial Neural Networks Modelling,
- Probabilistic performance models, where the main approaches are the Bayesian and Markov models.
- Network level performance models
- Project-level performance models

Nonetheless, although the great range of existing performance models, deterministic and probabilistic models are the ones that are attracting the most attention. They are referred broadly as the basic groups by many authors (Lytton, 1987; Huang 1993; Li *et al.*, 1997; Hong and Wang, 2003; Ortiz-García *et al.*, 2006; Amin 2015; Abaza, 2016a; 2016b; Amin and Amador-Jiménez, 2016; Hassan *et al.*, 2017).

4.4.1. Deterministic models

Road agencies employ deterministic models when they have historical pavement condition information or enough survey data to identify statistically-significant pavement deterioration trends. Regression analyses are conducted for developing these models. They establish a relationship between two variables or more, which is not exact and have some variability. This variability and its magnitude are dependent on different factors, like the quality of data, the appropriateness of the selected independent variable or variables to forecast the dependent variable, and the range of data in the data set.

Since the relationship between the dependent variable, the predicted one, and the independent variable or variables, the predictor(s) is not exact, it must be determined the best statistical fit of the data. A usual approach in pavement management is the least square regression techniques, which minimizes the sum of the squared differences between the curve generated by the regression formula and the available data points. These differences between values are called residuals.

Different forms for those curves can be deployed for best fit the data, such as linear, quadratic, sigmoid, etc. and, hence, different shapes are adopted for the deterioration. Sometimes, only a single independent variable is employed to predict the dependent variable. In these cases, pavement age (or years since last major rehabilitation) or traffic volumes (total or heavy traffic volumes) are typically used as predictors. Nevertheless, in these situations, a more simply model results which can overcome the problems of incomplete databases, where some variables are not available, or they are not trustful. For example, if a multiple regression model has traffic volume as one of the independent variables, and no traffic counts have been carried out for year, these data cannot be introduced in the equation, and the accuracy of the prognosis would be lower and questioned.

Nonetheless, it is more usual to use multiple linear regressions to predict future condition, by using several statistically significant explanatory variable to model the variation of the dependant variable, typically a condition index, such as, IRI, PSI, PCI, etc. A scatter-plot of the available data should be analyzed to better estimate the model shape. The analysis of variance (ANOVA) is commonly employed to identify statistically significant independent variables. The multiple regression is able to develop empirical performance models by estimating parameters and coefficients of independent variables (explanatory) in deterministic mathematical equations in order to explain most of the variations in the dependent variable. The produced model could include linear and nonlinear terms of transformed variables. The goodness of the fitted model is assessed by means of different values. Further discussion is provided in section 4.5.

The first deterministic performance models were carried out after analysing the AASHO Road Test data, and could predict the number of accumulated ESAL applications the pavement could support as a function of drop in serviceability, layer material properties and thicknesses, subgrade strength and environmental factors (AASHO, 1962). They were applicable for project-level design.

The HDM III performance models forecast roughness and several distresses on asphalt pavements like cracking and rutting, including independent variables as present distress level, subgrade strength, environmental factor, traffic load and age (Watanatada *et al.*, 1987a). Initially, they were conducted for project-level analysis, but some authors adapted them for network-level analysis (Ferreira *et al.*, 2003).

Many of the most famous deterministic models that have extended broadly, such as the Washington State in the USA (LeClerc and Nelson, 1982; Jackson and Mahoney, 1990) and The World Bank's HDM models, in the different versions, must be previously verified in the new conditions, i.e. coefficients must be calibrated using local performance data.

4.4.2. Probabilistic models

Unlike deterministic regression equations that predict a precise value for an index, probabilistic models estimate the probabilistic distribution of the expected value (Uddin, 2006). While both models can develop the scope of predicting future condition, the probabilistic ones are able to incorporate uncertainty in pavement performance, which is assumed to be closer to reality (Golroo and Tighe, 2009; Kobayashi *et al*., 2012; Thomas and Sobanjo, 2013). Pavement performance is recognized to be probabilistic in nature, which requires assuming different levels of uncertainty (Li *et al*., 1997; Tjan and Pitaloka, 2005; Hong, 2014; Abaza, 2016a). Therefore, over the last three decades many authors have developed different forms of probability-based models (Anastasopoulus *et al*., 2011; Lethanh *et al*., 2015). Nonetheless, probabilistic models are not so employed as deterministic models in pavement management, likely because the majority of pavement management programmes are not prepared to input this kind of models without the conversion of the probabilistic model to a deterministic one. On the other hand, some agencies choose this approach as a way to incorporate this variability (AASHTO, 2012). Bayesian and Markov probabilistic models are attracting the greatest interest and can be stated as the most employed ones (Uddin, 2006).

4.4.2.1. Bayesian models

Bayesian statistical decision theory has arisen as emerging modelling technique for pavement management (AASHTO, 2012). It allows both subjective data from prior knowledge and experience and objectively collected data at present. Using these data, which can come totally from subjective or objective data, regression analysis is employed to develop the models. It differs from deterministic models in that each of the variables is assumed to be random and is associated to a probability distribution (Smith *et al*., 1979). These equations are then used for future deterioration estimations.

Since subjective data can be added to objective data, Bayesian regression can be deployed for highway administrations that have recently stated the implementation of a pavement management system or that have changed their procedure (and hence, no historical data are available), or that have introduced new materials in pavement section. It serves to override the influence of poor quality data or to update expert-based models as more data become available. Some examples of Bayesian models have been applied to pavement management systems, as in Mississippi (George, 2000),

4.4.2.2. Markov probabilistic models

In pavement management literature many examples about Markov and Semi-Markov transition probabilities can be found (Lytton, 1987; Haas *et al*., 1994; FHWA, 1998; Shahin, 2005). Probably the most widely employed probabilistic model is the discrete time Markov chains, with a large quantity of examples in pavement management literature (Bein, 1994; Chua and Monismith, 1994; Wang *et al*., 1994; Ortiz-García *et al*., 2006; Adedimila *et al*., 2009; Bandara and Gunaratne, 2001; Zou and Madanat, 2011; Marzouk *et al*., 2012; Lethanh and Adey, 2013; Soncim *et al*., 2017), and even used for other infrastructures like bridges (Roelfstra *et al*., 2004; Tsuda *et al*., 2006), wastewater systems (Baik *et al*., 2006) and pipelines (Sinha and Knight, 2004; Sinha and McKim, 2007).

The Markov prediction model is dependent on the present pavement condition and assumes that the probability from changing from one condition state to another does not depend on time. It is a stochastic process ruled by three restrictions (Ortiz-García *et al*., 2006):

- The process should be discrete in time
- The process should have a countable or finite state space
- The process should satisfy the 'Markov property' (Isaacson and Madsen, 1976)

The Markov property says that, given any past and present states, any future state of the process depends only on the present state, and it is independent of the past states (Hillier and Lieberman, 1990). It is widely assumed that the Markov property is fulfilled in pavement deterioration (Kerali and Snaith, 1992).

There are three main elements in a Markov process: The state probability vector, the cycle or step time and the transition matrix and they all are related to the number of condition states and the number of transitions.

The state probability vector, or condition probability vector, is a row vector that shows the current condition of the pavement via the proportions of pavement in each range of condition. For example, if an index ranging from 100 to 0 is used, being 100 the best value and 0 the worst, it can be divided in 10 states: 100-91, 90-81, 80-71,..., 10-0. Supposing that the present condition index indicates that 50 % of sections has an index value in 90-81 range, 30% in 80-71 and 20% in 70-61, the state vector of that road, $A = \{a_1, a_2, ..., a_i, ...a_n\}$, is:

$$A=\{0;\ 0{,}5;\ 0{,}3;\ 0{,}2;\ 0;\ 0;\ 0;\ 0;\ 0;\ 0\} \qquad [4.1]$$

The vector must satisfy that the sum of all a_i should be equal to one, and they should be nonnegative.

The step time is the time interval considered between two stages. As data are collected on an annual basis due to seasonal climate change cycle, one-year step is usually adopted (Ortiz-García *et al*., 2006). However, shorter cycle times were also developed with successful results (Pérez-Acebo *et al*., 2017b, 2018a).

The transition probability matrix (TPM) represents the pavement deterioration with time and it is commonly denoted by ***P***. It is a squared matrix, with *n* rows and *n* columns, where *n* indicates the number of states that are considered. In the previous example, *n* = 10. The general form of *P* is given by

$$P = \begin{bmatrix} p_{11} & p_{12} & \cdots & p_{1n} \\ p_{21} & p_{22} & \cdots & p_{2n} \\ \vdots & & \ddots & \vdots \\ p_{n1} & p_{n2} & \cdots & p_{nn} \end{bmatrix} \qquad [4.2]$$

Each element p_{ij} expresses the probabilities of an element in condition *i* in stage *t* to shift to condition state *j* in stage *t+1* (Eq. 4.3).

$$p_{ij} = prob\left[X(t+1) = j \,/\, X(t) = i\right] \qquad [4.3]$$

Hence, p_{ij} is described as the conditional probability of a parameter or an index *X* (any variable) in stage *i* in present condition will be in stage *j* after exactly one cycle or step. Every TPM elements of the matrix must meet some constraints (Wang *et al*., 1994):

$$0 \leq p_{ij} \leq 1, \text{ for all i and j, and i, j = 0, 1, 2,..., n.} \quad [4.4]$$

$$\sum_{i}^{n} p_{ij} = 1, \text{ for all } i \text{ and } i = 0, 1, 2,..., n. \quad [4.5]$$

Elements p_{ij}, located in the main diagonal, represent the probability of a road pavement to remain in the same condition state after one cycle. Elements p_{ij} for $i > j$ indicates that the section can evolve to a better state in next stage. This idea cannot be assumed in a normal pavement deterioration process without a maintenance or rehabilitation action on the road. If pavement deterioration is evaluated by means of TPMs, elements p_{ij} are equal to 0 for each i greater that j. Furthermore, after applying condition given in Eq. 4.5, element p_{nn} is equal to 1, expressing that road section in their worst condition cannot deteriorate further. As a result, the general form of P matrix in order to represent only pavement deterioration is denoted by Eq. 4.6.

$$P = \begin{bmatrix} p_{11} & p_{12} & p_{13} & \cdots & p_{1n} \\ 0 & p_{22} & p_{23} & \cdots & p_{2n} \\ 0 & 0 & p_{33} & \cdots & p_{3n} \\ \vdots & \vdots & \vdots & & \vdots \\ 0 & 0 & 0 & \cdots & 1 \end{bmatrix} \quad [4.6]$$

Many authors state that, without rehabilitation and maintenance works, any section can only remain in the same state or deteriorate to the next condition state (Abaza *et al.*, 2004; Butt *et al.*, 1987; Ortiz-Garcia *et al.*, 2006). It is supposed that sections cannot deteriorate beyond one state during one transition according to a reasonable deployment of condition states and cycle length (Costello *et al.*, 2005; Abaza and Murad, 2007; 2009; Abaza, 2016b). Consequently, only element $p_{i,i}$ and $p_{i,i+1}$ will be present in TPM, as shown in Eq. 4.7:

$$P = \begin{bmatrix} p_{11} & p_{12} & 0 & \cdots & 0 \\ 0 & p_{22} & p_{23} & \cdots & 0 \\ 0 & 0 & p_{33} & \cdots & 0 \\ \vdots & \vdots & \vdots & & \vdots \\ 0 & 0 & 0 & \cdots & 1 \end{bmatrix} \quad [4.7]$$

Moreover, due to the condition imposed in Eq. 4.5, it can be concluded that:

$$p_{i,i} = 1 - p_{i,i+1} \quad [4.8]$$

However, TPMs in which sections could drop two or more level after one cycle were presented (Pérez-Acebo *et al.*, 2017b; 2018a).

If the present current state vector and the TPM are available, it is possible to calculate the condition vector of the road in next stage, by matrix multiplication, as denoted in Eq. 4.9 and 4.10.

$$A_1 = A_0 \times P_1 \quad [4.9]$$

$$A_2 = A_1 \times P_2 \quad [4.10]$$

Where A_0, A_1 and A_2 are the state probability vector of the section at time t = 0, t = 1 and t =2 respectively and P_1 and P_2 are the transition matrices corresponding to transitions form step 0 to 1 and from 1 to 2 respectively. If the TPMs are assumed to remain constant over time, the same matrix can be applied to any t to $t+1$ transition. Processes that fulfil this idea are called homogenous Markov chains and they show a stationary condition, assuming that the deterioration rate over the analysis period does not change. As a result, any state vector in future time t, or after t transitions, can be calculated applying Eq. 4.9 t times (Chapman-Kolmogorov equations):

$$A_t = A_{t-1} \times P = A_{t-2} \times P \times P = A_{t-2} \times P^2 = A_{t-3} \times P^3 = \ldots = A_0 \times P^t \qquad [4.11]$$

The homogenous Markov model was used by some author to model pavement deterioration. (Smith, 1974; Kulkami, 1984; Carnahan, 1988; Abaza *et al.*, 2004; Kallen, 2011). However, it is said to be more realistic to assume a non stationary Markov Chain because transition matrices would vary along the service life of pavement due to changes in traffic volumes, environmental factors or subgrade strength (Feighan *et al.*, 1988; Li *et al.*, 1997; Hong and Wang, 2003; Golroo and Tighe 2009). Nevertheless, much more data, time and effort are necessary as $t+1$ sets of data are necessary for t transition matrices. Usually, in order not to estimate t TPM for each t steps, the service life of a pavement or life-cycle used in Life-Cycle Analysis is divided into some periods which employs the same matrix for the all steps in that period (Butt *et al.*, 1987).

Various methods for estimating the transition probability matrix have been proposed in the literature. Generally, two main methods are deployed: analyzing historical data about road network deterioration over time or from experts' judgment (Adedimila *et al.*, 2009; Tabatabaee and Ziyadi, 2013). The first approach needs collecting distress data for years and applying an adequate method to estimate the deterioration rates. Usually Eq. 4.12 is deployed to calculate every p_{ij} :

$$p_{ij} = \frac{N_{ij}}{N_i} \qquad [4.12]$$

Where N_{ij} is the number of road sections in the network that evolve from i state to j state during one step, N_i is the total number of road sections that were in state i before the transition.

As observed, Markov models do not allow including other variables, like traffic loading or environmental factors. The only possible solution would be the creation of different families and for each family develop different transition matrices.

On the other hand, the semi-Markov approach is said to overcome the independence of time that is assumed in Markov chains. Semi-Markov models allow transition probability matrices to be created and employed together to provide piecewise increments of time.

Probabilistic models are concluded to be useful when predicting individual distress evolution (Shahin, 2005).

4.4.3. Subjective or Expert-Based models

Subjective or expert-based models incorporate subjective opinions for performance models in a less formal way. This approach can be useful if historical condition data are not available, if new practices or materials are employed or if administration has little confidence in its data.

Models can be developed in an informally or formally. In an informal way, an expert (or a group of some experts) proposes an equation for describing the deterioration rate for a particular set of conditions. For example, in indices that rates from 100 (better condition) to 0 (worst condition), a deterioration rate of 3 points annually is a usual value.

In a formal process, a panel of experts determines ages at which certain events take place. Different combination of ages and conditions are employed to obtain deterioration curves, which are introduced into a regression analysis to develop and equation. The panel could determine the condition at which a reconstruction is needed, and subsequently, the number of years at which the pavement would be reconstructed. Then, correlation is established between the year of construction of the road and predicted year of reconstruction. Intermediary points could be also included to better estimate the shape of the curve.

4.4.4. Artificial Neural Networks Models

Artificial Neural Network (ANN) models have increased its popularity for empirical modelling for last years, by means of parallel computations for knowledge representation and information processing (Tarawneh and Nazzal, 2014). It was proved to be useful to solve certain types of problems that are difficult to solve with traditional numerical and statistical methods. ANNs consists of a group or artificial neurons that are interconnected in a way similar to the architecture of the human brain (Basheer, 1998).

The architecture of a simple neural network model can be seen as a collection of nodes distributed over an input layer, one or various layers and an output layer (Fig. 4.2). The input layer contains the independent variables that are introduced, input neurons, and the output layer has the dependent variable or variables (Singh *et al.*, 2012). The number of neurons in the input layer is the same as the number of input variables employed to predict the desired output, i.e. the independent variable. The neuron(s) in the output layer represent the variable(s) that are aimed to be predicted. As seen in Fig. 4.2 the input and output layers are connected by means of one (or various) intermediate hidden layers. The quantity of hidden layers is adopted after trial and error according to the complexity of the problem.

The neurons between two adjacent layers are usually interconnected. The strength of each connection is expressed by a numerical value called a weight. The weights are calculated after a training process that consists of presenting input and output samples to the network (Fig. 4.3). The ANN is said to learn the relationship between input values and output values after adapting the weights of the connections.

There are many ways a neural network can be trained. The “backpropagation” technique, developed by Rumelhart *et al.* (1986) is the most popular process, being employed in many fields of science and engineering. By means of this method, the weights of the network are adjusted during the training phase to minimize the error. In each iteration the error propagates backward to minimize the error to a desired level. A neural network carries out “computations” propagating changes in activation amongst the processors. Usually a binary threshold or a sigmoid function is used as the activation or transfer function. The neural network gains its knowledge by training. When the neural network processes all the training data, achieving a state of equilibrium, new input data can be introduced for evaluation. ANN models can learn complex, highly nonlinear relationships and associations from a large body of data, as a PMS have (Flood and Kartam, 1994).

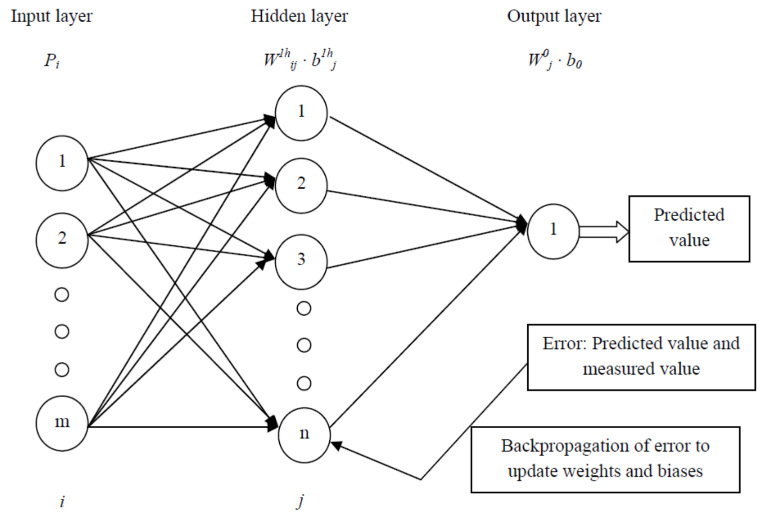


Fig. 4.2. Neural network flow diagram

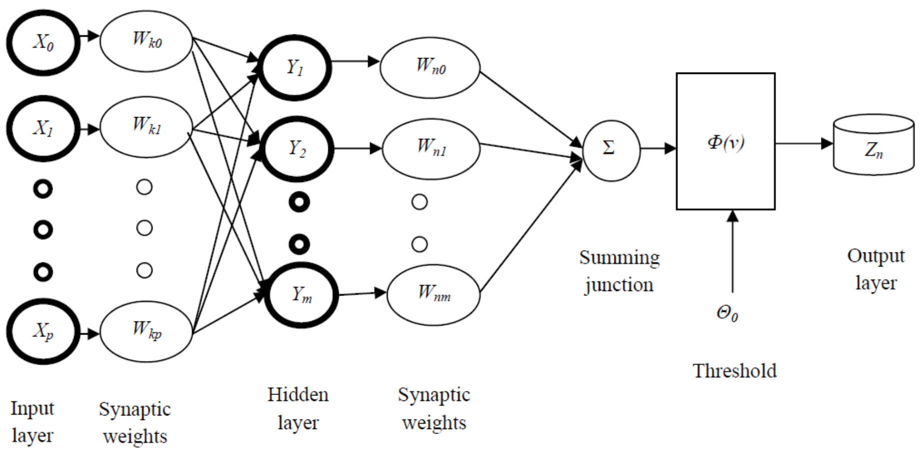


Fig. 4.3. Components of artificial neural network

As seen, ANN approach does not execute a series of fixed instructions, as a traditional computer programme or statistical analysis do. It solves, in parallel, to the inputs presented to it during the training period. On the other hand, ANN can also be regarded as a "*black-box*" approach since the results are produced without establishing casual relationships between input and output (Rohani *et al.*, 2011; Sollazzo *et al.*, 2017).

Apart from different areas of civil engineering, ANNs were also been deployed to predict pavement performance with successful results. Eldin and Senouci (1995b) presented an ANN to rate road pavement conditions. Terzi (2007) proposed a model to predict PSI considering distresses. Different kinds of ANNs were employed to forecast IRI evolution (Roberts and Attoh-Okkine, 1998). Kirbas and Karasahin (2016) compared regressions to ANN to predict PCI. Attoh-Okine (2001) employed an ANN model to assess pavement condition from distress by grouping different relevant pavement state variables. Owusu-Ababia (1998) proposed a procedure to predict cracking evolution, and Plati *et al.* (2016) used an ANN to assess pavement structural condition from FWD data. Other examples can be found in the literature, mainly with regard to pavement structural evaluation and IRI (Eldin and Senouci, 1995a; Far *et al*., 2009; Shah *et al*., 2013; Tarawneh and Nazzal, 2014; Ziari *et al*., 2016; Mirabdolazimi and Shafabakhsh, 2017; Sollazzo *et al*., 2017). ANN approach was also suggested within infrastructure engineering, such as for maintenance cost estimation and prioritization (Fwa and Chan, 1993; Bosurgi and Trifirò, 2005; Woldemariam *et al*., 2015; Amin and Amador-Jimenez, 2016), for pavement friction in airports (Fwa *et al*., 1997) and for aging analysis in asphalt binders (Xiao *et al*., 2012).

4.4.5. Empirical, mechanistic and mechanistic-empirical models. Project-level and network-level models

Road agencies use pavement deterioration models at project- and network-level. At network-level, performance models are employed for planning, programming and budgeting, whereas, at the project level, they are used to design pavements, to conduct life-cycle cost analyses, to select optimal designs with the least total costs, and in trade-off analyses where the annualized costs of new construction, maintenance, rehabilitation and user costs are taken into account for a specific pavement design (George, 2000).

Deterioration models can be either based on mechanistic principles, field observations, or a combination of both. Generally, an empirical approach is used for network level analyses, whereas a mechanistic or mechanistic-empirical approach is preferred for project level evaluation. Regression analyses are commonly used for fitting model equation parameters to an empirical set of data. For example, the District of Columbia Department of Transportation and the Georgia Department of Transportation developed network level models with empirically obtained data and statistical measures, like the t and F statistics (Stephenson, 2010).

On the other hand, project level models are said to be developed through more elaborate measures (Gallegos *et al*., 2013). They usually utilize mechanistic or mechanistic-empirical models with various forms. Mechanistic models are based on some primary response (behaviour) parameter like stress, strain or deflection. Most of the cracking and rutting models employed in mechanistic-based pavement design and project-level evaluations came from extensive laboratory testing and accelerated pavement testing (Roberts *et al*., 2003; Molenaar, 2003). They are recommended to be calibrated with field performance data as they do not adequately consider material variability, real world environment and traffic loads, and their interaction (Uddin, 2006). Consequently, appropriate performance measures should be identified and collected in order to develop and improve deterioration models. The Long-Term Pavement Performance (LTPP) database is an example of collected data. The LTPP programme was established to collect pavement performance data as one of the major research areas of the Strategic Highway Research Program (SHRP) in the USA. It provides data from over 2.500 test section located on in-service highways throughout North America, including existing and newly constructed pavements (FHWA, 2015).

With regard to the pavement design, it can be stated that nowadays, pavement design, especially in North America, has shifted from purely empirical approach (AASHTO, 1993) to mechanistic-empirical (M-E) approach (Pérez-Acebo *et al*., 2018b). M-E design upgrades on empirical design methods by introducing mechanistic behaviour, correlating recoded distresses to applied loads in the road structure (Retherford and

McDonald, 2013). The Mechanical-Empirical Pavement Design Guide (MEPDG) was produced in 2014 through the National Cooperative Highway Research Program (NCHRP) Project 1-37A (NCHRP, 2004) and subsequently delivered to the American Association of State Highway and Transportation Officials (AASHTO, 2008). It is one of the most thorough implementations of M-E design and provides pavement engineers powerful predictive tools through its design method and software packages, AASHTOWare Pavement Me Design (AASHTO, 2011), allowing for more cost effective and reliable pavement designs.

The MEPDG can be deployed for flexible and rigid pavements and includes the most used pavement structures in North America, incorporating the theoretical analysis to field observations in order to include the intricate consequences of traffic loading, climatic effects and material aging on pavement distresses. Nonetheless, MEPDG performance models were developed and calibrated with the LTPP programme data and, therefore, differences can be found when introducing "local" inputs, such as climate, material characteristics, traffic patterns or construction or maintenance and rehabilitation methods. Consequently, suggested deterioration models should be checked and calibrated with local data, as many states of the USA carried out, reporting either a partial or full calibration of the MEPDG on a local level. Some examples of these states are Montana (Von Quintus and Moulthrop, 2007), North Carolina (Muthadi and Kim, 2008), California (Ullidtz *et al.*, 2008), Texas (Banerjee *et al*., 2009), Arkansas (Hall *et al*., 2011), Iowa (Kim *et al.*, 2010) and Washington (Li *et al*., 2009a; 2009b). Hence, apart from being employed for designing durable pavement sections, the MEPDG contributes to the pavement management providing reliable and consistent performance models, calibrated both nationally and locally.

Furthermore, a national guideline for calibration, the Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide (AASHTO, 2012), derived from NCHRP Project 1-40B (Von Quintus *et al*., 2009) is receiving considerable research at present. On this basis, the results from a local validation of rutting data recorded in the PMS of the Ministry of Transportation of Ontario (MTO) showed that a unique optimal solution could not be achieved because rutting models were imprecise (Wasem and Yuan, 2013). Similar conclusion was reached in another study performed in Ontario (Yuan *et al*., 2012), demonstrating that no new distress models were better than the existing PMS model of the MTO. In contrast, fatigue and IRI models proposed in the MEPDG were concluded to be applicable in Louisiana (Wu *et al.*, 2013a) and in Tennessee (Zhou *et al*., 2013). However, some road administrations cannot calibrate the MEPDG models as required inputs are not available. Hence, Nassiri *et al.* (2013) identified the significant inputs for an IRI model in Alberta (Canada). Moreover, the recently published Pavement Management Guide (AASHTO, 2012) proposed the performance models shown in the MEPDG, and expects more accurate models as the models become calibrated in more states and countries.

4.4.6. Family modelling and site-specific models

Apart for the presented models, The Pavement Management Guide (AASHTO, 2012) proposes two approaches for deterioration modelling: pavement family models or site-specific models. A pavement family model is a unique employed to assess the evolution of a group of pavement sections with similar characteristics. A site-specific model is based on the unique characteristics of a particular pavement section.

4.4.6.1. Family models

Generally, pavement management system databases do not include all the variables that could be considered for pavement modelling or the data are not complete for all the years. In these cases, the family modelling approach was created. This method simplifies the modelling process, selecting a unique independent variable, usually age or traffic volume, which is used to forecast future conditions. The obtained equation only employs a single variable because the rest of variables were used to group pavement sections into ***families*** with similar characteristics and performance patterns. The equation is employed to predict future condition for pavement sections that fulfil the family definition. The definition of the pavement families must be clear and comprehensive enough that each pavement section in the database fall into one, and only one, pavement family.

A first family division could be asphalt- and concrete-surfaced pavements, which are supposed to have different deterioration rates. A further classification could be the road type, national or regional, which reflects a traffic volume division, without requiring traffic counts. Similarly, classifications according to “heavy” or “light” traffic volume can be carried out. The idea consists of dividing the road network into sub-networks with similar performance characteristics. These factors could also be the geographic location, surface type, function classification, heavy traffic volumes or combinations of them.

The major advantage of the family modelling approach is the use of some variables to classify pavement sections into families, rather than relying on the accuracy of the values for determining future condition.

Usually, deterioration models are developed for each pavement family by plotting the condition and inspection age of the section for all the sections that fulfil the requirements of the group. Normally, regression techniques are used.

As a model must be produced for each family, the number of classifications created has influence on the complexity of the PMS. For instance, if 3 pavement types are selected, 3 condition indices and 3 traffic categories, 27 (3 x 3 x 3) models have to be developed. If one of the families has not a complete range of data, it may be temporarily combined with a family with similar deterioration rates, until more data become available.

4.4.6.2. Site-specific models

Other road administrations prefer employing the unique characteristics of each pavement section to forecast pavement performance. A usual example of this type of model is the multiple variable regression equations, where the predicted condition is based on the specific data available in the database for that section. The predictions are considered to be site-specific because two pavement sections with identical condition information will not be expected to have the same deterioration rate if some variables introduced in the model are different, such as the climate, the pavement thickness or traffic.

A good example for observing the utilization of family and site-specific models is the Pavement Management System of the Colorado Department of Transportation (CDOT), which employs both types of curves (Keleman *et al.*, 2003).

4.5. Model reliability

Models that are created by means of regressions are usually controlled by statistical methods to know how well the model fits the data. The coefficient of determination (R^2) is said to be the most employed statistical parameter to check the "goodness of fit" of the model, i.e., the degree to which the model fits the data. The coefficient of determination represents the amount of variability that is possible to be explained by the model and it gives an idea about the goodness of the future conditions predicted by the model. It is the ratio of the sum of squares due to regression divided by the sum of squares about the mean, as shown in equation [4.13]:

$$R^2 = 1 - \frac{SS_{err}}{SS_{tot}} \qquad [4.13]$$

Where R^2 is the coefficient of determination, SS_{err} is the sum of squares of residuals and SS_{tot} is the total sum of squares. The coefficient of determination has a value between 0 and 1 (or in percentages, between 0 and 100 percent), and a higher values means a better fit of the model to the given dataset. Nevertheless, sometimes models with lower R^2 can be employed if the road agency determines that it represents better existing pavement deterioration. Moreover, other times R^2 values can be misleading, if, for example, models were created forcing the curve to intersect the x-axis in an exact point. Hence, these artificially constrained models cannot be compared directly with unconstrained models.

Sometimes it is necessary to eliminate spurious data or outliners, i.e., data points that seem to be erroneous or that do not make sense. For example, if a 20 year-old pavement section gives values for an index that indicate that it is in perfect condition, it can be removed. However, investigation is needed to know if those values are outliners (indicating that a recent rehabilitation was made and not recorded) or it is a legitimate data point.

Other statistical methods that are used to evaluate the reliability of the models are the standard error of estimate, root mean square error (RMSE), the analysis of residuals plots and the Student's *t* test and *F* test. Further discussion about these methodologies is provided in Chapter 7.

4.6. Update requirements

Over the chapter, the importance of the deterioration models was constantly underlined and therefore, the models should be reviewed periodically to verify that they represent actual deterioration rates. Usually, performance models are reviewed more intensely when a pavement management system is being implemented and the first models are created. Later, once models are verified, only minor changes may be introduced, but rarely important changes, which are only carried out when new rating procedures or condition indices are employed.

Sometimes, at the beginning of the implementation of a pavement management system, highway administrations construct their performance models based on experts' opinion, and later, these models incorporate real data as they become available. In this case, a Bayesian approach can be useful, as commented in section 4.4.2.1.

4.7. Main applications of performance models

By means of pavement management data, performance models can be developed to evaluate and predict the future condition of pavement sections. These models can be used to:

- Evaluate the performance of different pavement designs or mixes
- Model the performance of different maintenance and rehabilitation (M&R) treatments
- Conduct forensic studies
- Calculate remaining life
- Calibrate MEPDG models

4.7.1. Evaluation of the performance of pavement designs or mixes

Network-level performance models can be deployed to assess the performance of different pavement design or mixes in different ways. For instance, deterioration rates can be compared for different pavement families, flexible and semi-rigid pavements, subject to similar traffic volumes. Moreover, new mix designs or pavement designs can be evaluated and compared to existing traditional pavement models. Furthermore, if the model allows introducing some independent variables as different pavement layer thickness, it could be concluded the better design for each traffic volume fulfilling some service life requirements.

4.7.2. Modelling the performance of M&R treatments

Road agencies usually records performance data about rehabilitation and preventive maintenance works carried out in the pavement network. This record of treatment application has a great importance because it provides information about the benefits associated with the use of specific rehabilitation or maintenance treatments and it can explain the variations in pavement condition indices due to those activities. Rehabilitation and maintenance data needs the same referring system as the original pavement management system, which is generally carried out by any road administration.

In order to propose treatments, rehabilitation and maintenance models must be developed, similarly to models for other pavement sections. When both models are available, one model for a pavement section without any maintenance or rehabilitation work and another model for a pavement section with a specific treatment, the benefits associated with the rehabilitation, maintenance or preventive maintenance can be determined in terms of the additional are original pavement performance curve (model with no treatments) and the treated model curve (Fig. 4.4).

4.7.3. Conduction of a forensic study

Forensic studies are normally performed on pavement sections that failed prematurely to know the factor that influenced the atypical performance. They are also useful to know whether changes to existing design or existing practices are required. Forensic studies are usually carried out on an isolated pavement section, with very detailed project-level assessment methodologies, including a detailed pavement condition analysis, non-destructive testing, coring and material testing. Deterioration models at this level are site-specific and employ pavement design software with different load conditions.

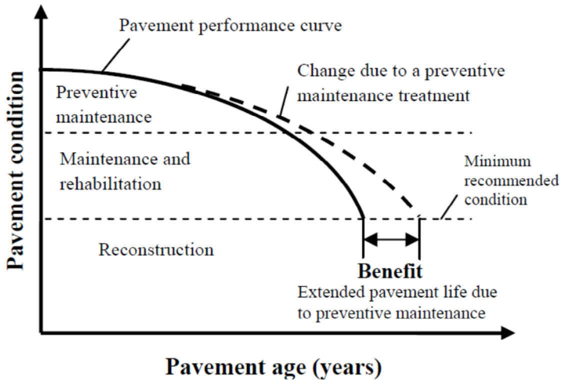


Fig. 4.4. Benefits of treatments. Pavement performance models, with no treatments, with preventive maintenance, with maintenance and with rehabilitation.

This kind of studies can be conducted to establish the differences in deterioration between the same rehabilitation or maintenance techniques in different regions or with different material properties. They may be also applied for determining the reasons for any differences in performance. These studies help suggesting changes in treatment selection, in material specifications or in construction practices (AASHTO, 2012).

4.7.4. Calculation of the remaining service life

One of the most interesting outputs from performance models is the possibility of knowing the remaining service life of a pavement section. Fig. 4.5 represents the known performance curve of a specific pavement structure, and the dashed line in that figure represents the predicted evolution of the pavement condition. If the road agency fixed a minimum threshold for that index of the Fig. 4.5, the minimum service level, the number of years before arriving to that point can be estimated. The remaining service life is generally expressed as a number of years, but it may be indicated in other units, such as, number of heavy vehicles, number of ESAL (equivalent single axle load), etc., depending on the independent variables of the model.

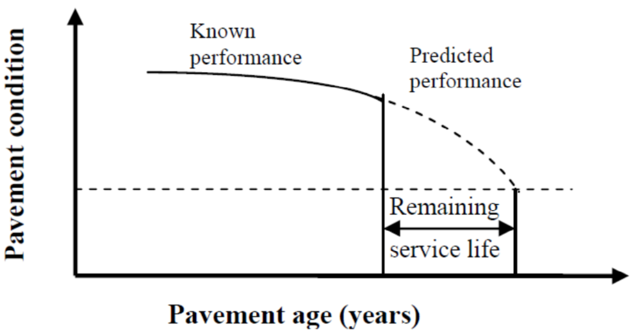


Fig. 4.5. Calculation of the remaining service life by means of performance models

Different thresholds are generally used, with the aim of providing different kind of alarms to pavement engineers. Usual thresholds are a first one called desirable minimum (or maximum), indicating that, after trespassing that limits, actions must be taken, and the absolute minimum (or maximum), determining the minimum acceptable level for a road in that network according to the prescriptions of the road agency.

4.7.5. Calibration of the MEPDG models

As commented in the chapter, the AASHTO (2008) published the Mechanistic-Empirical Pavement Design Guide (MEPDG), which changed the methodology employed until that moment for the design of pavement structures, combining mechanistic principles and empirically obtained results in situ. This new approach is expected to improve the efficiency of pavement designs, the prediction of the pavement performance and the evaluation of the maintenance and rehabilitation treatments over the life of the pavement structure.

As previously explained, the MEPDG was initially calibrated and validated with the FHWA Long-Term Pavement Performance (LTPP) programme (with more than 2.500 pavement sections), which tried to reproduce the majority of the pavement sections normally used in North America. Nevertheless, the great variability in terms of geography, climatic conditions, construction materials, construction practices, traffic compositions and volumes and other variables make desirable to calibrate the models proposed in the MEPDG at the local level using local field pavement data.

4.8. Conclusion of the chapter

The chapter presented the performance models as a vital and essential element in any pavement management system. Its importance comes from the capability of predicting future pavement condition, the ability to select the most cost-effective treatment strategy for a pavement and from the possibility of indicating the right moment to carry out the maintenance or rehabilitation work. The predicted variable must be established before starting any deterioration model.

The main phase when developing a performance model is to select the type of model that is going to be applied. There is a great variety of models, which can be classified according to several criteria. Each type of model is better adapted to certain circumstances, and therefore, careful analysis must be conducted when choosing the performance model. Models can also be classified according to their application, i.e., network-level or project-level projects, and they can be based on mechanistic principles, on empirical results or on both. A model can be developed to include some pavement families that meet some requirements, a family model; or be developed for a site-specific pavement.

Chapter 5. Proposed models for IRI and skid resistance performance prediction

5.1. Introduction to the chapter

It this chapter, roughness and skid resistance performance prediction models that have been proposed over the world in the last decades are commented. Some of them may were developed by researchers and other may even be in use in a pavement management system by any road agencies.

With regard to the roughness evolution models, firstly a general overview of affecting factors that were identified by authors is presented in section 5.2. The main document that lists the impacting factors on roughness was carried out by Paterson (1987) from the research funded by the World Bank. It is also commented further research to establish the factors that influence roughness evolution in different ways.

Then, in section 5.3 developed models for roughness progression are commented. They are grouped according to the classification of Chapter 4. Deterministic and probabilistic models attract the main attention of the section, as both are referred as the basic models. Deterministic models for roughness progression are divided in empirical and mechanistic-empirical models. Among the empirical ones, the HDM-III and specially the HDM-IV models are extensively commented, as they were developed following the research initiated by Paterson (1987) and carried out by the World Bank (Kerali *et al*., 2004). Other international projects, which were supported by various countries, are also exposed. Additionally, some recent empirical models are also mentioned. On the other hand, the Mechanistic-Empirical Pavement Design Guide (AASHTO, 2008) is becoming the main mechanistic-empirical model around the world.

Moreover, the probabilistic models are also extensively presented, with most of the latest finding for overcoming their principal problems: the calculation of transition probabilities and deciding between homogeneous or non-homogeneous Markov chains. A probabilistic model, recently developed for predicting the roughness progression on the National road network of the Republic of Moldova, is described (Pérez-Acebo *et al*., 2017b; 2018a). Finally, some of the recently suggested Bayesian and subjective models are identified, with the focus on the factors that must be introduced in the modelling.

Regarding the skid resistance, in a similar way to roughness, factor that have impact in the skid resistance evolution are deeply exposed in section 5.4, including some test that are used for calibrating those factors. Then skid resistance evolution models proposed by different researchers are described and commented.

5.2. Factors affecting roughness progression

Road roughness is said to be a good indicator of ride comfort, as the general public think that a good road as one that provide a smooth ride. This idea was verified for the first time in the pavement performance test sponsored by the AASHO when it was concluded that the subjective evaluation of the pavement based on mean panel ratings was primarily influenced by roughness.

The adoption of the IRI as a standard for roughness measures by the Federal Highway Administration led to become the widely used indicator for roughness around the World. IRI was related to PSI in an exponential form (Paterson, 1986). Additionally, a correlation was developed between IRI and pavement distress using neural networks and it was shown that the correlation coefficient between IRI and the distress variables was 0,944, indicating that IRI can be employed to represent pavement distresses (Lin and Hsiao, 2003). Therefore, when it is difficult to collect distress measurement due to limited resources, IRI can be used as a reliable index for maintenance and rehabilitation works. Despite the importance of the pavement smoothness, considered as one of the key measures of pavement performance, the contribution of different factors, such as pavement structure, rehabilitation techniques, climate, traffic levels, layer materials and properties, and pavement distress, to changes in pavement roughness is not well documented.

As stated by Paterson (1987), the progression of roughness with time is a complex phenomenon, which includes a combination of composite distress that depends on the following factors:

- Deformation due to traffic loading and rut depth variation,
- Surface defects from cracking, potholes and patching
- A combination of aging and environmental factors.

Paterson (1987) described perfectly the process of mechanisms of distress that affect the roughness (Fig. 5.1):

> *"Traffic axle loadings induce levels of stress and strain within the pavement layers which are functions of the stiffness and layer thickness of the materials and, which under repeated loading, cause the initiation of cracking through fatigue in bound materials and the deformation of all materials to various degrees, dependent upon the materials properties. Weathering causes bituminous surfacing materials to become brittle, and thus more susceptible to cracking and to disintegration (which includes raveling, spalling, and edge-breaking). Once initiated, cracking progresses in area and severity to the point where spalling and, ultimately, potholes develop. Open cracks on the surface and poorly maintained drainage systems permit excess water to enter the pavement, hastening the process of disintegration, reducing the shear strength of unbound materials, and thus, increasing the rate of deformation under the stresses induced by traffic loading. The cumulative deformation throughout the pavement depth is manifested in the wheelpaths as ruts are more generally in the surface as an unevenness or distortion of profile defined by roughness. Environmental effects of drainage, weather and seasons influence the strength and behavior of the pavement materials under traffic, and can cause distortion and volume changes which contribute to the roughness.*
>
> *Pavement roughness is therefore the result of a chain of distress mechanisms and combines the effects of various modes of distress. This process of interactive causes and effects, resulting ultimately in roughness, is a key concept in the approach to modeling. (…). Roughness cannot be considered in isolation form from these other causes as is evidenced by the correlation between different distress types."*

Paterson (1987) indicates that for pavement management systems (and for life-cycle prediction), the models should predict the expected change of condition in the future over a given period of time or under the transit of one extra axle road, when the current pavement condition is known. Therefore, the model should be incremental and of the following form:

$$Change\ in\ condition = f\left(current\ condition,\ pavement\ strength\ and\ age\ characteristics,\ environment,\ incremental\ time,\ incremental\ traffic\right) \quad [5.1]$$

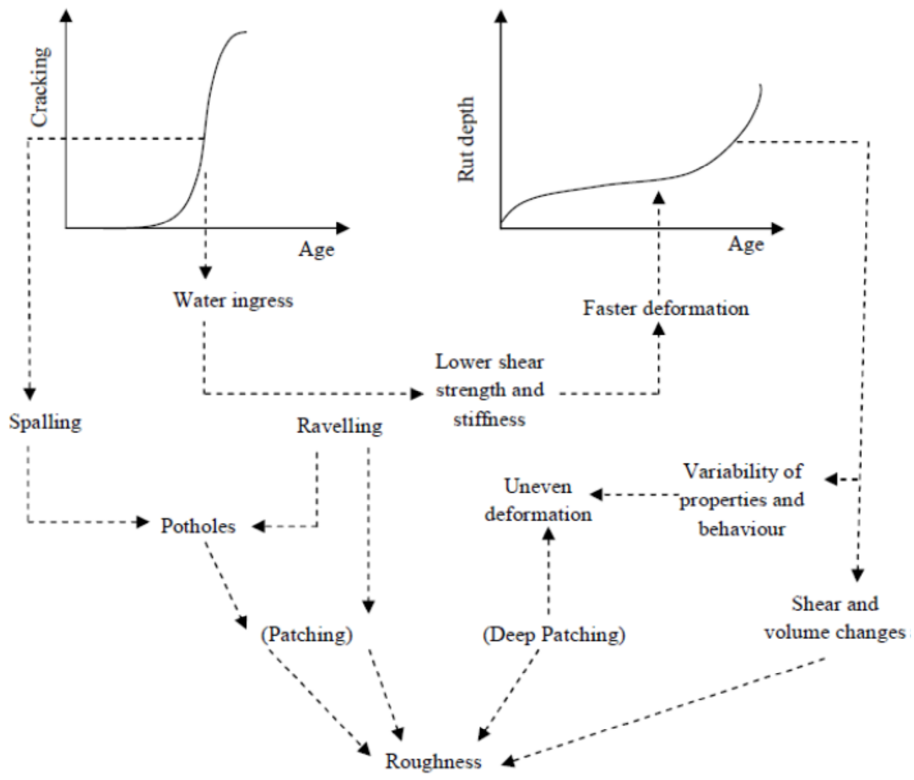


Fig. 5.1. The mechanisms and interactions of distress in paved roads (adapted from Paterson, 1987).

Moreover, models predicting absolute levels of distress are said to be of limited use because they are typical only of the average construction technique and quality particular to the study area. Paterson (1987) developed an increment model for IRI that incorporated all the factors affecting the roughness: traffic loading, pavement strength and type, aging, environment and secondary distress. It was an empirical model and was incorporated the model of the HDM-III, commented further in section 5.3.1.1.4. Additionally, another more simple model was also proposed, predicting the trend of the absolute level of roughness as a function of the four primary parameters: the cumulative traffic loading in Equivalent Single Axle (ESA), pavement age, pavement strength and a generalized environmental coefficient.

Most modern researches also indicate the factors that mainly influence the roughness progression. For instance, Madanat *et al.*, (2005) analyzed the roughness data available in the Washington State's PMS database and concluded that the most important factor of IRI evolution in asphalt concrete (AC) pavements and overlays are previous year IRI value, cumulative number of equivalent singe axle loads (ESALs), base thickness, total thickness of AC, including all overlays, age of pavement and minimum temperature in the coldest month average over the life of the pavement. Additionally, Perera and Kohn (2001) with long-term pavement performance (LTPP) data exposed that major factors in roughness progression in flexible pavements were design and rehabilitation parameters, climatic conditions, traffic levels, material properties and extent and severity of distress. Moreover, Wen (2011) analyzed design factors that have impact on initial IRI in 442 pavements and concluded that the main factors influencing initial pavement roughness were hot mix asphalt (HMA) layer thickness, project location (urban or rural), base type, HMA mix classification and pavement length.

With regard to environmental conditions, Perera and Kohn (2001) conducted an extensive analysis of the contribution of climate factors to the rate of change of IRI values in different test sections according to environmental zones and individual environmental zones. It was observed that asphalt pavements in the wet-freeze zone have the highest potential for change in roughness out of all environmental factor. In dry-freeze areas, higher rates of roughness progression were associated with higher annual precipitation, higher freezing indices and higher amounts of fine in the base layer. In dry no-freeze areas, higher deterioration rates were observed with higher mean annual precipitation, higher number of days above 32 °C, and higher plastic limits of subgrade. In wet-freeze zones, higher deterioration was related with lower total pavement thickness (sum of surface, base and subbase), lower annual precipitations, lower number of wet days, higher freezing indices, and higher amounts of fines in base layers. In wet no-freeze regions, higher rate was related to higher number of days above 32 °C, higher plasticity index of subgrade, higher moisture content of subgrade, higher fine contents in the subgrade and in sections with base layers that had higher fine content (passing No. 200 sieve > 50%).

As exposed in previous chapter, several types of performance models exits. Models suggested by authors and institutions for roughness evolution are grouped according to commented classification.

5.3. Proposed models for longitudinal roughness

5.3.1. Empirical models

Generally, in empirical models, measured or estimated variables are related to loss of serviceability or some other measure(s) of deterioration and pavement age, usually through regression analysis. It is said that the usefulness of these empirical equations is limited due to the database employed in their development. The regression equations are only valid under certain conditions and should not be applied in other circumstances (Saba, 2006). One of the most important and widely used empirical models for roughness progression is the HDM-4, developed by the World Bank, which can be adapted to different climates by means of calibration coefficients. This model is commented extensively due to its global importance.

5.3.1.1. Models proposed by the World Bank

In chapter 3 it was commented that the World Bank carried out a correlation experiment, the International Road Roughness Experiment (IRRE) with the aim of correlating the different roughness measurement devices (Sayers *et al*., 1986a). In the data process was observed that almost all the devices produced measures on the same scale, if a suitable scale was chosen. Thus, a new objective was added to the experiment, the development of an international index, which resulted in the International Roughness Index

It seems reasonable that if the World Bank promoted the standardization of roughness measurements and the development of an international standard, achieved by the IRI; the World Bank decided to include the IRI as an indicator of pavement roughness for their projects, and consequently, as for many other parameters, the World Bank itself developed IRI evolution models. These models were created to forecast the roughness evolution of roads included in the projects funded by the World Bank.

The different versions of the models have been employed in many countries and have fundamental to justify increasing road maintenance and rehabilitation funds in many countries. The models have been deployed to assess the economic viability of road projects in more than 100 countries and to optimise the economic benefits to road users under different levels of expenditures. Therefore, it confirms the applicability of the models in diverse climate and conditions.

The HDM-4 analytical framework is based on the concept of life cycle analysis. It can predict the road deterioration, the road works effects, the road user effects and the socio-economic and environmental effects over the life cycle of a road pavement, which is generally 15 to 40 years.

Consequently, the HDM-4, and also previous version, developed deterioration models to forecast future condition of roads, and the improvement observed after rehabilitation and maintenance works (Fig. 5.2a)

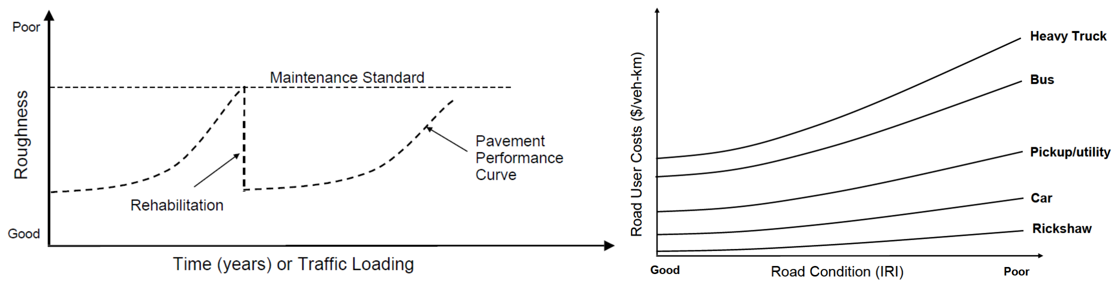


Fig. 5.2. a) Concept of life-cycle analysis in HDM-4, b) Effect of road condition on vehicle operating costs for rolling terrain (Morosiuk et al., 2004)

Additionally, the road condition also has impact on road users, measured in terms of road user costs, which include vehicle operation costs (fuel, tyres, oil, spare parts consumption, etc.), costs of travel time (for passengers and cargo) and cost to the economy of road accidents. For instance, Fig. 5.2b shows the impact of road condition, expressed in terms of IRI, on the cost of different modes of transportation. In next sections, models proposed for IRI evolution in the HDM-III and HDM-IV are commented.

5.3.1.1.1. Concepts about the models of the World Bank

Taking into account the two main groups for classifying condition projection methods, deterministic and probabilistic models, in HDM, **deterministic** models are employed. Additionally, deterministic models can be divided into mechanistic or empirical approaches. Mechanistic approaches are based on the knowledge of the stress and strains in the pavement obtained from fundamental theories of behaviour. They are usually very data intensive and rely on parameters which are difficult to quantify in the field. On the contrary, empirical models are usually based on statistical analyses of deterioration trend observed in a specific area. However, these models may not be useful for other areas.

In order to take advantage of the each approach and to overcome its disadvantages, Paterson (1987) followed a structured empirical approach for modelling the road deterioration and maintenance effects in the HDM-III model. It relied on the identification of the functional form and primary variables that affect each pavement property from both mechanistic and empirical information and the using various statistical techniques to combine their influence and impact. Therefore, resulting models combined both the theoretical and experimental bases of mechanistic approaches with the behaviour observed in the field under real circumstances. Consequently, proposed models can be defined as **mainly structured empirical models**.

Moreover, there are two types of predictive models: **absolute** models and **incremental** models. Absolute models forecast the condition at a particular point in time as a function of the independent variables of the model. On the contrary, incremental models give the change in the condition form an initial state as a function of the independent variables. Absolute models are said to have the disadvantage that they are generally confined to some specific conditions and cannot be employed outside those specific condition on which they are based. Incremental models are more flexible can be applied to different initial conditions.

The World Bank categorized road pavements in different categories. The main division include the difference between paved and unpaved roads. (It must be reminded that road projects funded by the World Bank included unpaved roads in undeveloped countries). For unpaved roads, there is only one category, unsealed roads. With regard to paved roads, it uses three groups according to the surface class: bituminous, concrete or block pavements. A further division is related to the specific surface type in each surface class. Among bituminous surfacing, there are two types: asphaltic mix (AM) and surface treatments (ST). The last classification indicates the base type of the pavement. The possibilities are granular base (GB), asphalt base (AB), stabilized base (SB), asphalt pavement (AP) and rigid (concrete) base (RB). The AP base type is employed when a surfacing is laid on top of an existing asphalt pavement and the RB base when a surfacing is laid on top of an existing concrete pavement. Integrating surface and base types, each type is described by a four-character code, combining the surface and base types codes indicated previously. Thus, the possible bituminous pavements are listed in Table 5.1.

Table 5.1. Pavement classification system of bituminous pavements in the HDM-IV (Morosiuk et al., 2004).

Surface category	Surface class	Surface type	Surface type	Pavement type
Paved	Bituminous	Asphalt mix (AM)	Granular base (GB)	AMGB
			Asphalt base (AB)	AMAB
			Stabilised base (SB)	AMSB
			Asphalt pavement (AP)	AMAP
			Rigid (concrete) base (RB)	AMRB
		Surface treatment (ST)	Granular base (GB)	STBG
			Asphalt base (AB)	STAB
			Stabilised base (SB)	STSB
			Asphalt pavement (AP)	STAP
			Rigid (concrete) base (RB)	STRB

The World Bank developed models for the types defined in Table 5.1 except AMRB and STRB, which are not included in the actual version of the HDM-4. The HDM-4 underlines the importance of the calibration factors since each mode of distress progresses with a different rate in different environments, and hence, models should always be calibrated to local condition before being incorporated to the pavement management system. With this aim, models include calibration factors, denoted by *K* with different subscripts, which multiply factor in order to scale a particular stress.

Furthermore, the HDM-4 models included adjustable model coefficient values, referred to as the a_i values. They are assigned to different pavement types, such as the ones indicated in Table 5.1, with default values, but they can be adjusted by users.

5.3.1.1.2. Factors included in the models of the HDM-4

The main factors that are generally introduced in the HDM-4 models are traffic, climate and environment, and age of pavement.

With regard to traffic, the main basic concept is the Annual Average Daily Traffic defined by Eq. 5.2:

$$AADT = \frac{Total\,annual\,traffic\,in\,both\,direction}{365} \qquad [5.2]$$

Additionally, two indices are also employed for deterioration prediction: Numbers of vehicles axles (*YAX*) and number of equivalent standard axle loads (*ESA*).

The number of vehicle axles is defined as the total number of axles of all vehicles traversing a given link in a specific year. For each vehicle type (*k*), the number of vehicles axles, YAX_k, traversing a section in a specific year is obtained from the volume of traffic of that type multiplied by the number of axles per vehicle of that type *k* (Eq. 5.3). For the total number of axles, *YAX*, in a year, the *YAK*'s for all vehicle types must be summed (Eq. 5.4).

$$YAX_k = \frac{T_k \cdot NAXLES_k}{ELANES \cdot 10^6} \qquad [5.3]$$

$$YAX = \sum_{k=1}^{K} YAX_K \qquad [5.4]$$

Where *YAX* is the annual total number of axles of all vehicle types (in millions per lane), T_k is the annual traffic volume of vehicle types *k* (k = 1, 2,..., *K*); $NAXLES_k$ is the number of axles per vehicle type *k*; *ELANES* is the effective number of lanes for the road section.

The equivalent standard axle load factor is defined as the number of applications of standard 80 kN dual-wheel single axle load that would cause the same amount of damage to a road as one application of the axle load being considered. The value of *ESALF* for each vehicle type may be specified by the model user or calculated from axle load information.

For each vehicle type, ESA_k is computed employing information on the different damaging effects of various axle configurations. For each type of axle group *j*, a standard load, $SAXL_j$, is employed to determine the loading ration. The equation to calculate *ESALF* is:

$$ESALF_k = \sum_{i=1}^{l_k} \frac{P_{ki}}{100} \sum_{j=1}^{J_k} \left(\frac{AXKL_{kij}}{SAXL_j} \right)^{LE} \qquad [5.5]$$

Where $ESALF_k$ is the equivalent standard axle load factor for vehicle type k, in equivalent standard axle loads, l_k is the number of subgroups i (defined in terms of load damage) of vehicle type k (i = 1, 2,..., l_k); P_{ki} is the percentage of vehicles in subgroup i of vehicle type k, LE is the axle load equivalency exponent (4,0 as a default value); J_k is the number of single axles per vehicle of type k; AXL_{kij} is the average load on axle j of load range i in vehicle type k (tonnes), $SAXL_j$ is the standard single axle load of axle group type j, usually the value of 8,16 tonnes for dual-wheel single axle is used for all single axles.

As seen, the factor $ESALF_k$ is an average over all vehicles of type k, loaded and unloaded, in both directions on the given section road. The annual number of equivalent standard axle loads is referred as to *YE4* and is obtained by Eq. 5.6:

$$YE4 = \sum \frac{T_k \cdot ESALF_k}{ELANES \cdot 10^6} \qquad [5.6]$$

Where *YE4* is the annual total number of equivalent standard axle loads, in millions/lane.

The cumulative number of equivalent standard axle loads (*ESA*) since the last rehabilitation or construction works (*NEA*) is calculated by Eq. 5.7:

$$NE4 = \sum_{y=1}^{AGE3} YE4_y \qquad [5.7]$$

Where *NEA4* is the cumulative number of equivalent standard axle loads since last rehabilitation (overlay) or reconstruction, in millions/lane, $YE4_y$ is the number of equivalent standard axle loads in year y, in millions/lane; and *AGE3* is the number of years since the last rehabilitation, in years.

Regarding to climate and environmental factors are also taken into consideration by the HDM-4 as the climate has impact on the rate of pavement deterioration. These factors are mainly related to temperature, precipitation and winter conditions. The HDM-4 classifies the environment in five moistures (Table 5.2) and in five temperatures (Table 5.3), which have been increased compared to HDM-III. Some climate factors are also used:

- The Mean Monthly Precipitation (MMP), expressed in mm/month.
- The freezing index, FI, which expresses the cumulative effect of the intensity and duration of subfreezing (< 0 °C) air temperatures.
- The temperature range (TRANGE) is the mean monthly ambient temperature range.
- The number of days in a year when the ambient temperature exceeds 32 °C (90°F), DAYS90.
- The drainage coefficient, which are determined by taken into account the quality of drainage and the percentage of time that the pavement is exposed to moisture levels approaching saturation.

All these coefficients are employed for concrete pavement deterioration models and hence, are not further commented.

Table 5.2. Moisture classification according to HDM-4 (Morosiuk et al., 2004).

Moisture classification	Description	Thornthwaite moisture index	Annual precipitation (mm)
Arid	Very low rainfall, high evaporation	-100 to -61	< 300
Semi-arid	Low rainfall	-60 to -21	300 to 800
Sub-humid	Moderate rainfall, or strongly seasonal rainfall	-20 to +19	800 to 1600
Humid	Moderate warm seasonal rainfall	+20 to +100	1500 to 3000
Per-humid	High rainfall, or very many wet-surface days	> 100	> 2400

Table 5.3. Temperature classification according to HDM-4 (Morosiuk et al., 2004).

Temperature classification	Description	Temperature range (°C)
Tropical	Warm temperatures in small range	20 to 35
Sub-tropical – hot	High day cool night temperatures, hot-cold season	-5 to 45
Sub-tropical – cool	Moderate day temperatures, cool winters	-10 to 30
Temperature – cool	Warm summer, shallow winter freeze	-20 to 25
Temperature – freeze	Cool summer, deep winter freeze	-40 to 20

Finally, the age of the pavement or the time since the last reconstruction, rehabilitation or maintenance work is introduced in the HDM-4 models. Four variables are defined:

AGE1 is used with preventive treatments. It is the time, in number of years, since the latest preventive treatment, reseal, overlay, pavement reconstruction or new construction activity.

AGE2 is related to surfacing age. It is the time, in number of years, since the latest reseal, overlay, pavement reconstruction or new construction activity.

AGE3 is related to rehabilitation age. It is the time, in number of years, since the latest overlay, pavement reconstruction or new construction activity.

AGE4 is related to the base construction age. It is the time, in number of years, since the latest reconstruction or new construction activity that involves the construction of a new base layer.

For bituminous pavements, the HDM-4 modelled the following pavement defects of bituminous pavements: drainage, cracking, raveling, potholing, edge break, rutting, roughness, texture depth, skid resistance

5.3.1.1.4. IRI performance models in HDM-III

Pavement deterioration is said to be a complex mechanism in which both external variables and distress modes interact. This is the case for the roughness model, which uses the output, directly or indirectly, from other distress models. Thus, the main hypothesis employed in the roughness evolution model of the HDM-III was that the various mechanisms that give rise to roughness changes must be included as components of the model, following ideas expressed previously by Paterson (1987). Consequently, an incremental model was adopted and the components that have impact on roughness were classified into three broad groups:

- **Structural deformation**. It refers to the deformation in the pavement materials caused by the shear stresses produced by traffic loadings. This group also includes the effects of environment factors on material strength and rutting evolution under loads. More specifically, the rut depth by itself does not increase the roughness if the rut depth is uniform. It is the variation of the rut depth which influences the roughness as deviations in the longitudinal profile. The typical variations of rut have medium wavelengths, in the range of 2 m to 10 m, and shorter if the cause is the base deformation.

- **Surface distress**. It refers to defects like cracking and potholes, which are normally associated with shallow-seated distress originating in either the surfacing or base of the pavement. This type of defects have an usual range in size from than 0,3 m to 2 m in diameter, which corresponds to a waveband of about 0 to 5 m wavelengths. Cracks are also included due to the local or "birdbath" depressions that often develop in cracked areas and to the effects of wide or spalled cracks.
- **Environmental factors**. Apart from the factors that have impact on structural strength, there are others that also influence roughness. Primarily temperature and moisture fluctuations and foundation movements, such as subsidence are included in this group.

The model developed by Paterson (1987) for roughness evolutions included five components: structural deformation, rutting, cracking, potholing and environmental factors. The equations proposed in HDM-III are:

$$\Delta IRI = K_{gp}\left[134 \cdot e^{m \cdot t} \cdot (SNCK + 1)^{-5} \cdot YE4 + 0{,}114 \cdot (RDS_b - RDS_a) + 0{,}0066 \cdot \Delta CRX + \right.$$

$$\left. + 0{,}42 \cdot \Delta POT\right] + m \cdot IRI_a \qquad [5.8]$$

Where

$$SNCK = \max[1{,}5; (SNC - \Delta SNK)] \qquad [5.9]$$

$$\Delta SNK = 0{,}0000758 \cdot [\min(63; CRX_a) \cdot HSNEW + ECR \cdot HSOLD] \qquad [5.\ 10]$$

$$ECR = \max[\min(CRX_a - PCRX; 40); 0] \qquad [5.11]$$

ΔIRI is annual incremental increase of roughness, in m/km IRI
IRI_a is the roughness at the start of the analysis year, in m/km IRI
m is the environmental coefficient (Table 5.4), where m = 0,023·K_{ge}
K_{ge} is the calibration factor for environmental coefficient
t is the time since latest overlay or construction (*AGE3*), in years
SNCK is the modified structural number for the pavement, reduced for the effect of cracking
YE4 is the annual number of equivalent standard axles, in millions/lane
RDS_b is the standard deviation of rut depth at the end of analysis tear, in mm
RDS_a is the standard deviation of rut depth at start of analysis year, in mm
ΔCRX is the annual incremental change in area of indexed cracking, in per cent
ΔPOT is the annual incremental change in area of potholing, in per cent
PCRX is the area of previous indexed cracking in old layer, in per cent
K_{gp} is the calibration factor for roughness progression

The main indictor for pavement strength in the HDM-III roughness model is *SNCK*, which considers the reduction in pavement strength caused by cracking in the bituminous layers, both in surfacing and in the underlying bituminous layers.

The model employs two calibrations factors, K_{gp} and K_{ge}. K_{gp} is employed to adjust the rate of progression. Specifically, K_{ge} is deployed to adjust the environmental coefficient, *m*. As the data for the model were collected in Brazil, the value of *m* was set to 0,023 for the Brazil climate. For other climates, K_{ge} is selected to adjust the value of m to that which is appropriate for that climate area. Essentially, the value of K_{ge} is the ratio of the value of m for the appropriate climate and 0,023 (Eq. 5.12)

$$K_{ge} = \frac{m}{0{,}023} \quad [5.12]$$

The values of the environmental coefficient m for the climates defined in HDM-III are listed in Table 5.4.

Table 5.4. Environmental coefficient "m" by HDM-III climate zones (Paterson, 1987)

Moisture classification	**Thornthwaite Moisture Index**	**Temperature classification**		
		Tropical Non-freezing	**Sub-tropical Non freezing**	**Temperate Freezing**
Arid	-110 to -61	0,005	0,010	0,025
Semi-arid	-60 to -21	0,010	0,016	0,035
Sub-humid	-20 to +19	0,020	0,030	0,065
Humid, wet	20 to 100	0,025	0,040	0,10 – 0,23

Variants of the general model exposed exist (Paterson and Attoh-Okine, 1992). They can be used when one or more of the distress parameters are missing.

5.3.1.1.5. IRI performance models in HDM-IV

The roughness model developed by the World Bank in the HDM-4 is based on the model shown for HDM-III and has also the same five components. While the structural, cracking, rutting and environmental factors are similar to the HDM-III, the potholing factor has been changed. The relationship between factors is shown in Fig. 5.3. Moreover, pavement roughness is combined with shoulder deterioration and edge breaks in order to provide the model for effective roughness, which is the roughness on narrow pavements where users are forced to use the shoulder to pass other vehicles. The relationship between inputs is shown in Fig. 5.4.

The model proposed by the World Bank in the HDM-4 is also an incremental model, which is the sum of various components (Eq. 5.13):

$$\Delta RI = \Delta RI_S + \Delta RI_C + \Delta RI_r + \Delta RI_p + \Delta RI_e \quad [5.13]$$

Where

ΔRI is the total incremental change in roughness during analysis year, in m/km IRI

ΔRI_S is the structural component of roughness, explained in section 5.3.1.1.5.1

ΔRI_C is the cracking component of roughness, explained in section 5.3.1.1.5.2.

ΔRI_r is the rutting component of roughness, explained in section 5.3.1.1.5.3.

ΔRI_p is the potholing component of roughness, explained in section 5.3.1.1.5.4

ΔRI_e is the environmental component of roughness, explained in section 5.3.1.1.5.5

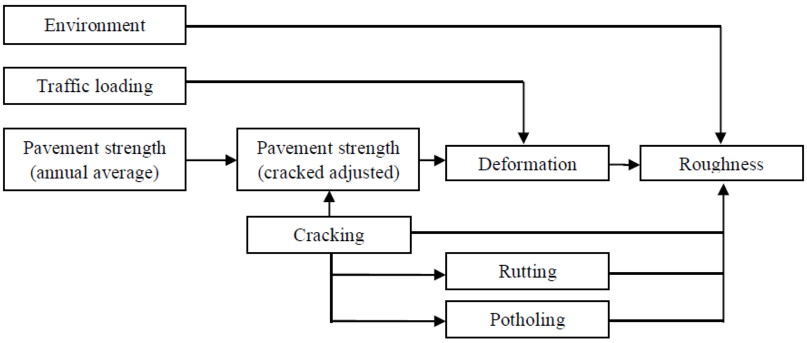


Fig. 5.3. Dependence of roughness on other model parameters (adapted from Morosiuk et al., 2004)

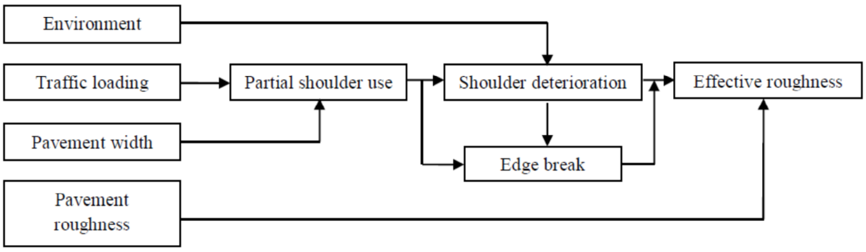


Fig. 5.4. Combination of shoulder deterioration, edge break and roughness to calculate effective roughness (adapted from Morosiuk et al., 2004)

As it is an incremental model, the roughness of the pavement at the end of the analysis year is given by:

$$RI_b = \min\left[\left(RI_a + \Delta RI\right), a_0\right] \qquad [5.14]$$

Where RI_b is the roughness of the pavement at the end of the analysis year (IRI m/km); RI_a is the roughness of the pavement at the start of the analysis year (IRI m/km), and a_0 is the upper limit of pavement roughness specified by the user. HDM-4 proposes 16 IRI m/km as default value.

The annual average roughness of the pavement for a given analysis year is obtained from:

$$RI_{av} = 0{,}5 \cdot \left(RI_a + RI_b\right) \qquad [5.15]$$

Where RI_{av} is the annual average roughness of the pavement for the analysis year (IRI m/km). This value is used in the Road User Effect Model.

5.3.1.1.5.1. Structural component

The HDM-4 structural component of roughness is obtained from Eq. 5.16:

$$\Delta RI_S = K_{gs} \cdot a_0 \cdot \exp\left[K_{gm}(m) \cdot AGE\right] \cdot \left(1 + SNPK_b\right)^{-5} \cdot YE4 \qquad [5.16]$$

Where

$$SNPK_b = \max\left[\left(SNP_a - dSNPK\right); 1{,}5\right] \qquad [5.17]$$

$$dSNPK = K_{snpk} \cdot a_0 \cdot \left[\min\left(a_1; ACX_a\right) \cdot HSNEW + \max\left(\min\left(ACX_a - PACX; a_2\right); 0\right) \cdot HSOLD\right] \qquad [5.18]$$

ΔRI_s is the incremental change in roughness due to structural deterioration during analysis year, in m/km IRI
dSNPK is the reduction in adjusted structural number due to cracking
$SNPK_b$ is the adjusted structural number due to cracking at end of analysis year
SNP_a is the adjusted structural number at start of analysis year
ACX_a is the area of indexed cracking at start of analysis year, in per cent
PACX is the area of previous indexed cracking in old surfacing, in per cent
HSNEW is the thickness of the most recent surfacing, in mm
HSOLD is the total thickness of previous underlying surfacing layers, in mm
AGE3 is the age since last overlay or reconstruction, in years
YE4 is the annual number of equivalent standard axles, in millions/lane
m is the environmental coefficient (Table 5.5)
K_{gm} is the calibration factor for environmental coefficient
K_{snpk} is the calibration factor for SNPK
K_{gs} is the calibration factor for the structural component of roughenss
a_0 and a_2 are coefficients and their values are indicated in Table 5.7

For the pavement strength indicator, the HDM-4 model uses the adjusted structural number (*SNP*) instead of the modified structural number (*SNC*) employed in HDM-III. As seen in Eq. 5.12, the calibration factor for the environmental coefficient *m* was K_{ge}, defined as ratio of the appropriate m value for the climate and 0,023. With the aim of maintaining the default value of all the other calibration factors in both HDM-III and HDM-4, the default value of the calibration factor for *m* has been set to 1,0 in the HDM-4 model. With the aim of distinguishing this factor from that used in HDM-II, it has been re-named as K_{gm} in HDM-4. Hence, the appropriate values of *m*, shown in Table 5.5, are input directly into the HDM-4 structural component of the roughness model (Eq. 5.16).

Table 5.5. Environmental coefficient "m" in HDM-4 (Morosiuk et al., 2004).

Moisture classification	Temperature classification				
	Tropical	**Sub-tropical hot**	**Sub-tropical cool**	**Temperature cool**	**Temperature freeze**
Arid	0,005	0,010	0,015	0,020	0,030
Semi-arid	0,010	0,015	0,020	0,030	0,040
Sub-humid	0,020	0,025	0,030	0,040	0,050
Humid	0,025	0,030	0,040	0,050	0,060
Per-humid	0,030	0,040	0,050		

Apart from the calibration factor assigned to the structural component, another calibration factor, K_{snpk}, was introduced into the relationships for predicting the change in structural strength of the pavement due to cracking. It allows the user of the model altering the influence of cracking on pavement strength according to the historical data available.

5.3.1.1.5.2. Cracking component

The cracking component of roughness in HDM-4 model is the same as in HDM-III, with addition of a calibration factor, and is obtained from Eq. 5.19:

$$\Delta RI_c = K_{gc} \cdot a_0 \cdot \Delta ACRA \qquad [5.19]$$

Where

ΔRI_c is the incremental change in roughness due to cracking during analysis year, in m/km IRI

$\Delta ACRA$ is the incremental change in area of total cracking during analysis year, in per cent

K_{gc} is the calibration factor for the cracking component of roughness

For calculating $\Delta ACRA$, information related to structural strength of the pavement must be introduced in the proposed models, such as Benkelman beam deflection, resilient modulus of soil cement, etc. The complete model is available in Morosiuk *et al.* (2004).

5.3.1.1.5.3. Rutting component

The rutting component of roughness in the HDM-4 model is obtained by:

$$\Delta RI_r = K_{gr} \cdot a_0 \cdot \Delta RDS \qquad [5.20]$$

Where

Δ_{RIr} is the incremental change in roughness due to rutting during analysis year, in m/km IRI

ΔRDS is the incremental change in standard deviation of rut depth during analysis year, in mm

K_{gr} is a calibration factor for the rutting component of roughness

As seen, the incremental increase in roughness due to rutting in HDM-4 is a function of the standard deviation or rut depth, as in HDM-III. Nevertheless, the value of the coefficient a_0 (shown in Table 5.7) was adapted to the change in definition of rut depths from those measured under a 1,2 m straight-edge in HDM-III to those measured under a 2,0 m straight-edge in HDM-4

5.3.1.1.5.4. Potholing component

The potholing component of roughness is calculated by means of Eq. 5.21:

$$\Delta RI_p = K_{gp} \cdot a_0 \cdot (a_1 - FM) \cdot \left(NPT_{bu}^{a2} - NPT_a^{a2} \right) \qquad [5.21]$$

Where

$$NPT_{bu} = NPT_b \cdot \left[1 - \left(\frac{Ppt}{100} \right) \cdot \left(1 - \frac{Fpat}{365} \right) \right] \qquad [5.22]$$

ΔRI_p is the incremental change in roughness due to potholing during analysis year, in m/km IRI

NPT_a is the number of potholes units per km at start of the analysis year

NPT_b is the number of potholes per km at the end of the analysis year

NPT_{bu} is the number of potholes per km at end of the analysis year, as perceived by the road user

FM is the freedom to manoeuvre index

Ppt is the percentage of potholes patched

Fpat is the frequency of pothole patching, in days

K_{gp} is the calibration factor for the potholing component of roughness

a_0 and a_2 are coefficients and their values are indicated in Table 5.7.

To calculate this component a long process were conducted. Paterson (1987) simulated the effects of different sizes and frequency of potholes on roughness and obtained a relationship with a high degree of correlation:

$$\Delta RI_p = 6{,}0 \cdot V_{pot} \qquad [5.23]$$

Where

ΔRI_p is the incremental change in roughness due to potholing, in m/km IRI

V_{pot} is the volume of potholes, in m^3/km

This relationship was obtained in an experiment in which the vehicle hit all potholes and the limited field data available suggested that the actual relationship between volume of potholing and IRI was much lower. The reason is that drivers will try to avoid potholes as long as road and traffic condition make it possible. Hence, the coefficient adopted in the HDM-III model was 0,16, i.e., 35 times smaller than the one of the experiment. It must be underlined that the effect of potholes on a vehicle is complex, which is a function of the occurrence and sizes of potholes and the freedom to manoeuvre of the vehicle to take avoiding action. In the case that all the potholes were in the wheelpaths and the vehicle had not freedom to manoeuvre, due to road width or traffic congestion, the vehicle would hit all the potholes, i.e., the 100 per cent of them. On the contrary, if only few isolated potholes exist in a two lane road with no other traffic, driver would probably avoid most of them, or even all.

The spatial concurrence of potholes may be regarded as random, both longitudinally and laterally. If potholes developed continuously, it will be reached the point where it is impossible to avoid all potholes even without traffic. Nevertheless, even when a great quantity of potholes is present, vehicles will not hit 100 per cent of them due to the lateral distribution. It is clear that linear relationships do not exist between number of potholes and the effect on vehicles in terms of received impacts. It is suggested that the percentage of potholes hit and hence, the resulting roughness effect, will follow a pattern as shown in Fig. 5.5 for different levels of freedom to manoeuvre.

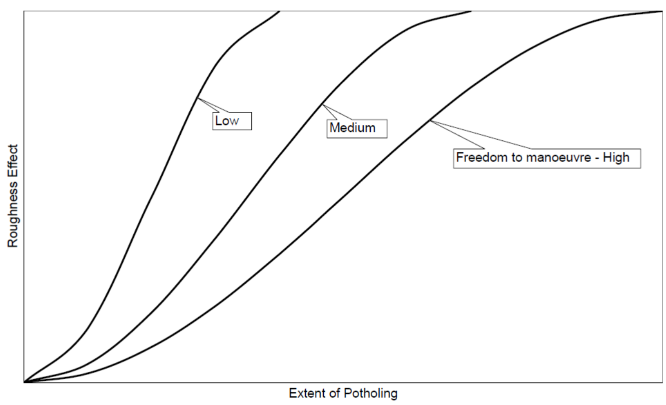


Fig. 5.5. Conceptual model of potholing effect on roughness (Morosiuk et al., 2004).

The HRTS team in Malaysia suggested a simple linear model for a freedom of manoeuvre index with a scale of 0 to 1, with these assumptions (NDLI, 1995):

> With a pavement width of 7 m and no traffic, a driver will have complete freedom to avoid potholes. With a pavement width of 3 m or traffic volume of 5.000 *AADT*, the driver will not be free to manoeuvre.

With these assumptions, a freedom to manoeuvre model was proposed:

$$FM = [\max[\min(0{,}25 \cdot (CW - 3);1);0]] \cdot [\max((1 - AADT/5000);0)] \qquad [5.24]$$

Where

FM is the freedom to manoeuvre index

CW is the carriage width, in m

AADT is the two-way traffic flow, in veh/day.

The relationship is exposed in Fig. 5.6.

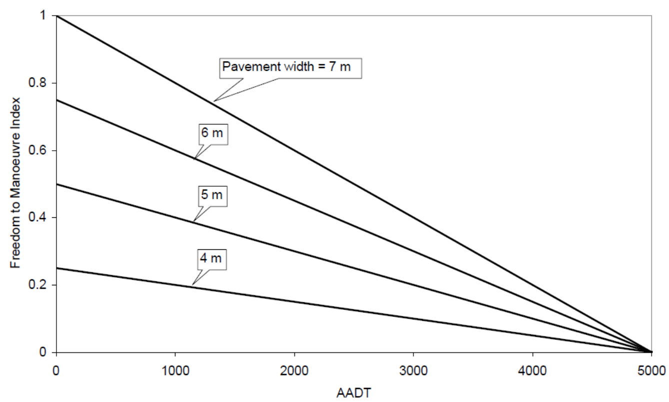


Fig. 5.6. Freedom to manoeuvre index (FM) (Morosiuk et al., 2004)

The FM index can then be introduced to the potholing component of roughness model as indicated in Eq. 5.25 (NDLI, 1995):

$$RI_p = \min[a_0 \cdot (a_1 - FM) \cdot NPT^{a2}; a_3] \qquad [5.25]$$

Where

RI_p is the roughness due to potholing

NPT is the number of potholes units per km

a_0, a_1, a_2 and a_3 are model coefficients

If the HDM-III relationship is converted from per cent area to the number of potholes units per km, it implies that roughness effect of 1.000 potholes units per lane and km would be approximately 0,84 m/km IRI, which seems a very low value. The HRTS team affirmed that at this pothole rate the incremental roughness would 10 m/km IRI with total freedom of manoeuvre and 20 m/km IRI without manoeuvre freedom. Thus, the coefficient values for the above model are shown in Table 5.6 and model is represented in Fig. 5.7.

Table 5.6. Coefficient values for original potholing components of roughness model (Morosiuk et al., 2004)

a_0	a_1	a_2	a_3
0,0000125	2	1,5	20

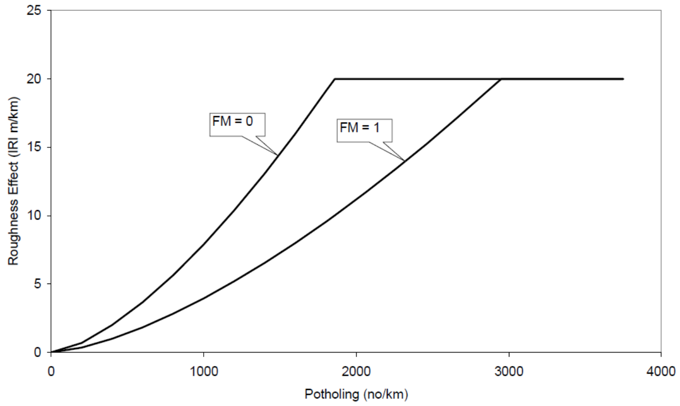


Fig. 5.7. Original HRTS proposed model for potholing component of roughness (Morosiuk et al., 2004)

Nevertheless, HDM-4 models the patching of potholes to occur at regular intervals throughout the year, unlike the other routine maintenance activities, whose effects are modelled only at the end of each analysis year. If potholes are patches more frequently, the user will be exposed to their effects for a shorter period of time. Hence, the frequency of patching (*Fpat*) and the percentage of potholes patched (*Ppt*) during each campaign needs to be incorporated into the model.

There will be difference in the effects of potholes existing at the start of the year (NPTa) if no patching was applied in the previous year and new potholes appearing during the year (ΔNPT). For instance, if patching frequency is one month, the initial potholes will all be repaired after one month and will have no effect for the remaining 11 months of the year. Consequently, new potholes will appear at regular intervals and be repaired at regular intervals. Hence, the two terms NPT_a and ΔNPT requires a different application of the term *TLF* (Time lapse factor). Riley (1998) developed a model to predict the incremental change in roughness due to potholing, incorporating the maintenance frequency of pothole patching. This model employed the original HRTS equation (Eq. 5.25) and the *TLF* variable as commented above and was selected for use in the draft version of the HDM-4 model, version 1 (Morosiuk *et al*., 2001)

$$\Delta RI_p = a_0 \cdot (a_1 FM) \cdot \left[(NPT_a \cdot TLF + \Delta NPT \cdot TLF/2)^{a2} - NPT_a^{\,a2} \right] \qquad [5.26]$$

Where

ΔRI_p is the incremental change in roughness due to potholing during analysis year, in m/km IRI
ΔNPT is the incremental change in pothole units during analysis year, in number/km
NPT_a is the potholes units per km at start of the analysis year
FM is the freedom to manoeuvre index
TLF is the time lapse factor

Finally, Eq. 5.23 was the equation that was introduced in the version 2 of the HDM-4 model.

For potholing factor calculation, cracking initiation and ravelling models must be employed, which depend on strength properties of the pavement structure.

5.3.1.1.5.5. Environmental component

The environmental component of roughness is calculated by means of Eq. 5.27:

$$\Delta RI_e = K_{gm} \cdot m \cdot RI_a \qquad [5.27]$$

Where

ΔRI_e is the incremental change in roughness due to the environment in analysis year, in m/km IRI
RI_a is the roughness at the start of the analysis year, in m/km IRI
m is the environmental coefficient (Table 5.5)
K_{gm} is the calibration factor for the environmental component (default value is 1,0).

The environmental component imitates the ideas of the HDM-III, but the definition and symbol of the calibration factor K_{gm} was modified as commented in section 5.3.1.1.5.1, resulting in the value of the environmental coefficient, m, which is introduced directly into de model.

5.3.1.1.5.6. General overview of the HDM-4 model for roughness

The coefficient values for the various roughness components are shown in Table 5.7.

Table 5.7. Coefficient values for roughness components

Pavement type	Roughness component	Equation	a_0	a_1	a_2
All pavement types	Structural	5.18	134	-	-
	dSNPK	5.20	0,0000758	63	40
	Cracking	5.21	0,0066	-	-
	Rutting	5.22	0,088	-	-
	Potholing	5.23	0,00019	2	1,5

The value of a_0 in the potholing component was modified to 0.00019 instead of the original value of 0,000125 proposed by the HTRS team, to accommodate the change to the standard size of a pothole unit.

Some examples of roughness evolution are shown. In Fig. 5.8, a relatively weak pavement that has a low traffic volume is plotted and in Fig. 5.9 a strong pavement with high traffic volume. The contribution of each component can be observed in the figures. Whereas with low traffic volumes, the environmental component is undoubtedly the main contribution to roughness, with high traffic volumes the rutting components can be said to be the main contributor.

Additionally, the HDM-4 Manual itself proposes two modifications to the HDM-4 Roughness model (Morosiuk *et al*., 2004). The explanation for that is that the roughness of a pavement at the end of a year with regard to the maintenance and rehabilitation works required to be performed may be different from the effective roughness of the pavement perceived by road users. Consequently, Riley (2000) proposed two roughness values, which are included in the HDM-4 Manual (Morosuik *et al*., 2004):

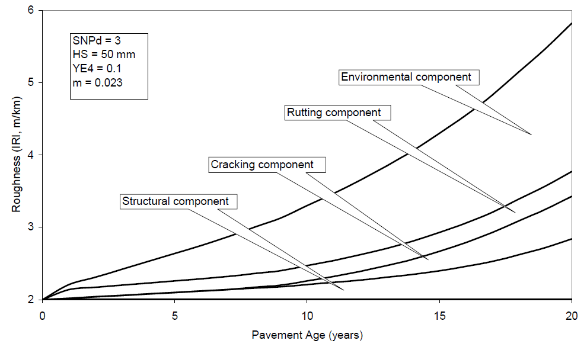


Fig. 5.8. HDM-4 predicted rates of roughness progression in a low traffic volume road (Morosuik et al., 2004)

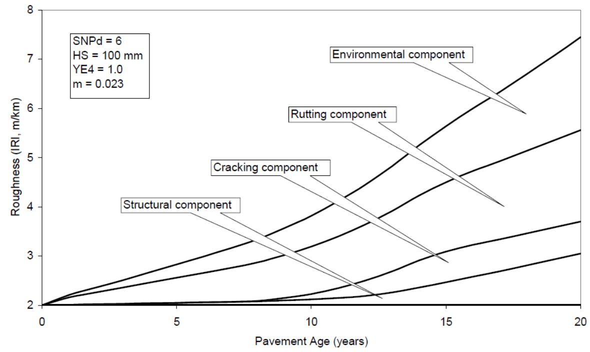


Fig. 5.9. HDM-4 predicted rates of roughness evolution in a high traffic volume road (Morosuik et al., 2004).

- The roughness of the pavement representing its longitudinal profile, excluding the effects of potholes or partial shoulder use. It can be employed to conduct M&R works
- The average roughness experienced by users including the transient effects of potholes and partial shoulder use. It can be employed as the roughness of the pavement in the road user effect relationships, referred to as the effective roughness of the pavement.

The first modification, name Pavement Roughness for Work Effects, is the roughness at the end of the analysis year, proposed for employment as a trigger level for Works Effects, excluding the potholing component (Eq. 5.28)

$$RI_b = \min[(RI_a + \Delta RI); a_0] \qquad [5.28]$$

Where

$$\Delta RI = \Delta RI_S + \Delta RI_C + \Delta RI_r + \Delta RI_e \qquad [5.29]$$

RI_b is the roughness of the pavement at the end of the analysis year, in m/km IRI
RI_a is the roughness of the pavement at start od the analysis year, in m/km IRI
ΔRI is the incremental change in roughness during analysis year, in m/km IRI
a_0 is the upper limit specified by any road agency or user for pavement roughness (default = 16)
And the variables ΔRI_S, ΔRI_C, ΔRI_r and ΔRI_e are the different components of the roughness model.

With regard to the second modification, it considers that on narrow roads vehicles may be forced to make partial use of the shoulders when meeting oncoming traffic or when overtaking. When vehicles are obliged to use the shoulder, they experience a higher roughness than the one predicted by the model, especially if the shoulders are unsealed. This model is included in the HDM-4 Manual (Morosiuk *et al*., 2004).

5.3.1.2. Models in the COST Action 324 Long term performance of road pavements

COST Action 324 had the aim of integrating research studies throughout Europe in the field of pavement performance. It was conducted by a consortium of 16 organisations from 15 European countries, in a two year project, under shared funding by the European Commission and the participating organisations (European Commission, 1997). The technical basis for the project was laid under the auspices of the European Commission's COST-programme European Co-operation in Scientific and Technical Research. All the participants in the Action were representatives of the national road research laboratories (Table 5.8), united in the Forum of European National Highway Research Laboratories (FEHRL). This means that they all had direct contacts with their national operators of PMS.

Table 5.8. Countries and institutions participating in COST Action 324 (European Commission, 1997; Ministerio de Fomento, 1998a).

Country (ISO Code)	**Institution (Acronym)**
Austria (AT)	Bundesforschungs-&Prüfzentrum Arsenal (BPFZ), Institut für Strassengerhaltung, TU Wien (ISTU)
Belgium (BE)	Centre des Recherches Routières (CRR) / Opzoekingscentrum voor de Wegengbouw (CRR/OCW)
Switzerland (CH)	Ecole Politechnique Federale de Laussanne (WPFL)
Denmark (DK)	Vejteknisk Institut (Danish Road Institute) (DRI)
Spain (ES)	Centro de Estudios y Experimentación de Obras Públicas (CEDEX)
Finland (FI)	Valtion Teknillen Tutkimuskeskus (Technical Research Centre of Finland) (VTT)
France (FR)	Laboratoire Central des Ponts et Chaussées (LCPC)
United Kingdom (GB)	Transport Research Laboratory (TRL)
Greece (GR)	National Technical University of Athens (NTUA)
Hungary (HU)	Kozlekedestudomanyi Intezet Rt (KTI Rt)
Ireland (IE)	National Road Authority (NRA)
Netherlands (NL)	Dienst Weg- en Waterbouwkunde (DWW)
Portugal (PT)	Laboratório Nacional de Engenharia Civil (LNEC)
Sweden (SE)	Vag och Transportforskningsintitute (Swedish National Road and Transport Research Institute) (VTI)
Slovenia (SI)	Zavod za gradbenistvo Slovenije (ZAG), Druzba za Drzavne Ceste (DDC)

In the project, the factors that affect pavement performance were identified and ranked, according to their level of influence, as perceived by the COST Action 324 members. It also included an inventory of the work conducted until that time in the field of pavement performance studies, comprising also an inventory of the pavement performance models that existed at that moment in the 15 countries. Moreover, COST Action 324 established a list of performance indicators that in conjunction determine the main measures of pavement performance. The indicators are:

- Longitudinal profile
- Transverse profile (rutting)
- Cracking
- Surface defects (including ravelling, bleeding, potholes, crazing and fretting)

- Skid resistance
- Texture (macro and micro)
- Structural adequacy (deflection)

The representatives from each of the 15 countries assigned values to the various factors in the rating trees reflecting the conditions in their country. The perceived importance of these indices was concluded to be different for the 3 road types (bituminous, rigid and granular) (Table 5.9), ranking the indices separately for each road type. Full details of the results and their importance can be found in Potter and Langdale (1996).

Table 5.9. Perceived relative importance of performance indicators (European Commission, 1997).

Pavement type (main structural element)					
Bituminous		Rigid		Granular	
Factor	**Normalised weighting**	**Factor**	**Normalised weighting**	**Factor**	**Normalised weighting**
Transverse profile	1,89	Surface cracking	2,13	Transverse profile	1,85
Skid resistance	1,57	Transverse profile	1,58	Structural adequacy	1,60
Structural adequacy	1,50	Longitudinal profile	1,54	Skid resistance	1,60
Surface cracking	1,29	Structural cracking	1,42	Structural cracking	1,35
Structural cracking	1,29	Skid resistance	1,29	Longitudinal profile	1,35
Surface defects	1,29	Surface defects	1,25	Surface defects	1,25
Longitudinal profile	1,18	Structural adequacy	0,79	Surface cracking	0,80

In COST Action 324, an extensive inventory was conducted of the pavement performance models that were in use in PMS in Europe for flexible and semi-rigid pavement structures in that time. Hence, the resulting inventory can be seen as the state-of-art in pavement performance modelling in the management of the national networks in Europe in that time. Table 5.10 shows the compiled deterioration models.

Table 5.10. Summary of performance prediction models (European Commission, 1997).

Country	**Longitudinal profile**	**Transverse profile**	**Surface cracking**	**Structural cracking**	**Structural adequacy**	**Surface defects**	**Skid resistance**
Austria (AT)	--	1	-	2	-	-	-
Belgium (BE)	I	I	I	I	-	I	-
Switzerland (CH)	-	-	-	-	-	-	-
Denmark (DK)	2	I	-	I	-	I	-
Spain (ES)	-	1	-	2 and I	-	-	-
Finland (FI)	1	1	-	2 and I	-	I	-
France (FR)	-	-	-	--	-	-	-
United Kingdom (GB)	-	1	-	1	1	-	2
Greece (GR)	-	-	-	1	-	-	-
Hungary (HU)	1	1	-	I	1	I	1
Ireland (IE)	-	-	-	-	-	-	--
Netherlands (NL)	-	1	1	-	-	1	-
Portugal (PT)	-	-	-	1	-	-	-
Sweden (SE)	1	2	-	1	I	-	-
Slovenia (SI)	-	-	-	-	-	--	-

Note: 1 = one model. 2 = two models. I = Condition index model that includes that characteristic.

The performance models for longitudinal profile were expressed as IRI, but also as Bump Integrator in

Denmark. Models are available for primary and secondary roads both at the network and project level and they were all empirical, based on field data.

In Denmark, for longitudinal profile analysis, the Bump Integrator, a RTRRMS device, is used. The equation employed for forecasting pavement performance is:

$$BI(AGE) = a \cdot AGE^2 + b \cdot AGE + c \qquad [5.30]$$

Where *BI*(*AGE*) is the Bump Integrator at a certain pavement age, *AGE* is the pavement age and *a*, *b* and *c* and material constants. As seen, it is second degree equation and the only independent variable is the pavement age.

In Finland, the IRI evolution model for Asphalt Concrete overlay is calculated from Eq. 5.31, and for cold mix overlay is obtained from Eq. 5.32.

$$IRI_{(t+1)} = 0{,}13 + 1{,}03 \cdot IRI_{(t)} \qquad [5.31]$$

$$IRI_{(t+1)} = 0{,}14 + 1{,}04 \cdot IRI_{(t)} \qquad [5.32]$$

Where $IRI_{(t+1)}$ is the roughness (IRI) prediction for the next year and $IRI_{(t)}$ is the measured roughness (IRI) at year *t*. These models are curious since they do not depend on traffic or environmental factor. The IRI value after one year is dependent on the IRI value at present, which is increased a 3% (AC overlays) or a 4% (cold mix overlays) and a constant quantity, 0,13 and 0,14 for AC overlays and cold mix overlays, respectively. Moreover, it can be observed, the deterioration of a cold mix overlay is a bit greater than an asphalt concrete overlay.

In Hungary, two models, depending on different factors, are employed in Hungary for IRI prediction (Eq. 5.33 and 5.4.

$$IRI = e^{(a + b \cdot AGE)} \qquad [5.33]$$

$$IRI = e^{(a + b \cdot FORG)} \qquad [5.34]$$

Where IRI is the roughness (m/km), *AGE* is the age of wearing course, *FORG* is the repetition number of the vehicles expressed in passenger car units, and *a* and *b* are constants. As seen, two different based only on one variable are employed. Each model follows an exponential model, which must be calibrated.

In Sweden, the IRI performance model employed is expressed by Eq. 5.35:

$$IRI = 1{,}51 + 4{,}8 \cdot 10^{-2} \cdot AGE + 6{,}97 \cdot 10^{-4} \cdot FI - 5{,}54 \cdot 10^{-2} \cdot W -$$

$$-1{,}29 \cdot 10^{-3} \cdot th1 + 139 \cdot D_{900} + 2{,}39 \cdot 10^{-3} \cdot TAGE \qquad [5.35]$$

Where IRI is expressed in m/km, AGE is the time since last measure (in years), FI is the freezing index (in °C · days), W is the width of the road, th1 is the thickness of bitumen bound layers (in mm), D900 is the deflection 900 mm away from a 50 kN load applied by FWD (in mm) and TAGE is the total age of the road (in years). As observed, several factors are introduced. The positive and negative influence of each factor is reflected in the signs in the equation. Time since last work, freezing index, a bigger deflection and total age of the road contribute increasing the IRI values, where higher values on bituminous layer thickness and road width reduce roughness-

The model inventory- concluded that none of the models in use in that moment in the participating countries could be suitable for use in pavement management at the European level. Consequently, new models were recommended to be developed rather than adapting existing models to pan-European conditions.

5.3.1.3. Performance analysis of road infrastructure PARIS Project

The PARIS project (Performance Analysis of Road Infrastructure) was aimed to develop robust pavement performance models for the European inference space of traffic loading and climate for flexible and semi-rigid pavements. The project took place from October 1, 1996 to September 30, 1998. The European Commission under the Transport Research and Technological Development Programme of the Fourth Framework Programme funded the 50% of the project and the rest was funded by the nineteen organisations from fifteen European countries participating in the project (European Communities, 1999).

In order to develop the European pavement deterioration models, the PARIS project, built on the results of the COST Action 324, used the data from the experiments ongoing in that time in Real-time Loading Testing (RLT) and in Accelerated Loading Testing (ALT) research, after having been brought together to a common database following the normalisation built on the theoretical basis laid in COST Action 324. The project also employed data on large parts of the road network obtained in 1997 and 1998. The project was focused in the development of models for 4 types of distress: cracking, rutting, ravelling and longitudinal roughness. Data on commented distress were collected for all test sections and was established as dependent variables. The data on explanatory variables (of those distresses) was divided in the following groups: construction, maintenance and rehabilitation, traffic and climate, with the information recorded in each group in Table 5.11.

At the PARIS project for longitudinal unevenness was modelled with time at the test sections. Although different devices were employed in that moment in Europe, those different to IRI were converted to IRI. No test section from Spain was introduced in the research. Flexible and semi-rigid pavement were analysed separately.

When modelling IRI progression, the time series plot showed for most test section a linear progression with time. Hence, a linear relationship was selected to model the annual change in IRI, of the form of Eq. 5.36:

$$y = A + B \cdot X \qquad [5.36]$$

Where y is the measured IRI value (mm/m), x is the age of the last overlay or age of the construction (years), A and B are model parameters, the intercept and the slope, respectively.

The output of each regression analysis is a set of parameter values for A and B for a test section. Fig. 5.10 shows the distributions of the change in IRI per year (i.e., the slope) obtained for flexible and semi-rigid pavement, with the median value and the 10 and 90 percentile values.

Table 5.11. Data in some of the explanatory variables in the PARIS project (European Communities, 1999)

Field	Field Name	Description
Data fields for construction data	PARIS_ID	Identification number for PARIS test section
	Pave	Type of pavement construction
	Surf	Type of surface layer
	Th_Bit	Total thickness of bituminous layers
	Th_Rig	Total thickness of rigid layers
	Th_Gra	Total thickness of granular base layer(s)
	Th_Sub	Total thickness of subgrade layer(s)
	Subgrade	Type of subgrade
	Rig_Type	Type of rigid layers
Data fields in the maintenance and rehabilitation data	PARIS_ID	Identification number for PARIS test section
	Y_Const	Year of construction
	Y_Over	Year of latest overlay
	Y_Surf	Year of latest surface layer
Data fields for traffic data	PARIS_ID	Identification number for PARIS test section
	Year	Year of survey
	Dveh	Number of vehicles per day in direction of test section
	D%Tru	Percentage of trucks in direction of test section
	Lveh	Number of vehicles per day on test section
	L%Tru	Percent of trucks on test section
	Esal	Number of 10 tonnes ESAL's per year on test section
Data fields for climate data	PARIS_ID	Identification number for PARIS test section
	T_Day	Average yearly day temperature
	W_Day	Number of days per year with a maximum temperature above 25 °C
	C_Day	Number of days per year with a minimum temperature below 0 °C
	Freez	Freezing index
	Rain	Average yearly rainfall

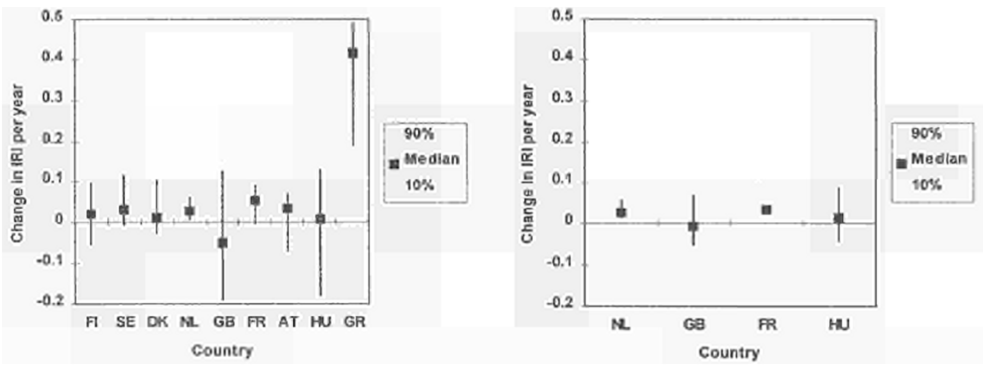


Fig. 5.10 Distribution of the change in IRI per year by country for flexible (left) and semi-rigid pavements (right) (European Communities, 1999).

From the results observed in Fig. 5.10, it was concluded that the change in IRI per year was very small at test sections. 90% of them had a change in IRI below 0,1 mm/m per year, which meant that after ten year, 90% of the test section would have only changed 1,0 mm/. The change in IRI in most of the section could be negligible and even was under the measurement error. Moreover, the slope of the model was not statistically different form zero and a negative slope could be found. In practice, this all meant that for these test sections, there was no change in IRI.

Similar low IRI incremental values were commented by other authors. Paterson (1987) stated that over a nominal 15-year interval between major maintenance activities conducted at optimum timing, roughness would increase by only about 0,1 m/km (or 3 to 5 %) per year on high-volume roads (over 1.000 veh/day) and by about 0,2 m/km IRI (or 5 to 10 %) per year on low-volume roads. Therefore, sometimes the levels of precision were inferior to these typical annual increments.

IRI progression in Greece was substantially higher than in other countries, as observed in Fig. 5.10. After analysis, it was concluded that the different deterioration rate was due to the fact that the IRI was determined from Bump integrator.

A large number of possible explanatory variables were checked to explain the differences observed in the annual IRI deterioration rate. However, no direct relationships could be found between one of these independent variables and the IRI change rate. It was observed that flexible pavement roads with low volume have a greater rate of deteriorate than other roads, but the relationship was poor and hence, no model coefficients were established. For semi-rigid pavements no data clusters were found.

The PARIS Project obtained some conclusions about the longitudinal roughness evolution:

- A linear model was suitable for describing the evolution of the longitudinal roughness with time.
- The IRI change rate was very limited for the test sections analysed.
- In flexible pavements, data from Greece showed a greater deterioration rate, which can be explained as measures were taken by means of another device, the Bump Integrator
- In flexible pavements, no direct relationships were found between IRI annual change rate and various cluster variables. It was found that usually low volume roads with thin asphalt pavement are more susceptible to change IRI with time than primary roads.
- For semi-rigid pavement no cluster variables were found.

Some explanations were tried to find for the second conclusion. Test sections of the PARIS project were located on primary roads, which are well built and hence, had low IRI values and since IRI was developed in the international study in Brazil (Sayers *et al*., 1986a) on a wide variety of road pavements, including non-paved, the range in possible IRI-values is large. From PARIS project, IRI seems not to be sufficiently sensitive to describe the small changes in longitudinal roughness on the national primary networks in Europe.

5.3.1.4. NordFoU Project – Pavement performance models

NordFoU is a cooperation programme for Nordic countries focused on the research and development in the road sector. The programme started in December 2004 when the road authorities of the Nordic countries (Sweden, Finland, Norway, Denmark and Norway) signed the agreement to coordinate their research and development effort. Four programmes were established, and one of them deals with development of deterioration models for flexible pavements, which was called NordFoU project – Pavement Performance Models. The aim of this project was to evaluate, improve and adopt existing pavement performance models to Nordic condition. The PPM project also aims at recommending harmonized data collection and delivering a practical system/tool to be employed to calculate future conditions of the pavements (Busch *et al*., 2010).

Within the project, identification of the operational pavement performance models on network level in the Nordic countries and models and systems suitable from other parts of the world were assessed (Saba, 2006; Busch *et al*., 2010). Moreover, data required for network level modelling were listed. All Nordic countries have been employing one or another form of deterioration prediction models. Generally, these models are simple, based on linear extrapolation of historic data in their Pavement Management Systems (Saba, 2006).

Denmark utilizes deterioration models both in pavement management systems and in pavement design. For roughness prediction, two models are used: one for increasing roughness and the other one for decreasing roughness. The model for increasing roughness explains how the smoothness deteriorates over time if no maintenance work is conducted (Fig. 5.11).

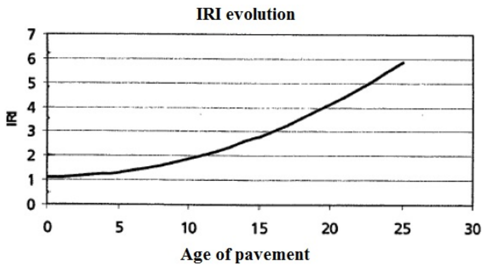


Fig. 5.11. Illustration of the model for increasing roughness in Denmark (Saba, 2006)

The advantages of this model are that it is very simple, is easy to implement and that, intuitively, the model seems to be correct. On the contrary, the model has some disadvantages. It is purely empirical and it does not provide the relationship with other factors like traffic, climate and road materials, which are said to be key factors in IRI progression. Furthermore, it is only applicable for the road conditions for which it was calibrated, the main Danish roads.

If the road section is rehabilitated with a new wearing course, the roughness will be improved and IRI decreases. The roughness of the rehabilitated section is dependent on the roughness of the old pavement since a better foundation will provide a better evenness to the new pavement (Fig. 5.12).

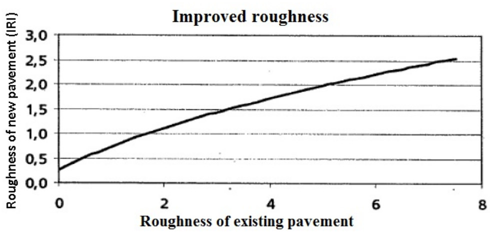


Fig. 5.12. Illustration of the Danish model for roughness of rehabilitated pavements (Saba, 2006)

The advantage of this model is that is simple and easy to use. However, the disadvantages are its difficult to intuitively assess the correctness of the model, the impossibility of explaining the relationship to parameters like traffic, climate and road materials, decisive for IRI development and that the model is only calibrated for Danish primary roads.

In Norway, IRI data is collected on most of the national and county roads every year, dividing the road in homogeneous PMS-sections, which can vary in length from a few hundred meters to several kilometres. In order to calculate the future longitudinal roughness of a road, a simple linear extrapolation of historical data was used, expressed as IRI. The 90^{th} percentile for roughness (IRI) was predicted. The main steps of the extrapolation were:

- If there exists two measurements of pavement condition that were at least one year apart after the last registered maintenance activity (that covered at least 1/3 of the section length), the first and last measurement points are deployed to calculate the straight line extrapolation.
- If the previous step could not be employed, the condition development between the two last registered maintenance actions (covering at least 1/3 of the section length) were used based on the same principle as described in previous point.
- If neither point 1 nor 2 can be employed, the average of all positive (> 0) contributions to the condition parameter values was used to estimate future linear development.

If there is no historical data, average values dependent on traffic levels are utilized (Table 5.12). The predicted annual increasing rate is between the maximum and minimum values established in Table 5.16 for different AADT values. (Saba, 2006; Busch *et al*., 2010)

Table 5.12. Maximum, minimum and average values for annual increase in roughness in the Norwegian PMS (Saba, 2006)

AADT		**< 1000**	**1000 – 3000**	**3001 – 5000**	**5001 – 15000**	**> 15000**
Roughness (mm/m IRI per year)	**Max**	0,4	0,35	0,30	0,30	0,30
	Average	0,2	0,15	0,10	0,10	0,10
	Min	0,1	0,05	0,05	0,05	0,05

IRI change rate values are low and it can be said that are in concordance with those indicated in the PARIS project. Additionally, roads with lower traffics show greater deterioration. The Norwegian model does not need any advanced regression, but it is a very simple model whose goal is to provide information about which section would become critical in the next 1-3 years. Although it was not scientifically investigated, the model was thought to be adequate for its aim.

An assessment of models and systems from other parts of the world was conducted before choosing future modelling strategy. Selected models for further investigation were:

- HDM-4 model (described in section 5.3.1.1.5). It has been employed in many countries and the constant feedback has constantly developed the software that is used. It is used in all parts of the world, but specially developed for road networks in tropic climates and in underdeveloped countries. Many of the models included in the software have been extensively reviewed, undergoing both sensitivity analysis and calibration processes.
- Mechanistic-Empirical Pavement Design Guide (MEPDG) model, from AASHTO (2008) (section 5.3.2.1). It was identified was a complex model and had unsatisfactory results in the Nordic environment for unbound materials (Huvstig, 2009). Hence, extensive calibration should be necessary to implement and suit this model for Nordic conditions.
- MnPAVE is a system for the state of Minnesota (USA). The system has site specific and calibrated information, reducing the calculation time and give accurate results. On the other hand, there are

geographical and climatic limitations and it should require calibration to Nordic conditions.

- The Surface Curvature Index (SCI), which can be found with the High Speed Deflectograph (HSD), which can measures the slope of the deflection bowl at three points. Several projects with the SCI had been developed in the Nordic countries.

As a conclusion from this analysis of existing performance models, it was decided to assess the HDM-4 model with data from typical main roads from Denmark and Sweden as a first stage in order to check the applicability and accuracy of the HDM-4 in the Nordic environment.

In 2010, a set of pavement performance models for flexible pavements and a successful MATLAB software application, based on the HDM-4 model, was developed as the end of the PPM project. The suitability of the models and the MATLAB software application resulted in further interest and in 2012 a second phase of the PPM project was initiated for the network level models under the name Pavement Performance Models 2 (PPM2) – Validation of Performance Models (Skar *et al*., 2014).

When the MATLAB software application, renamed as “Pavement Performance Prediction Nordic2 (3xP Nordic) was developed a limited number of test sections were employed, all located in the southern part of Sweden. The aim of this second phase of the project was to increase the number of test section from Sweden, Norway, Denmark and Iceland to include a wider spectrum of climate zones and materials.

Based on the findings in PPM2 the 3xP Nordic software can be used to predict the rutting and roughness for flexible pavement structures in Sweden and in Norway, and for roughness in Denmark. The models for Sweden were validated on a large number of test sections and can be used with confidence. For the other countries, limited data were available and further calibration and validation was recommended. For Iceland no calibration factors were found due to the limited data available, but values from other test sections could be employed. The split between high and low heavy traffic for Sweden and Norway was based on a limited amount of data, requiring further research with heavy traffic volumes. Calibration factor using HDM-4 model in cited countries are shown in Table 5.13.

Table 5.13. Country wise section specific calibration factors (Skar et al., 2014)

Calibration factor	**Sweden**		**Norway**		**Denmark**
	High heavy traffic (≥ 1.000)	**Low heavy traffic (< 1.000)**	**High heavy traffic (≥ 1.000)**	**Low heavy traffic (< 1.000)**	**Average**
K_{cia}	0,99	0,74	0,74	0,74	0,74
K_{pp}	0,00	0,04	0,00	0,00	0,00
K_{rid}	0,09	0,01	0,00	0,10	0,00
K_{rst}	5,66	4,85	16,00	3,40	4,82
K_{gm}	0,48	0,78	0,35	1,75	0,12
K_{gs}	1,00	1,00	1,00	1,00	1,00
K_{gc}	0,44	0,33	1,50	0,50	0,04
K_{gr}	0,69	0,83	0,35	0,75	0,04

5.3.1.5. Other empirical models

At present, empirical IRI evolution models continue to be developed for road agencies around the world. Some recent proposed models are commented.

In order to achieve an equilibrium point between mathematical complexity and ease of implementation of a IRI progression model for network-level pavement management systems, Dalla Rossa *et al.* (2017) proposed a model as a function of the initial IRI (post construction or treatment) and pavement age, taking into account the effects of climate, sugrade, treatment type, pavement type, traffic loading and functional system (urban or rural) by means of calibration coefficient. Suggested model is described by Eq. 5.37:

$$\ln\left(\frac{IRI_i}{IRI_n - IRI_i}\right) = \beta_3 + \beta_2 \cdot e^{(Time \cdot \beta_1)} \qquad [5.37]$$

Where IRI_i is the initial IRI, IRI_n is the IRI in year n, β_1 and β_3 are parameters controlling the IRI increment rate, β_2 is a parameter controlling the year in which IRI begins to increase and Time is the number of year since IRI_i. The calibration coefficients are employed to describe when the deterioration process starts (β_2) and how fast this degradation happens (β_1 and β_3) for combinations of pavement type, traffic loading level, subgrade, climate and functional systems (urban or rural area).

Alaswadko *et al*. (2017) presented a new approach to develop a roughness prediction models for sealed granular roads at network level in Victoria (Australia), by means of hierarchical multilevel models that are able to account for the correlation among time series data of the same section and capture the effect of unobserved effects. Factor included as independent variables were:

- Traffic loading (*MESA*). From traffic volume in terms of number of commercial Heavy Vehicles it was calculated the cumulative traffic loading as million equivalent standard axles (*MESA*). It used a value of 3,1 for average number of axle groups per heavy vehicle and a growth factor for roads without data.
- Initial pavement strength (SNC_0). Since it was expected that initial pavement strength, in terms of the modified structural number (*SNC*), as suggested by Paterson (1987), had a significant contribution to roughness progression, and no data were available to calculate the pavement structural number, the following approach was used to estimate the initial pavement strength in terms of SNC_0:

  $$SNC_0 = \left[0{,}55 \cdot Log_{10}\left(MESA_{DL}/120 \cdot 10^6\right)\right] + 0{,}6 \qquad [5.38]$$

  Where $MESA_{DL}$ is the cumulative traffic loading data at design life calculate with expression described in Alaswadko *et al.* (2017).
- Thorhthwaite moisture index (TMI). This parameter was introduced to consider environmental effects of each area.
- Expansion potential of subgrade soils (*SSR*). As pavement performance is greatly affected by the presence or absence of unstable materials or weak materials in the foundations. There is no data available for CBR, but the different types of soils with their level of expansive potential were introduced based on a map of soil in Victoria (Australia)
- Drainage (*DRA*). The rating of drainage systems were extracted from relevant database for the relevant period and were rated as good or poor.

Alaswadko *et al*., (2017) stated that the traditional multiple regression models assume that the deterioration of all pavement sections is due to the same process and only depends on the predictors included, but some authors considered it inappropriate for analysing panel data (nested data) because it handles a single level of variation (Greene, 2004; Anderson, 2012; Niehaus *et al*., 2013). Unobserved heterogeneity is often present in panel data-sets, especially when considering a large number of sections due to unobserved section specific variables which should be ignored. In order to overcome these problems, a useful technique is the hierarchical linear modelling, a statistical modelling approach that captures the effects of variation at multiple levels (Raudenbus and Bryk, 2002; Niehaus *et al*., 2013). The obtained equation was:

$$Ln(IRI) = 2{,}3535 + 0{,}0166 \cdot Time + 0{,}0086 \cdot MESA - 0{,}5374 \cdot SNC_0 + 0{,}0541 \cdot SSR \qquad [5.39]$$

Where Time is the time variable in years and the other variables have been explained. As seen, climate and drainage condition were not statistically significant and were discarded from the model. On average, the IRI value increase by 0,0187 mm/m (1,67%) when all other variables remain constant, which a similar value obtained from other researches.

5.3.2. Mechanistic-empirical models

5.3.2.1. Models proposed by the AASHTO

5.3.2.1.1. Introduction

As commented in section 4.4.5, it can be stated that pavement design, especially in North America, has shifted from empirical approach to mechanistic-empirical (M-E) approach (Rethenford and McDonald, 2013; Pérez-Acebo *et al*., 2018b). From the early 1960s to 1993, all the versions of the American Association for State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures were based on limited empirical performance equations developed at the AASHTO Road Test in the late 1950s (Fig. 5.13) (AASHO, 1962). There was a need for achieving the benefits of a mechanistically based pavement design procedure and it was recognized when the AASHTO Guide for Design of Pavement Structures (AASHTO, 1986) was adopted. In order to meet that necessity, the AASHTO Joint Task Force on Pavements, in cooperation with the National Cooperative Highway Research Program (NCHRP) and the Federal Highway Administration (FHWA) sponsored the development of a M-E pavement design under procedure NCHRP Project 1-37A (NCHRP, 2004).

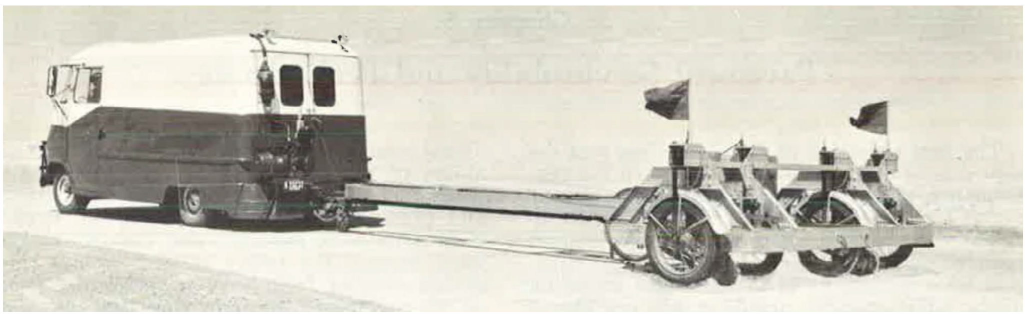


Fig. 5.13. Profilometer in the AASHO Road Test (AASHO, 1962)

Therefore, a main objective of NCHRP Project a-37A (NCHRP, 2004) was to develop a design guide that combined existing mechanistic-based models and data reflecting the current state-of-the-art in pavement design. This guide was produced in 2004 and released to the public for review and evaluation. A formal review of the products from NCHRP Project 1-37A was carried out by the NCHRP under Project 1-40A (NCHRP, 2006a). This review improvement many aspects and most of them were incorporated into the guide under NCHRP Project 1-40D (NCHRP, 2006b). Project 1-40D (NCHRP, 2006b) produced the Version 1.0 of the MEPDG software and updated the design guide document, which is known as the Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008).

The global aim of the Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008) is to provide a tool for designing of new and rehabilitated pavement structures, based in mechanistic-empirical principles. It implies that the design and analysis process calculates pavement responses (stresses, strains and deflections) and employs those responses to compute incremental damage over time. Moreover, the procedure empirically relates the cumulative damage to observed pavement distresses. The M-E based procedure of the MEPDG is shown in Fig. 5.14.

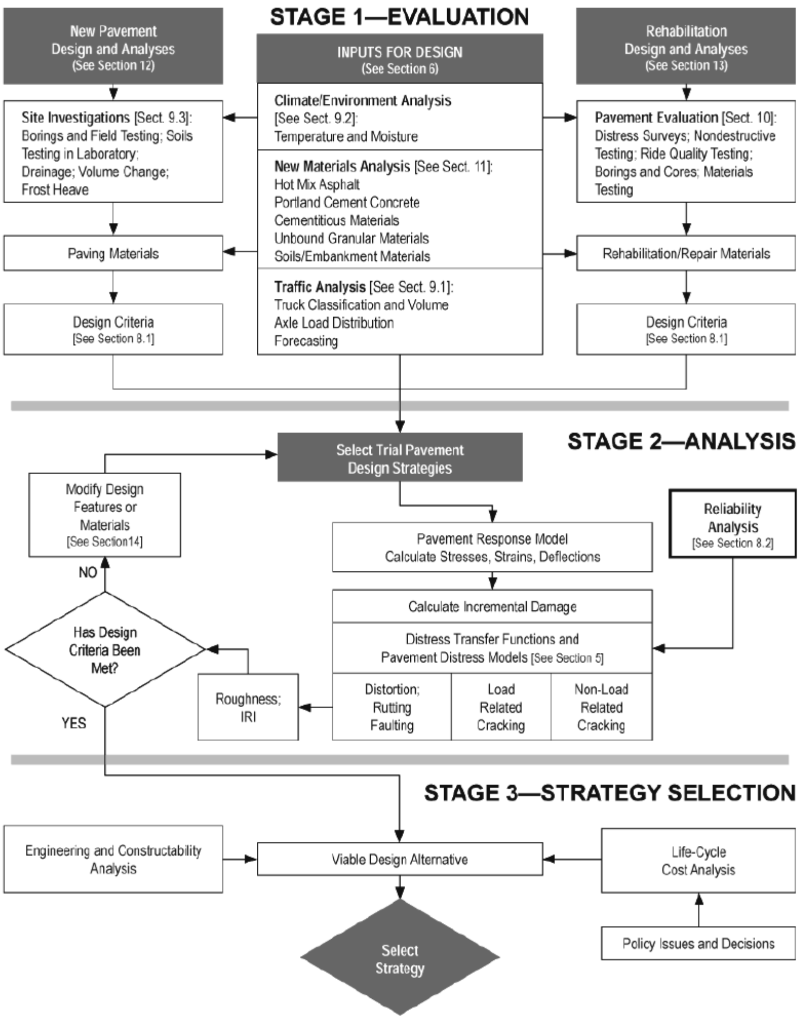


Fig. 5.14. Conceptual flow chart of the three-stage design/analysis process for the MEPDG (AASHTO, 2008).

The MEPDG can be regarded as a major change in the way pavement design is conducted. There are two fundamental differences between the Guide for Design of Pavement Structures (AASHTO, 1993) and the MEPDG (AASHTO, 2008). Firstly, the MEPDG is able to predict various performance indicators, as shown in Fig. 5.14: Secondly, the MEPDG relates directly materials, structural design, construction, climate, traffic and pavement management systems. This interrelationship between these items are exposed for hot mix asphalts (HMA), and for Portland cement concrete (PCC) in Fig. 5.15.

The procedure presented in the MEPDG is an iterative procedure, where the outputs are pavement distresses and roughness, not layer thickness, as in Guide for Design of Pavement Structures (AASHTO, 1993). The engineers must first take into account site conditions, such as traffic, climate, subgrade, existing pavement condition for rehabilitation) and proposes a trial design for a new pavement or rehabilitation strategy. Proposed trial is evaluated for adequacy against user input, performance criteria and reliability values through the prediction of distresses and roughness. If the proposed design does not satisfy the desired performance criteria at the specified reliability, it is revised and the evaluation process is repeated as many times as necessary. Hence, the engineer is involved in the design process and can consider different design characteristics and materials to meet the performance criterion for the site condition.

On the other hand, the MEPDG performance models were developed and calibrated with the Long-Term Pavement Performance (LTPP) data (Wasem and Yuang, 2013) and, hence differences can be found when introducing ‘local’ inputs, such as climate, material characteristics, traffic patterns or construction and M&R methods. As commented, models are being calibrated in different states of the USA and Canada. Moreover, a national guideline for local calibration, the Guide for the Local Calibration of the Mechanistic-Empirical Pavement Design Guide (AASHTO, 2010) was published, derived from NCHRP Project 1-40B.

5.3.2.1.2. Pavements considered in the MEPDG

The MEPDG can be deployed for flexible and rigid pavement and includes the most used pavement structures in North America. It includes transfer functions and regression equations in order to predict various performance indicators, which are regarded as key indices by many pavement management systems (AASHTO, 2008; 2015). These distresses that can be calculated by the MEPDG are listed in Table 5.14. They were calibrated using data extracted from the Long-Term Pavement Performance (LTPP) database.

Table 5.14. Indicators calculated by the MEPDG (AASHTO, 2008; 2015)

Hot-Mix Asphalt Surfaced pavement and HMA overlays	Portland cement concrete-surface pavements and PCC overlays
• Total Rut depth and HMA, unbound aggregate base and subgrade rutting • Non-load-related transverse cracking • Load-related alligator cracking, bottom initiated cracks • Load-related longitudinal cracking, surface initiated cracks • Reflection cracking in HMA overlays of cracks and joints in existing flexile, semi-rigid, composite and rigid pavements • Roughness (IRI)	For jointed plain concrete pavement (JPCP): • Mean joint faulting • Joint load transfer efficiency (LTE) • Load-related transverse slab cracking (includes both bottom and surface initiated cracks • Joint spalling (embebed into the IRI prediction model) For continuously reinforced concrete pavement (CRCP): • Crack spacing and crack width • Load transfer efficiency (LTE) • Punchouts For JPCP and CRCP: • Roughness (IRI)

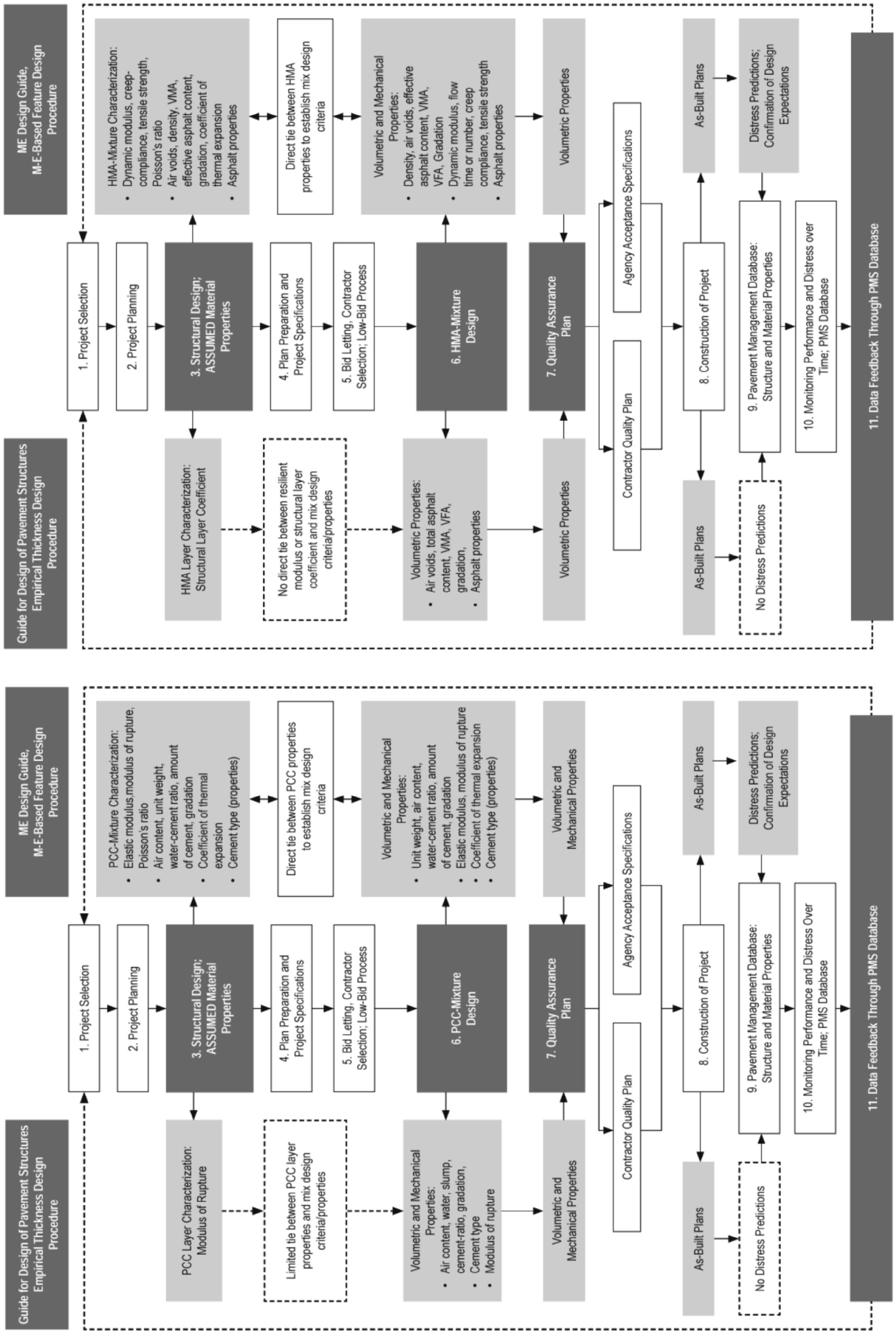


Fig. 5.15. Usual differences between empirical design procedure and an integrated M-E design system, for HMA-mixture classification (up) and for PCC-mixture classification (down) (AASHTO, 2008).

As seen, with regard to flexible pavements, the MEPDG is able to predict the performance of new and reconstructed HMA surface pavement and HMA overlays. The HMA-surface pavement types include the following types, illustrated in Fig. 5.16 and Fig. 5.17.

- Conventional flexible pavements. This group includes flexible pavements consisting of relatively thin HMA surfaces (less than 15 cm) and unbound aggregates base layers (crushed stone or gravel and soil-aggregate mixtures). Several pavements introduced in the calibration process had multiple aggregate base layers. Pavements of this type may also have a stabilized or treated subgrade layer.
- Deep strength flexible pavements. This group include flexible pavements consisting of a relatively thick HMA surface and a dense-graded HMA or asphalt stabilized base mixture placed over an aggregate base layer. They may also have a stabilized or treated subgrade layer. Several flexible pavements introduced in the global calibration process had asphalt stabilized base layers and would be defined deep strength flexible pavements.
- Full-depth HMA pavements. This group includes HMA layers placed on a stabilized subgrade layer or placed directly on the prepared embankment of foundation soil. This kind of pavements was also introduced in the global calibration process but they were fewer test sections than for conventional and deep strength flexible pavements.
- Semi-rigid pavements. This group include pavements in which HMA is placed over cementitious stabilized materials. Cementitious materials may include lime, lime-fly ash and Portland cement stabilizers. This type of pavement is also referred to as composite pavements in the MEPDG. Semi-rigid pavements were not included in the global calibration process and, hence, they are not recommended for analysis by means of the MEPDG.
- Full depth reclamation (In-place pulverization of conventional flexible pavements). This group includes cold in-place recycling of the HMA and existing aggregate base layers and hot in-place recycling of HMA. Cold in-place recycling as a rehabilitation strategy is regarded as reconstruction under the MEPDG design procedure and would be defined as a new flexible pavements. Hot in-place recycling as a rehabilitation strategy is considered mill and fill with an HMA overlay of the existing flexible pavement. The thickness of the hot in-place recycle material is considered part of the HMA overlay, as well as the thickness of the milled material. Nevertheless, full depth reclamation was not introduced in the global calibration of the MEPDG.
- HMA overlays. This group includes HMA overlays of all types of flexible and intact rigid pavements, with or without pavement repairs and surface milling. Pavement repairs and milling of the existing surface layer is taken into account by the MEPDG. The expected milling depth is an input value and pavement repairs are considered by entering the condition of the pavement prior to overlay placement. The MEPDG may also be deployed to design HMA overlays of fractured PCC slabs (break and seat, applicable to JPCP; crack and seat, applicable to JRCP; and rubblization, applicable to all PCC pavements). Nonetheless, HMA overlays of fractured PCC slabs were not introduced in the global calibration process.

As observed, the MEPDG does not forecast the loss of surface characteristics with regard to skid resistance.

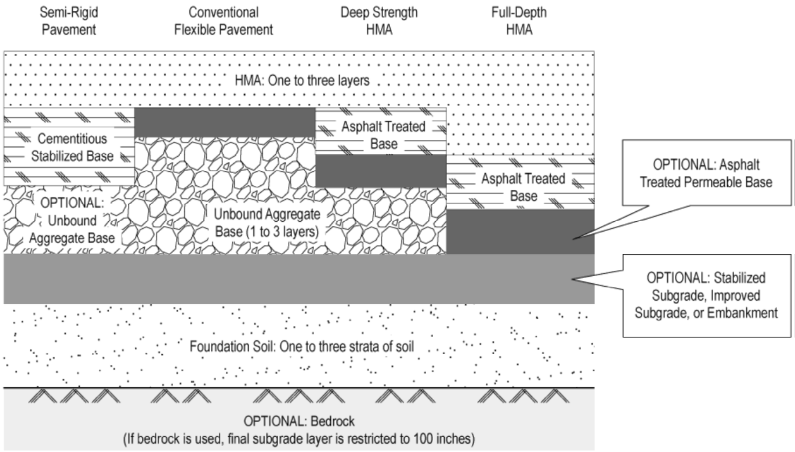


Fig. 5.16. New flexible pavement design strategies that can be simulated with the MEPDG (AASHTO, 2008; 2015).

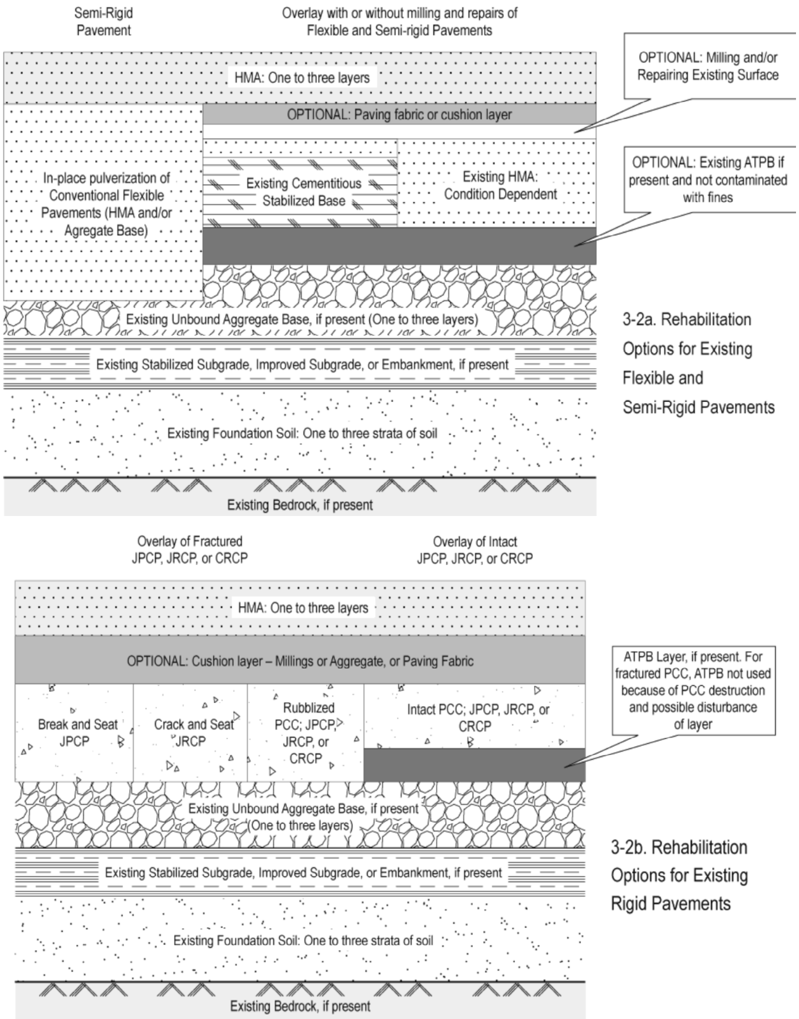


Fig. 5.17. FHMA overlay design strategies of flexible, semi-rigid and rigid pavements that can be simulated with the MEPDG (AASHTO, 2008; 2015)

5.3.2.1.3. MEPDG models for roughness

For new Hot Mix Asphalt pavement and Hot Mix Asphalt overlays of flexible pavements the equation is (AASHTO, 2015):

$$IRI = IRI_0 + C_1 \cdot RD + C_2 \cdot FC_{Total} + C_3 \cdot TC + C_4 \cdot SF \qquad [5.40]$$

Where

IRI_0 is the initial IRI after construction, in./mi

FC_{Total} is the area of fatigue cracking (combined alligator, longitudinal, and reflection cracking in the wheel path), percent of total lane area. All load related cracks are combined on an area basis-length of cracks is multiplied by 1 ft to convert length into an area basis.

TC is the length of transverse cracking, including the reflection of transverse cracks in existing Hot Mix Asphalt pavements, ft/mi

RD is the average rut depth, in inches

$C_{1,2,3,4}$ are calibration factor: C_1 = 40,0, C_2 = 0,400, C_3 = 0,008, C_4 = 0,015

SF is the site factor, which is calculated by means of Eq. 5.41

$$SF = Age^{1,5} \cdot \{\ln[(precip + 1) \cdot (FI + 1) \cdot p_{02}]\} + \{\ln[(precip + 1) \cdot (PI + 1) \cdot p_{200}]\} \qquad [5.41]$$

Where

Age is the pavement age, years

PI is the percent plasticity index of the soil

FI is the average annual freezing index, °F

$precip$ is the average annual precipitation or rainfall, inches

p_{02} is the percent passing the 0,02 mm sieve

p_{200} is the percent passing the 0,075 mm sieve

In the version of 2008 of the MEPDG (AASHTO, 2008), the site factor was calculated by means of Eq. 5.42:

$$SF = Age \cdot [0{,}02003 \cdot (PI + 1) + 0{,}007947 \cdot (\mathrm{Pr}\,ecip + 1) + 0{,}000636 \cdot (FI + 1)] \qquad [5.42]$$

Additionally, the Mechanistic-Empirical Pavement Design Guide (AASHTO, 2015) provides an equation for roughness for hot mix asphalts overlays over rigid pavements

$$IRI = IRI_0 + PCC\ C_1 \cdot RD + PCC\ C_2 \cdot FC_{Total} + PCC\ C_3 \cdot TC + PCC\ C_4 \cdot SF \qquad [5.43]$$

Where

$PCC\ C_{1,2,3,4}$ are calibration factors. $PCC\ C_1$ = 40,8; $PCC\ C_2$ = 0,575; $PCC\ C_3$ = 0,0014; $PCC\ C_4$ = 0,00825.

For calculating the rut depth, the model proposed by the MEPDG is based upon calculating incremental distortion or rutting within each sublayer. It employs the plastic vertical strain under specific conditions for the total number of trucks within that condition. It needs the rate or accumulation of plastic deformation, which is measured in the laboratory using repeated load permanent deformation triaxial test for both HMA mixtures and unbound material. Then, the relationship obtained in laboratory is adjusted to obtain the rut depth measured in the field.

The MEPDG models for cracking needs values like the tensile strain at critical locations and calculated by the structural response model, the dynamic modulus of the HMA measured in compression, the effective asphalt content by volume, the percent air voids in the HMA mixture, etc., apart from climate and calibration factors.

The MEPDG also presents the statistics resulting from the global calibration process for flexible pavement and HMA overlays of flexible pavements (Fig. 5.18) and HMA overlays of PCC pavements (Fig. 5.19), respectively. The standard error of estimate for new flexible pavements and HMA overlays of flexible and semi-rigid pavements is 18,9 in/mi and for HMA overlays of intact PCC pavements it is 9,6 in./mi. The MEPDG assumes that the standard error for HMA overlays of fractured PCC pavements is the same as for HMA overlays of intact PCC pavements. As seen, these models depend on different group of variables, including structural design and properties, climate, distresses and subgrade properties.

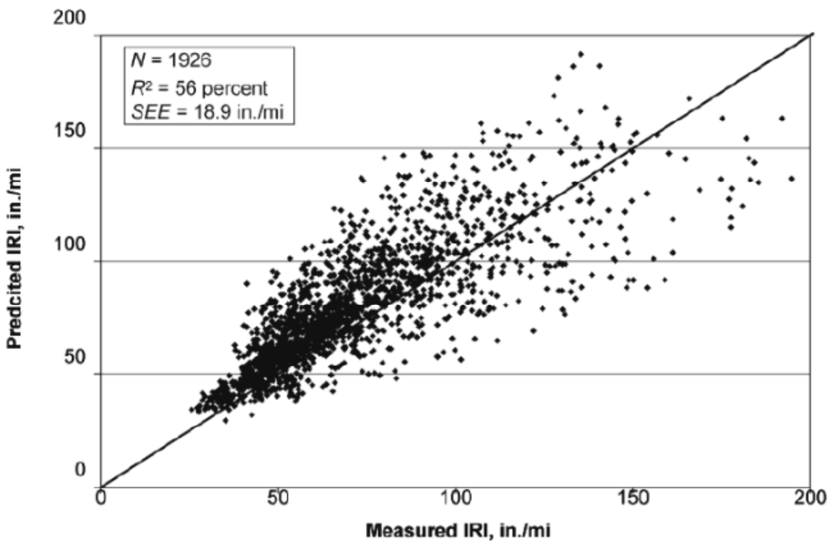


Fig. 5.18. Comparison of measured and predicted IRI values resulting from global calibration process of flexible pavements and HMA overlays of flexible pavements (AASHTO, 2008; 2015).

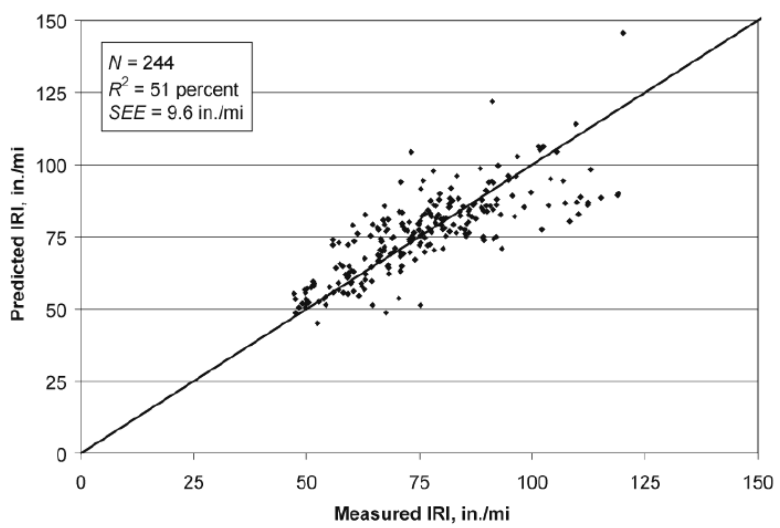


Fig. 5.19. Comparison of measured and predicted IRI values resulting from global calibration process of HMA overlays of PCC pavements (AASHTO, 2008; 2015).

As observed, these models achieve a coefficient of determination over 50 %, indicating that the models can explain more than the half of the variability of the data.

5.3.3. Markovian models

Although, most of the models try to represent the deterioration deterministically, intuitively the behaviour of pavements is probabilistic (Butt *et al.*, 1997), so probabilistic models are said to better represent the stochastic behaviour of pavements. Among probabilistic models, Markovian matrices are getting great attention. The first Markov model to describe pavement deterioration was developed for the Arizona PMS (Golabi *et al*., 1982). Moreover, the selection of an appropriate repair strategy is also an uncertain procedure. Consequently, apart from deterioration modelling with Transition Probability Matrices, they have been used to model rehabilitation and maintenance activities. Kulkarni (1984) highlighted its advantages:

- Future decision are not established but depend on how the pavement actually perform
- Actions to be taken at present can be identified. Additionally, likely actions to be taken in next few year are also identified with a high degree of probability
- It allows comparing the forecasted proportions in given conditions with the real proportions observed in the field, allowing identifying defects in construction, materials, quality control too.
- A dynamic decision model has the potential for significant cost saving by choosing less conservative rehabilitation actions what will meet the prescribed performance standards

Most of the times the transition probability matrices (TPMs) are used with road condition measurements that have a finite value range. Some examples are the PCI, which varies from 100 (excellent condition) to 0 (the worst condition) (Butt *et al.*, 1987; Tjan and Pitaloka, 2005), the Distress Rating, also from 100 to 0 (Abaza, 2016a) or from 10 to 0, as the Pavement Condition State (PCS) from Ontario, Canada (Li *et al.*, 1996; 1997). The main aspects that must be handled when developing TPMs are to decide how to calculate the elements of the matrices and the choice of using homogeneous or non-homogeneous matrices.

With regard to the first aspect, as commented in Chapter 4, the estimation of the elements of the TMPs has been usually conducted by means of two approaches: using experts' knowledge or collecting data for some time and applying an appropriate method to estimate the distress evolution rate (Pérez-Acebo *et al.*, 2017b; 2018a). In the first approach, a panel of experts is required to estimate the transition probabilities (Further description in section 5.3.6). In the second approach, Eq. 4.12 is generally used, but other techniques have been proposed in the last decades. Butt *et al.* (1987) employed the Fletcher-Powell algorithm (Sindal, 1972), a nonlinear programming approach, to determine the values of the parameters that minimized the sum of residuals, the absolute distance between the actual PCI vs. age data points and the points obtained (i.e. predicted points) by the Markov model over a 30 years data set. Mishani and Madanat (2002) developed a method to estimate the TPMs elements derived from a probabilistic model of the time spent in a specific state, including the approach to estimate its parameters. Abaza (2004) used a deterministic model based on the time durations spent in the established condition states, which were calculated from the performance curve obtained from the AASHTO model to estimate TPM elements. Tjan and Pitaloka (2005) calculated, for 10 condition states and for 10 homogeneous transitions (11 stages), a unique 10 x 10 matrix with the form of

Eq. 4.7 which has the less deviation, i.e. the minimum sum of square deviation. For 10 condition states and 10 transitions or step, 100 equations are generated after matrix multiplication. Gaussian elimination process was applied to predict a constant matrix for the 10 transitions with the least deviation. Ortíz-García *et al*. (2006) carried out an extensive analysis of three approaches in order to minimize the residuals. The first one requires the historical condition of each of the sites in the network, the second one uses the regression equation calculated from the original data and the third one needs the yearly distributions of the condition data. Results suggested the third one that, although the TPM does not always fit the regression curves so close as the others, it yielded a closer and comparable distribution to the original ones in all the tested cases.

Regarding to the second issue in TPMs calculation, it is the difference between homogeneous and non-homogeneous Markov chains. The homogeneous Markov chain is used when is assumed that the transition probabilities remain constant over time, indicating a steady-state condition. In this case, the condition state and the TPMs are related to each other by means of Eq. 4.11, allowing calculating the condition state in time *t* if the initial condition state is known. Nevertheless, as stated in section 4.4.2.2 the transition probabilities might change during service life of pavement due to changes in traffic volumes, environmental factors or subgrade strength (Li *et al.*, 1996; Golroo and Tighe, 2009). A non-homogeneous discrete-time Markov chain is represented in Eq. 5.44:

$$A_t = A_0 \cdot \left(\prod_{i=1}^{t} P(i) \right) \quad [5.44]$$

Where $P(i)$ is the probability matrix for transition i, A_0 is the initial state vector, A_t is the condition state vector in step t (or after t transitions).

Incorporating a different TPM for each transition provides a much more accurate prediction of future state (Li *et al.*, 1996; Hong and Wang, 2003, Abaza, 2006; Abaza and Murad 2009). Nonetheless, it requires much more intensive work as data for all the stages is needed. Solutions to overcome this problem have been proposed. For the analysis of the 30 years-service life of a pavement, Butt *et al.* (1987) proposed 5 zones in which the same TPM was used for a period of time of 6 years. Thus, a staged-homogeneous TPM was established for the period of 6 years and only 5 different TPMs were required. Li *et al.*, (1995; 1996) proposed non-homogeneous TPMs by means of Monte Carlo simulations. Abaza (2016a) indicated that the deterioration, i.e. the transition probabilities generally increase because of the progressive increase in traffic loading and the gradually degraded pavement structure. Hence, Abaza (2016b) proposed a simplified methodology based on a factor, C_i, which can be applied to present TPM to calculate transition matrices associated with staged-periods. The C constants, which are always larger than 1, since deterioration always increases, are estimated fro minimising the sum of squared error between predicted and observed values. This approach assumes that all TPM associated with a particular staged-time period increase in the same proportion. Alternatively, reliable C constants can be estimated from the calibration process if condition data are available over an analysis period of *t* transitions. Abaza (2016a) also presented a technique that only needs two steps of pavement distress evaluation. Moreover, an empirical approach is suggested for obtaining the TPMs in non-homogeneous Markov chains according to the two major factors that cause the transition probabilities increase over time: the increase in traffic loading, represented by ESAL applications and the decline in pavement structural capacity, which may be represented by the Structural Number (Abaza, 2017a).

On the other hand, TPMs have been also developed to assess the improvement obtained after rehabilitation or maintenance work and its subsequent different performance (Wang *et al*., 1994). Consequently, elements below the main diagonal may be different from zero. Dean and Baladi (2013) proposed Treatment Transition matrices (TTMs) where improvements after different treatments are indicated for IRI, rut depth, longitudinal cracking, transverse cracking and alligator cracking. Abaza (2017c) suggested an Empirical-Markovian model able to predict the overlay design thickness for flexible pavement based on the structural capacity of the original pavement, the annual traffic growth rate, the rehabilitation scheduling time and two calibration constants. Amador-Jiménez and Afghari (2015) analyzed the effect of surface treatments on bituminous pavements by means of TPMs. It was demonstrated that pavements with IRI values smaller than 1,4 m/km did not seem to benefit from surface treatments. Those with IRI values higher 1,66 m/km gained from 6 to 8 years of additional life. Moreover, Mandiartha *et al.* (2017) developed a process to evaluate the effectiveness of routine maintenance by means of TPM analysis based on historical costs and IRI progression data. The effectiveness is measured observing if the sections remains in the same condition state or move to the next worse state. Finally, Abaza (2017b) also presented two empirical Markovian based models to predict the transition probabilities associated with rehabilitated pavements: the first one is able to predict the staged-homogeneous transition probabilities used in a staged-homogeneous Markov model and the second one the non-homogeneous transition probabilities required in a non-homogeneous Markov model.

Recently, a new methodology for modelling IRI evolution in flexible and bituminous pavements has been proposed by means of transition probability matrices, which can be used in developing countries with data collected during a few years. Pérez-Acebo *et al.* (2017b; 2018a) developed transitions matrices for the national road network of the Republic of Moldovan with the IRI data collected twice a year in 2013, 2014 and 2015. Since the independence of Moldova in 1991 from the Soviet Union, no new roads have been constructed and the existing network have been maintained and rehabilitated in different moments and its performance have been evaluated since 1992 by means of IRI. From 2000 to 2012, the State Road Administration (SRA), a corporative road agency responsible depending on the Ministry of Transport and Road Infrastructure (MoTRI) has systematically collected roughness data covering 41 national roads, consisting of average IRI per km by road and year. Nonetheless, there are many gaps in the data set, with years without data or poorly represented. Since 2013, the SRA have followed a standardized IRI data collection framework in order to develop a complete national pavement management system, collecting data on National roads (Magistral and Republican, the most important ones) twice a year, in spring and in autumn.

The performance models were created for flexible (bituminous) and rigid (Portland concrete cement) pavements Pérez-Acebo *et al*. (2017b; 2018a). Family pavements were adopted according to the technical categories in which roads in Moldova are classified, based on Average Annual Traffic (ADT). Some pavements samples were available but they do represent neither the road network nor the road where they were taken. Consequently, the only possible classifications for roads were surface type and the technical categories, shown in Table 5.15. TPMs for technical categories 1 to 4 were established. It is assumed that, for the same inputs (ADT), similar or equivalent outputs (pavement layers and proportional thickness) were employed. IRI values were classified into six condition states (CS) of 2 m/km width, from the best one (IRI $\leq$ 2), until the worst one (IRI > 10).

Table 5.15. Technical Categories of Public Roads in Moldova (Minsterul Dezbolarri Regionale si Contructiiilor 2015)

Technical category	Average Daily Traffic (in physical vehicles)	Recommended road	Width (m)	Functional destination
1-a	> 16000	Highways	15 - 30	Roads with highly intense traffic, designed exclusively for auto-vehicles circulation
1-b	8001–16000	Express roads	15 - 30	Roads with intense traffic
2	3501–8000	Two lane roads	7,5	Roads with medium traffic
3	751–3500	Two lane roads	7	Roads with reduced traffic
4	200-750	Two lane roads	6	Roads with very low traffic
5	< 200	Two lane roads	-	Secondary roads

The main innovation of these researches is the introduction of a shorter step or cycle time for transition, only 6 months, instead of the traditional cycle time of 1 year (Butt *et al.*, 1987; Li *et al.*, 1997; Ortiz-García *et al.*, 2006). As the period in which IRI data were collected was short, from 2013 to 2015, to consider a greater number of transitions, a six-month cycle was adopted. However, using such a short step time implies that variations not related to roughness degradation but to seasonal variations could bias the results. Consequently, some assumptions had to be taken. Small improvements observed between two data collection were not related to roughness progression but to seasonal variations. Following previous research about seasonal variations (Karamihas, 1999; Karamihas *et al.*, 2001; Perera and Kohn, 2002; Baus and Henderson, 2008; Chang *et al*., 2010; Johnson *et al*., 2010) a maximum improvement of 0,50 m/km and 0,40 m/km was adopted for flexible and rigid pavements, respectively. Greater improvements would be considered as if a maintenance works had carried out in that section. The probabilistic models developed only consider rehabilitated pavements. Since no new roads have been constructed in Moldova since 1991, all existing roads have been rehabilitated in any moment in the last two decades. Therefore, pavements with varying time since last major rehabilitation were considered. Pavements with improvements of 4 m/km or more were considered as rehabilitated during the analysis period. This is the normal case in Moldova since normally pavement sections are nor rehabilitated until an IRI value of 5,0 or 6,0 is achieved. Therefore, if a section had an improvement in IRI between 0,5 (or 0,4 for rigid pavements) and 4 m/km, a maintenance activity was considered, and it was no longer considered. If an improvement over 4 m/km was recorded, it meant a rehabilitation and that transition was not introduced in the modelling (in order not to develop matrix elements under the main diagonal, only pavement deterioration was evaluated) but subsequent transitions were considered, as they represent the variation of a rehabilitated section, similar to the rest of sections in the network. For transition elements calculation, Eq. 4.12 was employed.

Developed models showed some interesting result. Firstly, obtained matrices showed that sections on a condition state cannot only remain in the same condition or shift to next one, as it is usually assumed in the TPM literature (Butt *et al*., 1987; Costello *et al*., 2005; Abaza and Murad, 2007; Abaza and Murad, 2009). However, in Moldovan roads, an important degradation rate occurs, and if more than 1 % of the sections in that condition had a condition state drop of 2 condition states or more, it was introduced in the matrix. These models were proposed to be used by other countries or road administrations in similar circumstances: a network with no new roads for a long time, M&R works performed in different moments, with pavement structure is unknown and without useful data from previous years. Just with a short period of systematic data collection, it is possible to develop pavement performance models (Pérez-Acebo *et al*., 2017b; 2018a).

5.3.4. Bayesian models

As commented in section 4.4.2.1, Bayesian modelling is a probabilistic approach that offers objectivity in expert panel evaluations and provides reliable results for a small database (Osorio-Lird *et al.*, 2017). It comes from the Bayes theorem, a probabilistic relationship that combines prior knowledge with observed data to create an adjusted probability of any event (Amador-Jimenez and Mrawira, 2011a; 2011b). Hence, Bayesian models combine prior knowledge of certain event probabilities (which can be obtained from expert criteria or results from previous experiments) with observed data of the response (likelihood) in order to produce an adjusted expression of the event probabilistic distribution (known as the posterior). The process is embedded in a probabilistic expression in which the mean is represented by the classical mechanistic equation and the variance is presented by the variability of the model predictions for any specified confidence interval.

The advantages of Bayesian approach is the ability to incorporate uncertainty and the possibility or introducing expert opinions to replace historical data when historical data are not available (Amador-Jimenez and Mrawira, 2011a; 2011b).

Some Bayesian models proposed for IRI evolution assessment are commented. Liu and Gharaibeh (2014) developed a new probabilistic approach for estimating the IRI values of asphalt pavements rehabilitated with thin HMA overlay. The model consisted of two related components. The first one was a set of ANN able to predict the roughness evolution in pavements without treatments. The second one was a set of Bayesian regression models for predicting the IRI decrease after rehabilitation with thin HMA overlay. In other words, the Bayesian regression model forecasts the probability distribution for the IRI reduction. This approach was selected due to the uncertainty and lack of clear patterns in the data which made deterministic models unviable. Proposed formula (Eq. 5.45) depends on the overlay acceptance quality characteristics (AQC), site conditions and treatment age:

$$\log(\Delta IRI) = \beta_0 + \sum_{i=1}^{l} \beta_j \cdot AQC_i + \sum_{j=1}^{m} \beta_j \cdot SF_j + \beta_{l+m+1} \cdot age + \varepsilon \qquad [5.45]$$

Where

ΔIRI is the reduction in IRI due to the thin HMA overlay
β is the parameter coefficients
SF is the site factor, which includes annual average air temperature, average annual rainfall, freezing index and average annual daily truck traffic
Age is the treatment age
ε is the error term, subject to a normal distribution
l is the number of AQCs
m is the number of site factors considered in the regressions.

The natural logarithm is employed to avoid predicting negative reduction in IRI. Moreover, since no information was available about the values, ranges or densities of the predictors of ΔIRI, noninformative priors were utilized, as suggested by Hoff (2009) and Congdon (2001).

Another example of Bayesian modelling was carried out by Amador-Jimenez and Mrawira (2011a) using a multi-level Bayesian modelling to calibrate mechanistic model parameters from historical data. The model is able to estimate the parameters from the observed data and expert criteria even in cases of missing data points. The aim was to demonstrate that a multi-level Bayesian regression can be employed to estimate a mechanistic deterioration model and characterize the uncertainty. It was adopted as mechanistic formulation part of the simplified roughness model developed by Paterson and Attoh-Okine (1992) for HDM-III, without surface distress information (Eq. 5.46). This equation relates the roughness deterioration to five casual factors, the environment, the surface defects, the structural deformation, traffic loading and the time since last treatment.

$$RI_t = e^{mt}\left[IRI_0 + a\cdot(1+SNC)^{-5}\cdot NE_t\right] \quad [5.46]$$

Where RI_t is the roughness at pavement aged t, IRI_0 is the initial roughness, NE_t is the cumulative ESALs at age t (million ESAL/lane), t is the pavement age since last rehabilitation or construction (years), m is the environmental coefficient, following Table 5.4, SNC is the structural number modified for subgrade strength. IRI_0 and a coffecients were obtained from fitting the model to observed data.

In the New Brunswick network information about pavement deflections, traffic loading and environmental data was available. However, layer thickness data and surface distress were no not available and hence it could not calculate modified structural numbers (*SNC*). Thus, Eq. 5.46 was modified to the data availability; the modified structural number was replaced by the AREA deflection basin parameter, which is expected to affect coefficient of the model. The initial IRI was assumed to be a stochastic variable, β, to be estimated by the proposed model (Eq. 5.47).

$$RI_t = e^{m\cdot age}\cdot\left(\beta + \alpha\cdot AREA^{-5}\cdot NE_t\right) \quad [5.47]$$

5.3.5. Artificial Neural Networks

Artificial Neural Networks (ANNs) have become a very powerful tool for modelling phenomena in which several variables are involved (Simpson *et al*., 1994). Many pavement performance models have been developed by means of ANNs for various pavement condition indicators (Lou *et al*., 2001; Yang *et al*., 2003; Amin and Amador-Jiménez, 2016; Kirbas and Karasahin, 2016; Amin and Amador-Jiménez, 2017). As commented in Chapter 4, the ANNs are usually composed of an Input layer, in which there are as many nodes as the independent variables considered in the analysis. Some ANNs developed for pavement roughness evolution modelling are commented. The interest is focused in the Input introduced in the ANNs, i.e. the independent variables that affect the dependent variable, in this case, the pavement roughness, generally expressed in IRI units (m/km, mm/m or ft/mi). They independent variables represent the factors that impact the roughness progression, although the way of this impact is unknown.

La Torre *et al*. (1998) proposed a multi-layer feed forward artificial neural network using part of the Long Term Performance Program (LTPP) database. A multi-layer ANN implies that there is no connection between nodes of the same layer, only between nodes of different layers. In this approach, 2 layer network were established and hence, there is only one hidden layer. There were 11 independent variables: asphalt concrete (AC) layer thickness, AC backcalculated elastic modulus (E) value, Unbound layer thickness, unboung layer backcalculated E value, subgrade backcalculated E value, annual number of days with temperatures over 90 °F, freeze index, annual precipitation, average annual ESALs, age of the pavement at the first IRI observation (*Age0*) and IRI measured for the pavement section at *Age0*.

Roberts and Attoh-Okine (1998) compared two ANN models to predict flexible pavement roughness for the road network in Kansas (USA) by introducing distress characteristics and traffic information. The variables introduced were 11 of 5 types: rutting (*RT*); fatigue cracking, which is divided into 4 variables: hairline alligator cracking (*FC1*), alligator cracking and pieces not removable (*FC2*), alligator cracking and pieces are loose and removable (*FC3*), and pavement with shoved forming a ridge of material adjacent to wheelpath); transverse cracking, divided into 3 variables: *T1*, *T2* and *T3* according to cracking severity; equivalent axle loads (*EAL*).and the International Roughness Index, the dependent variable (Fig. 5.20a).

The first model is the most common type, which can be described as a feedforward, fully connected backpropagation network having a linear activation function with supervised learning. The activation function is commonly referred as the dot or inner product function (Eq. 5.48) and the transfer function was the sigmoid function (Eq. 5.49).

$$f(X,W)=y=\sum_{i=1}^{n}W_i \cdot x_i + W_0 \qquad [5.48]$$

$$g(y)=\frac{1}{1+e^{-y}} \qquad [5.49]$$

The model for comparison was called the quadratic function ANN model, which develops its own network architecture using the specified activation function by means of a self-directed trial-and-error process. It grows to fit the problem, without requiring specifying the number of layers or nodes (Fig. 5.20b). The activation function is quadratic (Eq. 50) and the transfer function is linear (Eq. 5.51).

$$F(U,V,A,\ldots,F)=y=A+B\cdot U+C\cdot V+D\cdot U^2+E\cdot V^2+F\cdot U\cdot V \qquad [5.50]$$

$$g(y)=y \qquad [5.51]$$

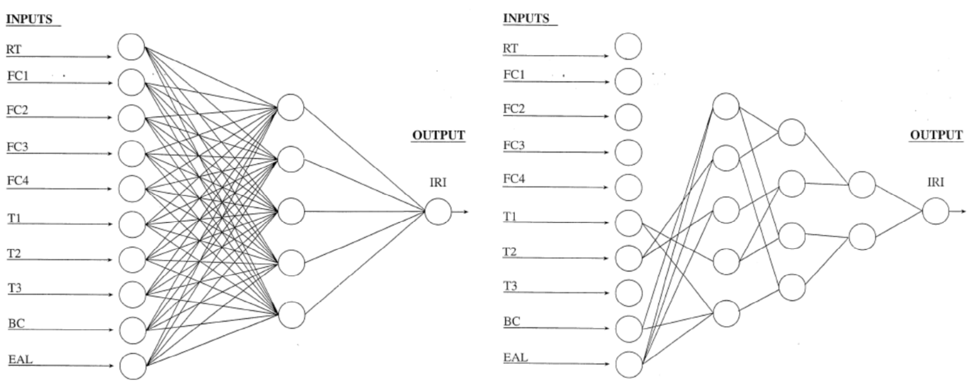


Fig. 5.20. ANN Models; a) Usual model, b) Quadratic (Roberts and Attoh-Okine, 1998).

The quadratic function ANN model was observed to perform better on the sample of patters than the dot product ANN model, suggesting that further research must be done, since the dot product type of ANN is the most widely used.

Ziari *et al*. (2016) analyzed an IRI prediction model for the short (1 and 2 year) and long term by means of ANN and compared with a group method of data handling (GMDH) method. Considered pavement sections were composed of asphalt concrete (AC) pavement on granular base and data were obtained from the LTTP database. Selected sections were not rehabilitated or maintained, with the aim of predicting roughness evolution. Introduced independent variables were classified in groups: **traffic condition** (equivalent single axle load (ESAL), average annual daily traffic (AADT), average annual daily truck traffic (AADTT)); **climate changes** (annual average precipitation (AAP); annual average temperature (AAT), annual average freeze index (AAFI); and **materials and structure of the pavement** (surface thickness (ST); pavement thickness, including surface, base and sub-base thickness (PT); Pavement age (age)). Results showed that GMDH models were not able to predict pavement performance in the short or long term. In contrast, the ANN models, with one hidden layer, predicted the pavement condition accurately in both short and long term.

5.3.6. Subjective models

Subjective models are also employed for pavement performance modelling. These models are based on experience and knowledge acquired by experts managing road networks. There are different techniques to helping in "catching" the experience in an organised way. This kind of models overcomes the lack of complete historical data for a highway administration.

Any of the models proposed so far can be used for developing a subjective model. For instance, Adedimila *et al*. (2009) utilized Transition Probability Matrices (TPMs) for capturing pavement deterioration in Nigeria. The aim was to estimate the type and timing of maintenance and rehabilitation works for single and multiple years. Due to the absence of historical records of IRI in the country, expertise of the highway engineers in the Ministry of Transportation was taken into account. For TPMs 7 condition states with a constant interval of 2,0 m/km were established, from "excellent" condition (0,0 < IRI < 1,9), to "failed" condition (IRI < 12,0).

Additionally, Soncim *et al.* (2017) developed transition probability matrices obtained from the knowledge and experience from experts in pavement management from road network in the state of Bahia, in Brazil. 20 condition states were introduced. The first one corresponded to an IRI value between 1,50 and 2 and the rest were subdivided into 0,5 km/m intervals. Matrices were obtained by means of the method of Delphi, a formal interviewing method. It consists of an agreement about a particular issue made by a group of experts and it aims to reduce the error range when making a decision. The matrices were produced as a function of traffic and climate. Two traffic variables were considered (medium and low volume) and two variables for climate conditions (sub-humid to humid and semi-arid to dry). After the experts' survey, 11 TPMs were obtained for each combination of conditions of traffic and climate, resulting in a total of 44 matrices.

As seen, subjective models can be used in developing countries where no historical data are available.

5.4. Factor affecting skid resistance evolution

5.4.1. Introduction

In chapter 3, some of the factors affecting the skid resistance were discussed. More precisely, according to Table 3.2, factors comprised within the groups “Vehicle operating parameters” and “Tyre properties” were analysed in Chapter 3. Those factors affect the different friction values that can be obtained if different skid resistance or texture testers are employed. For example, among vehicle operating parameters, vehicle speed, slip speed were underlined as main factors due to their importance when elaborating the curves of the friction models. It was demonstrated that different performances were obtained if those values vary. It was also remarked the difference between longitudinal and sideway friction. On the other hand, tyre or slider properties were also discussed, specifically if they were rubber or smooth tyre.

Nevertheless, when carrying out a test employing a specific device, the only factors that vary are the surface characteristics and the environment, remaining constant the rest of variables. The pavement surface is wetted with a previously standardized amount of water and friction is measured by means of a standardized measuring wheel (Kogbara *et al.*, 2016; Pérez-Acebo *et al.*, 2017a). As stated in Chapter 3, surface characteristics are characterized by the asperity of the pavement surface (Hall *et al*., 2009), usually divided in microtexture and microtexture, according to the levels established by PIARC (1987). Both levels depend on the material properties of the materials that compose the pavement surface.

5.4.2. Factors affecting the texture of pavements

Factors that affect directly the pavement texture are related to the aggregate, binder and asphalt mix properties and the post-placement treatment. While the coarse aggregate type principally affects microtexture, macrotexture is said to be affected by the maximum aggregate size, the coarse and fine aggregate types, mix binder content and viscosity, mix gradation and mix air content, as resumed in Table 5.16. Resulting texture from aggregate and asphalt mixes properties influence friction. In next sections, aggregate properties (5.4.3.) and asphalt mixture properties (5.4.4.) are analyzed separately.

Table 5.16. Factors affecting asphalt pavement microtexture and macrotexture (Henry, 2000; Rado, 1994; PIARC, 1995, AASHTO, 1976, Hall et al., 2009).

Factor	Microtexture	Macrotexture
Maximum aggregate dimensions		X
Coarse aggregate types	X	X
Fine aggregate types		X
Mix gradation		X
Mix air content		X
Mix binder		X

5.4.3. Aggregate properties affecting friction

As stated previously, the aggregate properties are a key factor on microtexture and, hence, the provided friction. More specifically, the aggregate polishing properties is said to be the most important characteristic (Ali *et al*., 1998). However, other properties, described below, also influence the friction.

5.4.3.1. Hardness and mineralogy

Hardness and mineralogy of aggregates largely influence the wear characteristic of the aggregate. The hardness of an aggregate is generally ranked by the Mohs hardness scale, which ranks from 1 (the lowest value of hardness) to 10 (the highest value). It evaluates the resistance of a mineral to be scratched by means of the Mohs hardness test. The highest levels of friction are provided by aggregates with hard, strongly blinded, interlocking mineral crystals embedded in a matrix of soft minerals (Henry, 2000; Hall *et al*., 2009). A combination of hardness values over 6 for hard minerals and values in the range 3 – 5 for soft minerals is recommended for the coarse aggregate in order to provide good frictional performance related to a balance of wear and polish resistance (Dahir and Henry, 1978).

Petrographic analyses are employed for determining the mineralogy, implying the optical and scanning electron microscopy (SEM). A 50-70 % content of hard minerals, an average crystal size of 200 µm and hard grains with angular tips are estimated to generate the highest coefficient of friction (Hall *et al*., 2009).

5.4.3.2. Shape, texture and angularity

These characteristics are key issues for the micro and macrotexture. While sharp and angular coarse aggregates interlock and provide a higher macrotexture depth, flat and elongated particles tend to be horizontally oriented and, hence, provide lower macrotexture depth. Consequently, sharp and angular aggregates are preferred. Uncompacted void content of coarse aggregate and fractured-face particles test are two of the most employed test for aggregate shape, texture and angularity characterization in the USA. A complete list of the most common test deployed can be found in Masad *et al*. (2007).

Some of the tests for the three parameters listed in Masad *et al*. (2007) are influenced by changes occurring during the tests and do not distinguish between changes in two parameters. Consequently, the Aggregate Imaging System (AIMS) was recommended for analysis of coarse and fine aggregates (Kogbara *et al*., 2016). AIMS is able to provide the distribution of shape, angularity and texture separately in an aggregate sample. It uses a mechanism to capture images at different resolutions based on a particle size and measure the three dimensions of aggregates (Masad *et al.*, 2007). Moreover, it provides a quick and accurate quantification of the influence of polishing on texture (Luce *et al*., 2007). It was found a good correlation between microtexture values from BPT with AIMS analysis but a very low correlation between MTD values from sand patch test and the ETD results from AIMS analysis (Araujo *et al*., 2015).

In Spain, the following test methods are employed to characterize these aggregate properties:

- Shape: UNE-EN933-3 "*Ensayo para determinar las propiedades geométricas de los áridos. Parte 3: Determinación de la forma de las partículas. Índice de lajas*." [Test for geometrical properties of aggregates. Part 3: Determination of particle shape. Flakiness index] (AENOR, 2012)
- Angularity: UNE-EN 933-5 "*Ensayo para determinar las propiedades geométricas de los áridos.*

Parte 5: Determinación del porcentaje de caras de fractura de las partículas de árido grueso." [Test for geometrical properties of aggregate. Part 5: Determination of percentage of crushed and broken surfaces in coarse aggregate particles]. (AENOR, 1999).

5.4.3.3. Soundness

Soundness is the capability of the aggregate to resist degradation due to climatic or environmental effects like wetting and drying, freezing and thawing, etc. The usual test method is the magnesium sulphate soundness test, which evaluates the maximum weighted average loss percentage of the aggregate after a specified number of hydration-dehydration cycles. The higher loss percentages, the more unsound aggregates are. Usually maximum values between 10 % and 20 % are estimated for good aggregates with good frictional performance (Kandhal *et al*., 1997; Kandhal and Parker, 1998; FHWA, 2005). In Spain, this test is specified under the UNE-EN 1367-2:2010 standard (*Ensayos para determinar las propiedades térmicas y de alteración de los áridos. Parte 2: Ensayo de sulfato de magnesio)* [Test for thermal and weathering properties of aggregates. Part 2: Magnesium sufate test] (AENOR, 2010a). It establishes a maximum of 15 % for roads with freezing in winter.

5.4.3.4. Abrasion or wear resistance

Abrasion or wear resistance indicates the aggregate resistance to mechanical degradation. Micro-Deval and Los Angeles (LA) test are the most deployed method to evaluate it. Micro-Deval test is reported to be a better indicator of the potential for aggregate breakdown, but LA is widely employed with good success (Folliard and Smith, 2003; Kandhal and Parker, 1998). For good frictional resistance, percent of loss below 17-20 for Micro-Deval test and below 35-45 in LA abrasion test are indicated (Kandhal and Parker, 1998; Wu *et al.* 1998; Prowell *et al.*, 2005; FHWA, 2005). Consequently, the coefficient of friction is significantly affected by the grading of aggregates employed in the bituminous mixtures (Zhang *et al.*, 2014). In Spain, the Los Angeles test is employed and is specified under the UNE-EN 1097-2 standard (AENOR, 2010b), and established values for hot mix asphalts are shown in Table 5.17 and for discontinuous or porous asphalt mixes in surface layers, values are shown in Table 5.18. (Ministerio de Fomento, 2015).

Table 5.17. LA test values for aggregates in hot mix asphalts in Spanish standards, PG-3 (Ministerio de Fomento, 2015).

Layer type	**Heavy traffic category**				
	T00 and T0	**T1**	**T2**	**T3 and shoulders**	**T4**
Surface	≤ 20	≤ 20	≤ 20	≤ 25	≤ 25
Binder	≤ 25	≤ 25	≤ 25	≤ 25	≤ 25 (*)
Base	≤ 25	≤ 25	≤ 30	≤ 30	-

(*) In service roads

Table 5.18. LA test values for aggregates in PA and discontinuous mixes in surface in Spanish standards (Ministerio de Fomento, 2015).

Mixture type		**Heavy traffic category**			
		T00 and T0	**T1 andT2**	**T3 and shoulders**	**T4**
Discontinuous	**BBTM A**	≤ 15	≤ 20	≤ 25	≤ 25
	BBTM B	≤ 15	≤ 15	≤ 25	≤ 25
Porous asphalt	**PA**	≤ 15	≤ 20	≤ 25	-

5.4.3.5. Polish resistance

Aggregates on pavement surface must be polish resistant in order to keep the established level of available microtexture during their design life (Austroads, 2011a). The polish resistance of an aggregate is defined as the capability to retain its microtexture after being grinded and sheared by repeated traffic loadings (Kogbara *et al.*, 2016). This idea of "*retained skid resistance*" has been traditionally underlined in all the definitions, expressing that the aggregate has been polished to a certain degree. This approach comes from the accelerate wear machine developed by the British Transport and Road Research Laboratory (TRRL) in 1952. In the test, the aggregate samples are stuck by a mortar to the rim of a vertical wheel and then, a rotating pneumatic tyre polishes the samples. After the polish stage, a pendulum friction test was carried out, and the value was recorded as "roughness number". From 1957, it has become the reference for the measurement of the polish resistance (Hosking, 1992). The modern and present version of this approach is the Polished Stone Value (PSV) test, further described in subsection 5.4.3.5.1.

One of the problems that road agencies over the world experienced when using the PSV method was the difficulty to find the reference stone, grit and tyres specified by BSI (2009). Hence, some agencies replaced reference materials by local materials, with similar properties (Austroads, 2011a).

5.4.3.5.1. Polished Stone Value (PSV) test

The test comprises two stages. Aggregates are placed into curved moulds by means of an epoxy binder. These moulds are bolted around the road wheel of the accelerated polishing machine (Fig. 5.21), including two control samples (made of the UK reference aggregate), are then subjected to wet polishing action. It includes three hours of polishing action with a coarse emery abrasive by means of a conditioned wheel for coarse polishing; and another three hours of polishing by means of a fine emery abrasive, using another conditioned wheel for fine polishing. After the six-hour process of polish, a British Pendulum Tester test is performed to measure the PSV. A higher value of PSV indicates a better polish resistance.

The British Standards of the PSV procedure are specified in the BSEN 1097.8:2009. There is also an ASTM procedure for the PSV, described in the ASTM D3319-11 (ASTM, 2011) and in the ASTM E303-93 (ASTM, 2013). In Spain, the method is defined in the UNE-EN 1097-8 *Ensayos para determinar las prestaciones mecánicas y físicas de los áridos. Parte 8: Determinación del Coeficiente de Pulimiento Acelerado* [Test for mechanical and physical properties of aggregates. Part 8: Determination of the polished Stone value] (AENOR, 2010c). However, some issues about PSV test must be underlined (Austroads, 2011a; Ramírez Rodríguez, 2017):

- It is a slow and time consuming test, which requires experienced operators.
- The selection of aggregates is a vital point, affecting enormously the results.
- It does not simulate the bituminous mix, and therefore, the binder, the aggregate curve and the fine coarse are not considered.
- The solid rubber tyre employed in the test does not accurately represent the real conditions in the contact area between the tyre and the pavement surface (Wilson, 2006). Furthermore, the polishing process may not be as severe as the one that takes places in situ.

- The PSV test can be employed as a way to compare aggregates, but not to measure the performance of a pavement regarding to skid resistance.

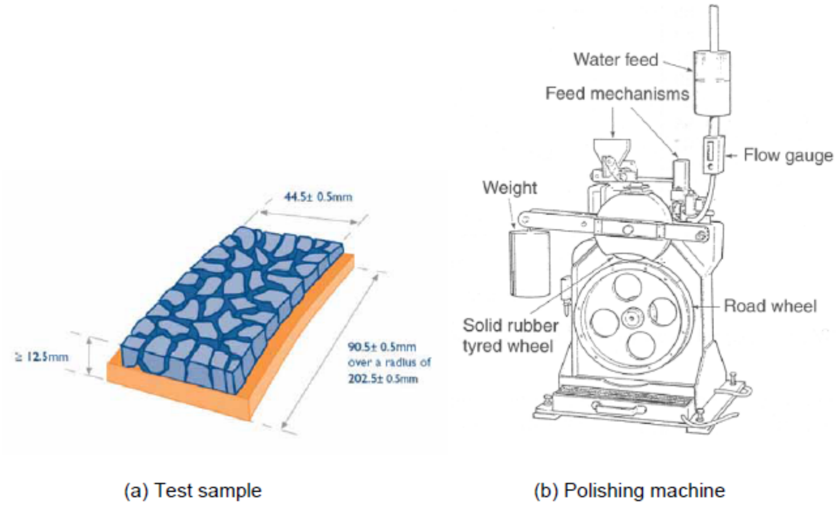


Fig. 5.21. Schematics of the test sample (Austroads, 2011a) and the polishing machine (Woodside and Woodward 2002).

Due to these differences, mainly about procuring the reference stone, grit and tyres required by the UK specifications, alternative models were developed. Some examples of this tendency is observed in Australia, where additional changes were included, PAFV test – Vertical Wheel Method (Standards Australia, 1999a) and PAFV test – Horizontal Wheel Method (Standards Australia 1999b).

5.4.3.5.2. PAFV Test – Vertical Wheel Method

Due to the difficulty to find the reference materials of the British PSV method, aggregate, grit and test tyre were replaced by locally available materials with similar properties. Moreover, some of the testing conditions were also modified, such as water feed rate and polishing time.

The equipment and the procedure are identical to the PSV method. As stated, reference aggregate comes from Australia, Panmure ballast. The wet polishing process starts with three hours with a silicon carbide for coarse polishing and continues with a two-hour polishing with optical emery for fine polishing (Standards Australia, 1999a). Then, the skid resistance is measured using the British Pendulum Tester. As differences are evident when comparing to the UK version, the Australian PSV was renamed as Polished Aggregate Friction Value (PAFV) (Standards Australia, 1999c).

5.4.3.5.3. PAFV Test – Horizontal Wheel Method

The state of New South Wales in Australia developed their own method for friction measurement in the 1960s. It employs a horizontal bed polished instead of the vertical wheel polisher (Fig. 5.22). Since 1962, it was used as the standard procedure for artificially polishing aggregates and was known as Test T233 (RTA NSW, 2001). Current standard is described in AS1141.41 (Standards Australia, 1999b). Aggregates are fixed into truncated wedge shape steel strips by means of an epoxy binder, including a control sample, made of the Australian reference aggregate. The samples are placed on a rotating flat table where they are polished under the action of four solid rubber wheels. The wet polishing process is composed of a first two-hour phase of polishing with a coarse abrasive (silicon abrasive) and a second two-hour phase of polishing with a fine abrasive (optical emery). Finished the four hour polishing process, the skid resistance is measured following the AS 1141.42 (Standards Australia, 1999c).

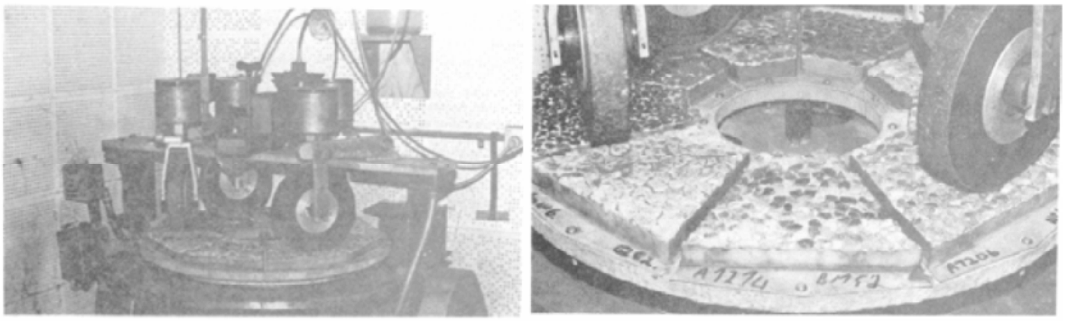


Fig. 5.22. Horizontal bed polished. Source: AS 1141.41: 1999 (Standards Australia, 1999b).

5.4.4. Frictional properties of asphalt mixtures

Asphalt pavement is manufactured with asphalt concrete, which can be defined as the mixture of coarse and fine aggregates and petroleum-based asphalt binder. The skidding resistance of an asphalt pavement depends on the factors of Table 5.16. Some of the main causes related to low available friction that can be controlled during the asphalt mix design are (Kogbara *et al*., 2016):

1) Aggregate quality
2) Aggregate gradation
3) Mix design method
4) Use of higher asphalt content than recommend by the mix design method.

The influence of the aggregate quality was exposed in section 5.4.3. The aggregate gradation and the finish quality of the surface mix are responsible of the surface macrotexture, which influence the water drainage (Hall *et al*., 2009). However, aggregate gradation has little impact on pavement microtexture.

Regarding to mix design methods, there are three main methods employed worldwide for bituminous mixes: Hveem, Marshall and Superpave. A deeper description can be found elsewhere (Swami *et al*., 2004; Al-Mistarehi 2014). The Superpave method is based upon the volumetric analysis, as Hveem and Marshall methods, and was created to replace the latter one. Unlike the Hveem and Marshall methods, which have an empirical basis, the Superpave method is performance-based, and hence, it can predict the performance of the constructed pavement, which is not possible by the other methods (Anderson and Bahia, 1997).

Asi (2007) evaluated the skid resistance of asphalt pavements was evaluated by means of the British Pendulum Tester (BPT). This research included different mix design methods (Marshall and Superpave) and asphalt content (optimum and 0,5 % and 1 % higher), Hot Asphalt Mix type and aggregate types(limestone and steel slug). The conclusions underlined the supremacy of the aggregate quality over the mix design method when available friction is evaluated. Other studies arrived to different conclusion referring to the best mix design method (Li *et al*., 2007; Wu *et al*., 2013b). But once again, the importance of the quality and type of aggregate is highlighted since no statistical evidence of texture change with Superpave and Marshall mix design methods have been found (Kogbara *et al*., 2016). The nominal maximum size of aggregate appears to be the main factor for the macrotexture (Stroup-Gardiner *et al*., 2004).

Different asphalt mixes at different ages were analyzed in order to identify the variables that influence the friction performance of surface asphalt mixes, using BPT for macrotexture (MPD) and laser profiler for microtexture (Ongel *et al*., 2009). It was observed that, with increasing age, the macrotexture increased and microtexture decreased. A scale was established which graded the MPD according to the mix type (open-graded > gap-graded > dense-graded), corroborating earlier studies (Page, 1993; Huddleston *et al*., 1993). However, these results contradict the idea that higher aggregate spacing gives lower friction (BPN) values. Differences with other studies were due to employed aggregates, reinforcing the idea that highlights the importance of aggregate type (Wu *et al*., 2013b).

5.4.4.1. Wehner Schulze Tester

With the aim of measuring friction in real asphalt mixes, the Wehner Schulze test method was developed during the 1960s by Wehner and Schulze at the Technical University of Berlin. The test samples are not individual aggregate piece, but samples of the actual surfacing, produced in laboratory or field cored. It has a wide acceptance in Germany, and is used more and more worldwide.

The tester consists of two units; a first one for polishing and a second one for friction measuring (Fig. 5.23, Fig. 5.24). Firstly, the test samples are placed in the polishing workstation (Fig. 5.25a), where rotating rollers polish it for a defined period under established conditions (Fig. 5.25b).

Then, samples are positioned in the friction measuring workstation and skid resistance is measured using a rotating disk equipped with rubber pads (Fig. 5.25c), in similar way to the Dynamic Friction Tester (section 3.3.9.2.2). Most of the procedure is controlled by pre-defined functions, and hence, variations introduced by operators are reduced.

5.4.4.2. Accelerated Polishing Machine

It is a device developed in New Zealand and is mainly based on the NCAT slab polishing machine, with some changes in the mechanical components and load controller so as to consider the higher macrotexture and irregularities of chip seal samples common in New Zealand (Wilson, 2006). It was developed specifically for being used with the DFT and polishing the prepared sample in the same circular motion as the DFT, aiming to simulate in situ traffic action in the wheel paths. The development of this machine had the permission of the developers of the NACT and the Purdue University, responsible of developing a similar device. The tester has three rotating pneumatic tyres (Fig. 5.26a). The slab sample is placed in the machine and is polished by the wheels in specified conditions of tyre pressure and rotation speed for a period (Fig. 5.26b). Water is supplied by pipes. A seasonal variation can be simulated by means of a more complex polishing process that includes applying some contaminants (with or without water) in various steps. Then, DFT measures are carried out (Fig. 5.26c). At present, the tester is not commercially available.

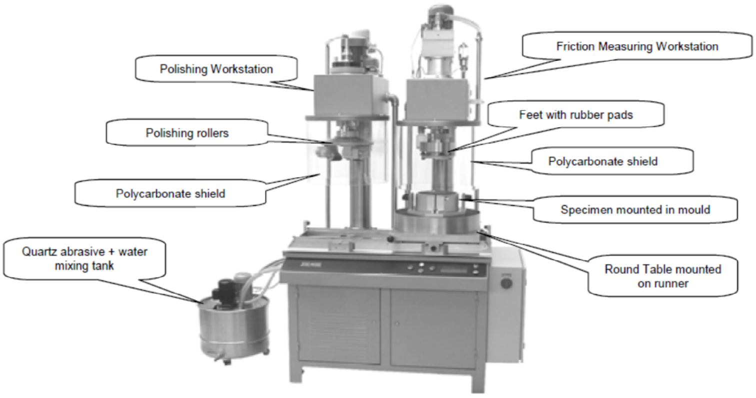


Fig. 5.23. Wehner Schulze machine layout.

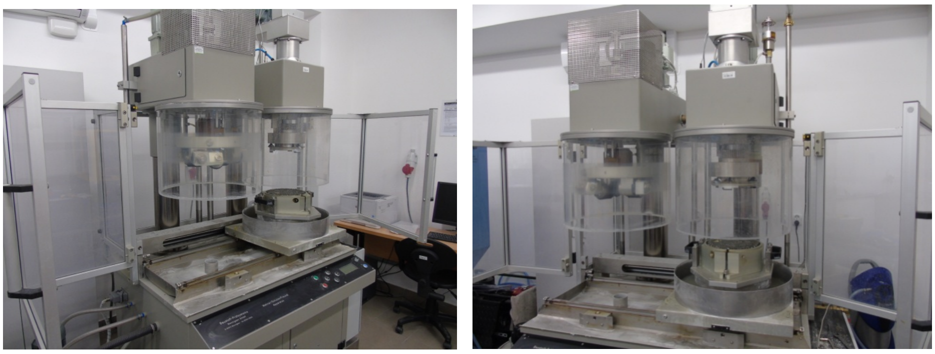


Fig. 5.24. Wehner Schulze machine at the Byalistok University of Technology. Source: author

Fig. 5.25. a) Wehner Schilze test samples, b) Polishing rollers at the polishing workstation, c) Rubber pads at the friction measuring workstation Source: author

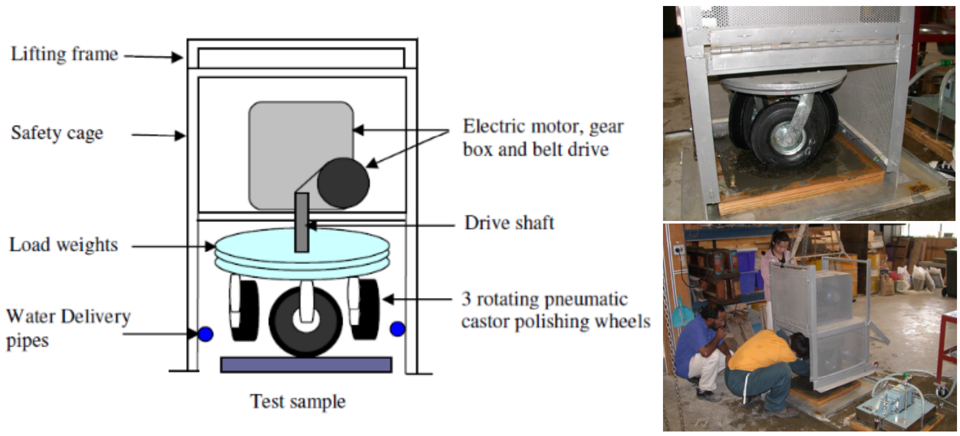


Fig. 5.26. The Accelerated Polishing Machine : a) Schematic front elevation,b) wheel assembly unit in operation, c) machine and DFT on the right (Wilson, 2006).

5.4.5. Environmental factors affecting skid resistance

Water (rainfalls), contaminants and temperature are said to be the main environmental factors that influence the skid resistance. They are analyzed in next sections.

5.4.5.1. Influence of rainfall

As stated, the main factor affecting skid resistance is the presence of water over the surface pavement. When the surface is dry and clean, pavements have very high skid resistance due to the good contact between vehicle tyres and surface. If a dry road starts being slightly wet, an important reduction in the friction occurs because the water film over the pavement acts as a lubricant between the tyre and the surface. It also reduces the contact area, and, hence, the friction. Bird and Scott (1936) proposed the first model for interpreting the friction variation caused by a rainfall from a dry road to wet and then dry again (Fig. 5.27).

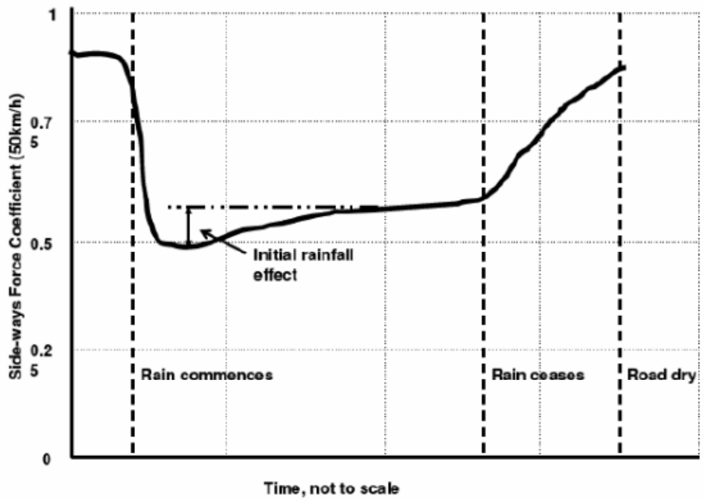


Fig. 5.27. Skid resistance variation during a rainfall (Bird and Scott, 1936).

Different researches have tried to quantify this reduction. Depending on the surface condition, Fricke (1990) showed that friction can be reduced from dry to wet conditions up to 37,5 % for speeds below 50 km/h and 31 % for speeds over 50 km/h. Harwood *et al.*, (1989) demonstrated that a 0,025 mm depth of water can reduce available friction by up to 75 % in roads with low skid resistance.

A thin water film has demonstrated to be enough to produce hydroplaning. The drainage paths, produced by the macrotexture, and the tyre tread contribute to eliminate the bulk of water, which becomes a key factor with high speeds. Nevertheless, a good adherence between tyre and pavement in wet conditions is only achieved if there are sufficient fine-scale sharp edges, i.e. microtexture, where high pressures can build up as the tyre passes. These high pressures are necessary to break through the water film and establish dry contacts between the tyre and the surface (Rogers and Gargett, 1991).

Allbert and Walker (1965) investigated the effect of a tread tyre and pavement surface texture on the skid resistance and indentified the three main zones of the tyre-pavement contact area (Fig. 5.28).

- The Sinkage Zone (Zone 1), or zone of unbroken water film. It appears at the leading edge of the tyre where there is no contact between the tyre and the pavement surface. A water wedge, a bulk, in front of the tyre is usually present. This area is also called the zone of hydrodynamic lubrication. The thickness of the film of water decreases progressively as it is squeezed out by hydrostatic pressure. The tyre floats on that water film.

- The transition zone, or thin film zone. The water film is locally broken by some aggregate asperities, making possible the water film to be penetrated. This area is also called the partial or mixed hydrodynamic lubrication zone. Microtexture of aggregates play a vital role as friction can be develop due to the aggregate protrusions and the hysteresis effect appears.
- The Tractive Zone (3), or Dry zone. Water is expelled and a dry tyre-pavement contact area is created. It is also called the boundary layer lubrication zone. A harsh microtexture is necessary in order to "break" the thin water film and to achieve dry contact. In this way, the whole surface microtexture can become and active actor for developing friction.

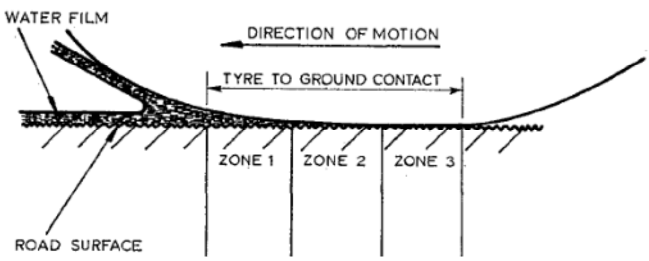


Fig. 5.28. Schematic representation of wet tyre footprint, with the "three zone" concept (Allbert and Walker, 1965).

The extension of each area, and, consequently, the degree of contact that can be obtained between tyre and pavement surface is determined by a series of factors (Austroads, 2005; Jellie, 2003, Wilson, 2006):

- Water film thickness. It depends on the rain intensity and duration and on the pavement surface drainage capability.
- Surface macrotexture depth. It can provide paths to shed water and it is also dependant on the available drainage elements on the road edges.
- Tyre tread depth, width and pressure.
- Vehicle speed.

Vehicle speed is a key factor in wet conditions. At low speeds, the water edge in front of the tyre is readily dispersed, the tractive zone (Zone 3) is large and friction forces may be developed. The tractive zone may represent the 50 % of the contact area. Nonetheless, with increasing speed, there is less time for expelling water from the tyre-pavement contact and, hence, the contact area is reduced and, consequently, the friction forces. This reduction can be minimised by means of drainage grooves, or more effectively, by means of a high macrotexture, which would not only develop drainage paths but also produce greater tyre deformations and increasing the effect of the hysteresis loses (Wilson, 2006).

If speed is increased from 50 km/h to 100 km/h, a 30 % to 70 % reduction of the tractive zone can appear (Kokkalis, 1998). If the tractive zone is null, and, hence, there is no contact between the tyre and the surface, an interrupted water film appears under the tyre, and the "hydroplaning" (or "aquaplaning") effect appears. The hydroplaning effect can be defined as:

"*the build-up of hydro-pressure beneath a tyre that partially of fully exceeds the capacity of the tyre to absorb it, and thus reduces the contact area between the tyre and the pavement*." (Austroads, 2005)

When the hydroplaning effect appears, the vehicle slides over the water film, which is unable to resist shear forces. There are two types of hydroplaning (Austroads, 2005):

- Viscous hydroplaning. It can appear at low speeds if the surface macrotexture is low. It only needs minimal water depth to occur, as it depends on the viscosity of the water, which prevents it from escaping from under the tyre footprint. This type of hydroplaning usually appears in braking manoeuvres and it can be enlarged if the tyres have limited or no tread.
- Dynamic hydroplaning. It appears when the vehicle overpasses a critical speed (dependant on tyre pressure). The water in front of the tyre, acting as a wedge, penetrates the tyre footprint and reduces the contact area. If total dynamic hydroplaning occurs, virtually no part of the tyres is in contact with the road surface, i.e. the tractive zone is inexistent. The vehicle speed must be reduced below the critical one in order to create a tractive zone and avoid the phenomenon.

In the zone 1 (sinkage zone), microtexture has a vital contribution, whereas macrotexture a more reduced one. On the contrary, in the zone 2 (Transition zone), macrotexture plays a negligible role, and microtexture a secondary one. Finally, in zone 3 (Tractive zone), both micro and macrotexture contributes to friction in wet conditions (Jellie, 2003).

5.4.5.2. Influence of contaminants

Surface contamination may interfere with the mechanism providing friction at the tyre-pavement contact, i.e. adhesion and hysteresis and can be originated from various sources (Austroads, 2005; Wilson, 2006):

- Water (discussed in 5.4.5.1), snow, ice and frost.
- Dust, clay, sand, loose gravel, tracked bitumen.
- Vehicle residues: oil, fuel, rubber, brake pad linings
- Farm waste, fallen leaves, grass clipping, lichen, moss.

The influence of individual contaminants on friction has not still developed. Contaminants can cover the pavement and accelerate its ageing process. On the other hand, contaminants not including water, snow, frost or ice, can be accumulated during dry periods and hazardous situation for road users can appear at the beginning of the first rain because the contaminants on the surface create a slick coating over it. Beenis and De Wit (2003) modelled the friction variation over a 5 minute-period with a short rainfall after a dry period (Fig. 5.29). The available friction is significantly reduced immediately after the rainfall and then it achieves a usual friction in wet conditions. After the rain finishes, the surface dries and a typical skid resistance in dry conditions is achieved again, similar to the available one before the rain started.

The presence of frost, ice, or snow over the road can mask the microtexture and/or the macrotexture, and hence, reduce the hysteresis effect. These contaminants can act as a lubricant in the tyre-surface interface too, implying a great reduction of friction. The three categories of winter condition on roads are (VTI, 1981):

- Hoar frost (white frost)
- Ice (thick ice glaze thin ice and ground icing)
- Snow (loose, compacted, slush and sleet).

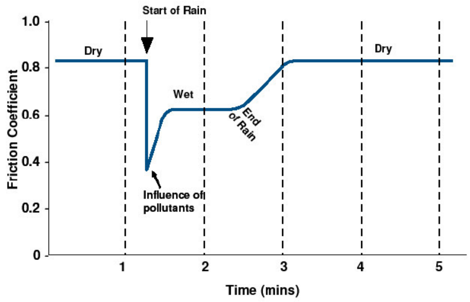


Fig. 5.29. Friction variation during a rainfall (Bennis and De Witt, 2003).

Extreme winter conditions are typical in cold alpine or semi-alpine areas, which is not the case of Biscay, with a oceanic clime and few kilometres over 700 m altitude. Therefore, contaminants related to extreme conditions are not further analyzed. A more deep description can be found in the literature (VTI, 1981; Gustafson, 1982, 1983; Dravitzki *et al.*, 2003).

Finally, tracked bitumen from road sections that are bleeding can also be defined as a contaminant because the binder can mask the microtexture. Different friction-generating mechanisms are said to be produced if aggregates are in this "blackened" state due to the reduced influence of the microtexture (Cenek *et al.*, 2003). The friction is said to be principally dependant on the shape characteristics of the aggregate, which generates hysteresis friction, and not in such important way, on the adhesive or viscous properties of the binder.

5.4.5.3. Influence of temperature

Except in extreme clime conditions, temperature does not affect the friction properties of crushed aggregates usually employed in pavement layers. On the contrary, as both tyre rubber and bituminous materials are made of visco-elastic materials, they are more sensitive to temperature changes because of the hysteresis mechanism of friction and hence, skid resistance is affected. Moreover, as described in section 3.3.6.2, most of the testing devices use rubber contact elements, which are also dependant on the temperature. It is generally adopted that with increasing temperature, the hysteresis losses reduce and the available friction also reduces (Jayawickrama and Thomas, 1998; Feighan, 2006), but different values for this reduction have been measured (Hosking and Woodford 1976a; Oliver 1980). The main disagreement is related to the fact that correction factor in template climes cannot be applied to more tropical or hot climes (Oliver 1980).

The effect of the temperature in the measured skid resistance can be summarized in the following bullets (Oliver, 1980; Hill and Henry, 1981; Hosking, 1992; Wilson 2006):

- With increasing air temperature, the measure friction tends to decrease.
- Changes in temperature primarily affect the friction properties of the tyre, and hence, affecting the coefficient of friction
- Tyre temperature is usually proportional to air and pavement temperature, and higher tyre

temperatures imply a decreasing friction coefficient.

- Water temperature has an insignificant effect on skid resistance
- Higher pavement temperatures imply friction reduction.

Some friction measuring devices are more sensitive to changes in temperature. In these cases, correction factors are required to compare collected data. An example of this can be observed in measures carried out by SCRIM. The Transport and Road Research Laboratory studied the influence of the temperature on SCRIM results (Hosking and Woodford, 1976b). With tyre temperatures varying from 9 °C a d 26 °C on concrete and bituminous surfacing, they concluded that increasing temperature reduced SFC by about 0,003 units per °C. Another study conducted on concrete and bituminous surfaces at midday and during the night revealed similar variation estimation and estimated that temperature variation appeared to account for about one-quarter of the overall change in skidding resistance from midsummer to midwinter. From these studies are combined in Fig. 5.30 to predict the relationship between temperature and SFC value at 20 °C.

Two equations were formulated. A linear regression analysis provided Eq. 5.52:

$$\frac{SFC_t}{SFC_{20}} = 1{,}106 - 0{,}0054 \cdot t \qquad [5.52]$$

Where SFC_{20} is the estimated SFC value at 20 °C, *SFCt* is the temperature measured in situ, at *t* temperature (°C). Eq. 5.53 has correlation a coefficient (*R*) of -0.79 for 64 observations. A better correlation (*R* = 0,81) is provided by Eq. 5.53, obtained with a hyperbolic relationship.

$$\frac{SFC_t}{SFC_{20}} = 0{,}548 + \frac{44{,}69}{t+80} \qquad [5.53]$$

The Transport and Road Research Laboratory also related tyre temperature (tyre) and the mean of the air and road temperature, $t_{air\text{-}road}$, by means of Eq. 5.54, with a correlation coefficient of 0,87:

$$t_{tyre} = 12{,}3 + 0{,}96 \cdot t_{air-road} \qquad [5.54]$$

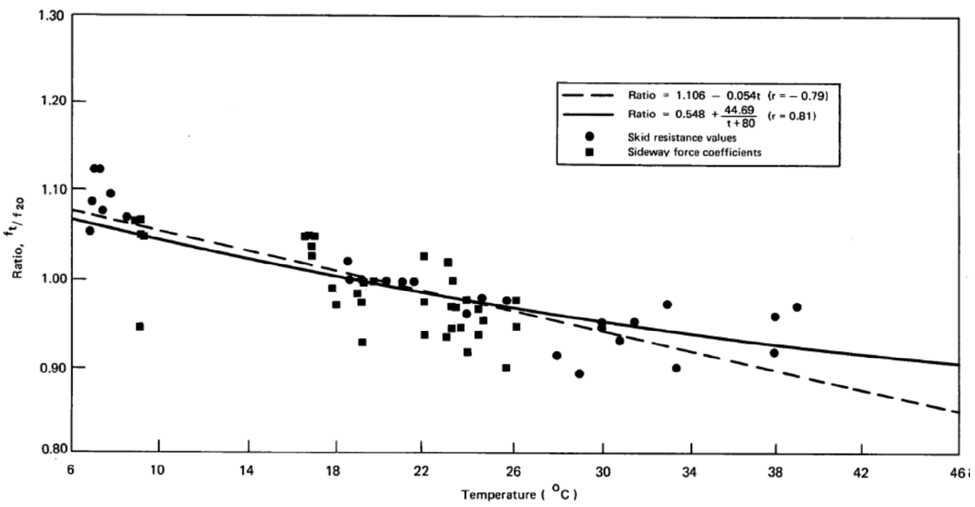


Fig. 5.30. Relation between change in skid resistance and temperature (Hosking and Woodford, 1976b).

It was also established that, during the usual period of test in Great Britain, from May to September), will not be often outside the range 20 °C to 57 °C (corresponding to a range of 8 °C to 45 °C for the main of road and air temperature). Hence, variation would be about 7,5 % at maximum. For a SFC value of 50 at 20 °C, it would range from 0,50 to 0,44 according to the temperature at the time of the set. Consequently, the temperature effect is not likely to be a key factor when mean summer values are employed (Hosking and Woodford, 1976b).

5.4.5.4. Influence of pavement age

A general model is accepted worldwide to represent the skid resistance performance with time (Fig. 5.31) (Kokkalis, 1998; Prowell *et al*., 2003, Kane *et al*., 2013).

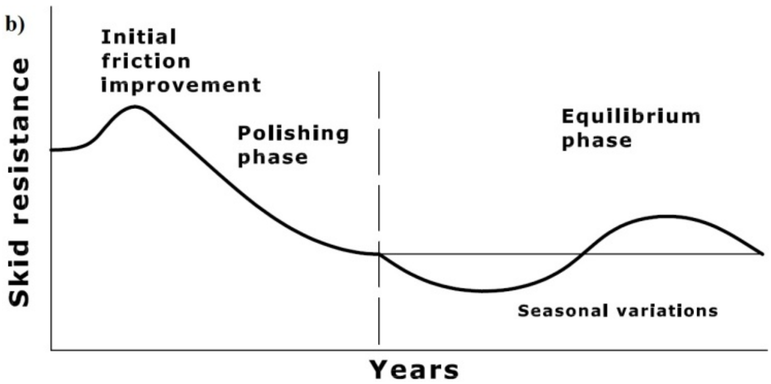


Fig. 5.31. Simplified general pavement skid resistance model (Kokkalis, 1998; Prowell et al., 2003, Kane et al., 2013).

In a new pavement, an initial skid resistance rise can appear if the aggregates are covered by a bituminous film. When the bitumen film is worn away, the aggregate microtexture is exposed and, hence, skid resistance increases. Then, once exposed, aggregates suffer from normal polishing process and their friction level reduces, until an equilibrium phase is achieved (Corley-Lay, 1998). In this equilibrium phase, as long as a constant traffic flow is constant, all the researchers described that the only fluctuations are seasonal variations (section 5.4.5.5) and short-term variations (section 5.4.5.6).

Consequently, the three phases of the skid resistance of a new bituminous layer can be listed as follows:

- Phase 1: Initial Roughening phase or initial improvement phase. Initial skid resistance values are relatively low because a bituminous film usually recovers the aggregates. Due to the traffic loads and environmental agents, the bitumen coating is worn away and the natural aggregate asperities appear, increasing the friction values.
- Phase 2: Polishing phase. Traffic loads polish the microtexture of the aggregates, according to their polishing resistance, and reduce the natural harshness of the stone. Consequently, available friction reduces. According to some researches, this friction decreasing phase is said to follow a negative exponential phase, (Mahmoud and Masad, 2007; Rezaei and Masad, 2013; Kassem *et al*., 2013). The skid resistance tends to settled down to an asymptotical value.
- Phase 3: Equilibrium phase. When the polishing process has reached a stable or near equilibrium phase for a determined traffic volume, the only friction variation is the seasonal variation. It follows

a cyclical seasonal pattern, where the highest values are collected in winter months, while the lowest values are recorded in summer (section 5.4.5.5).

Regarding to the duration of each phase, there is no consensus among researchers. Discussing about the initial improvement phase (Phase 1), this phenomenon takes places during the elimination of the bitumen film that covers the aggregates due to the traffic action (Jellie, 2003; Achútegi Viada, 2005). It depends on the binder type, its quantity and the heavy traffic characteristics. In Spain, this increasing skid resistance step is said to last for some weeks (Achútegi Viada 2005; Navarro *et al.*, 2011). When analyzing well constructed chipseal surfaces this initial improvement is not always observed. Nevertheless, when using stone mastic asphalt (SMA), the duration of this phase is longer because of the thicker coating of the binder. In SMA surfaces with a high quantity of binder, it was demonstrated that this phenomenon can predominately dominate the whole life of skid resistance performance (Woodward *et al.*, 2002). Woodwart *et al*. (2005) showed that a polymer modified surface had not still completed the aggregate exposure after 4 years of traffic and hence, the friction values continue being low (Fig. 5.32). This indicates that the traffic and environmental conditions were not sever enough to remove the bituminous film from the surface. It can be concluded that the modified bitumen was too elastic and cohesive.

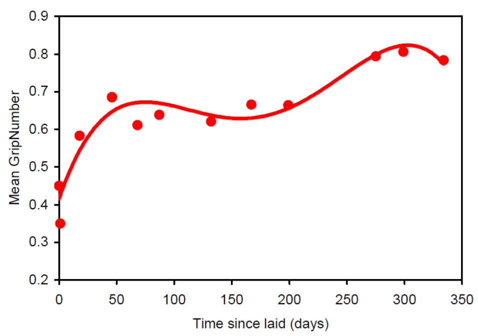


Fig. 5.32. Skid resistance variation with time in a 14 mm width section of stone mastic asphalt (Woodward et al., 2005).

Bullas (2005) exposed that surfaces with negative texture could produce cases where this kind of surfaces can be “slippery when dry” even with uncontaminated asphalt mixes, contrary to the general belief. Bullas (2005) called this phenomenon “*bituplaning*”, which can be described as the generation of low skid resistance value due to the a thick bituminous binder that covers the mix aggregates under locked wheel breaking (slip ratio of 100 %). Examples of crashes occurred in the UK and in the Netherlands in these situations leaded researchers to consider this phenomenon as an accident cause. Despite the fact that collected values were not lower than those collected in wet conditions, drivers do not adapt their speed when the road is dry as they do in wet conditions. Hence, the Netherlands Road Controlling Authorities employs signals with the message “*New road surface, longer stopping distance*” in order to prevent drivers of the risk of bituplaning (Bullas, 2005).

Regarding to polishing phase duration, disagreement also appears. In the literature, examples of a four or five year-duration (Wilson, 2006) and examples of polishing phases lasting for half year, one or two year (Navarro *et al.*, 2011, Kokkalis, 1998; Feighan, 2006) can be found. The necessary time to achieve the equilibrium point is largely dependent on the heavy traffic volume (Achútegi Viada 2005). Finally, in the equilibrium phase seasonal variations are only recorded, which are deeply described in next section. Without considering those seasonal fluctuations, friction values remain constant.

5.4.5.5. Influence of seasonal variations

After the initial improvement phase and the polishing phase, at the equilibrium point of a road, within the year, the lowest skid resistance values in wet road surfaces have been now to be registered in summer since 1931 (Bird and Scott, 1936). On the contrary, the highest values have been always recorded in winter months. This phenomenon is said to be caused by the combined effect of traffic and weather on the surface aggregate. In dry roads, the polishing effect action of traffic is dominant, but, when the road pavements are wet for long periods, surfaces recover some of their former texture and harshness (Rogers and Gargett, 1991). The magnitude of these seasonal variations is mainly dependent on the geological history and petrography of the used aggregates. The first study showing these seasonal variations was carried out for the UK Highway Agency. It analyzed the skid resistance measurement carried out by the SCRIM devices in different road section in the UK, in pavements that were polished (or that have at their equilibrium phase), every month during 11 years, from 1958 to 1968 (Hosking and Woodford, 1976b). Fig. 5.33 shows the results of this research, where the annual cyclical variation effect can be observed, with a remarkable seasonal variation.

Apart from researches in the UK (Hosking 1992, Rogers and Gargett, 1991, Salt 1977a; Salt 1977b), other studies have also highlighted the seasonal variation observed in friction values (Dahir and Henry 1978; Henry and Saito, 1983; Rice, 1997; Jayawickrama and Thomas, 1998; Navarro *et al.*, 2011). They mentioned that the shape of these fluctuations could be simulated with a sinusoidal function, with the highest values in winter months and the lowest values during summer.

Hosking and Woodford (1976a) estimated a 0,12 range for the amplitude of the variation, and an amplitude of 0,04 among summer months, measured by a SCRIM device.

Once in the equilibrium, differences between years are related to climate changes but are less important than the seasonal variations (Hosking and Woodford, 1976b; Pérez-Acebo *et al.*, 2017a). As observed in Fig. 5.33, there are small differences between the lowest values of each year

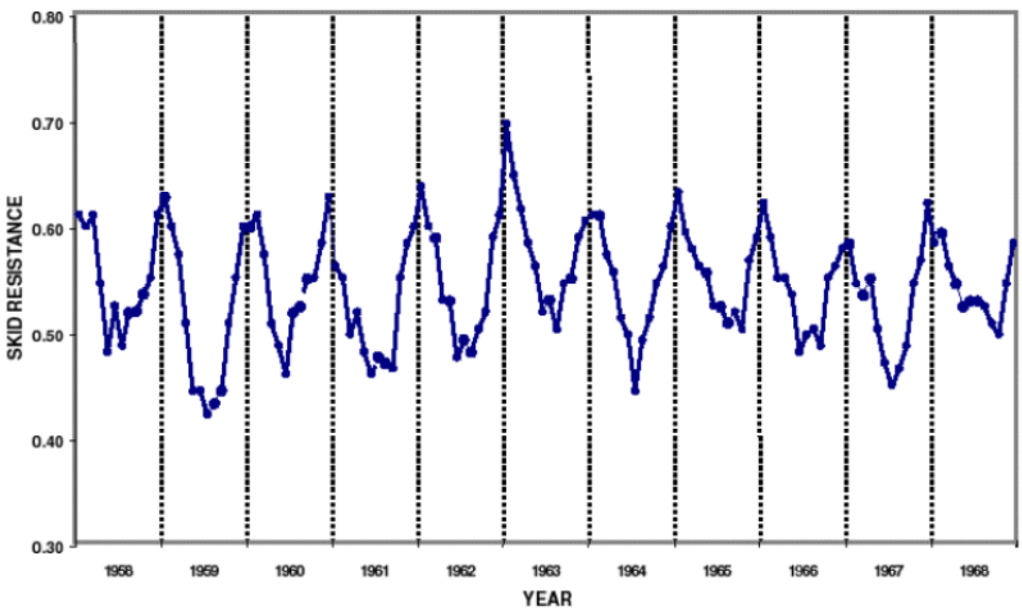


Fig. 5.33. SCRIM values collected during a eleven-year period (1958-1968) every month in UK (Hosking 1976b).

Cenek *et al.* (1999) conducted a research to study the long-term performance of the skid resistance from the beginning, when a new road is opened to traffic. Seven newly surfaced chipseal in 1988 in Northland (New Zealand) were selected. They were on a straight line and on flat grade slope, and different greywacke aggregates, with similar Polished Stone Value (PSV) were used in the construction. Measurements by means of a British Pendulum Tester were collected every 3 months from 1988 to 1998 (Fig. 5.34). The three phases described before (initial improvement, polish stage and equilibrium point) can be observed in Fig. 5.34, combined with the seasonal variations, with the lowest values in each phase in the summer months of the Southern Hemisphere (beginning of the year).

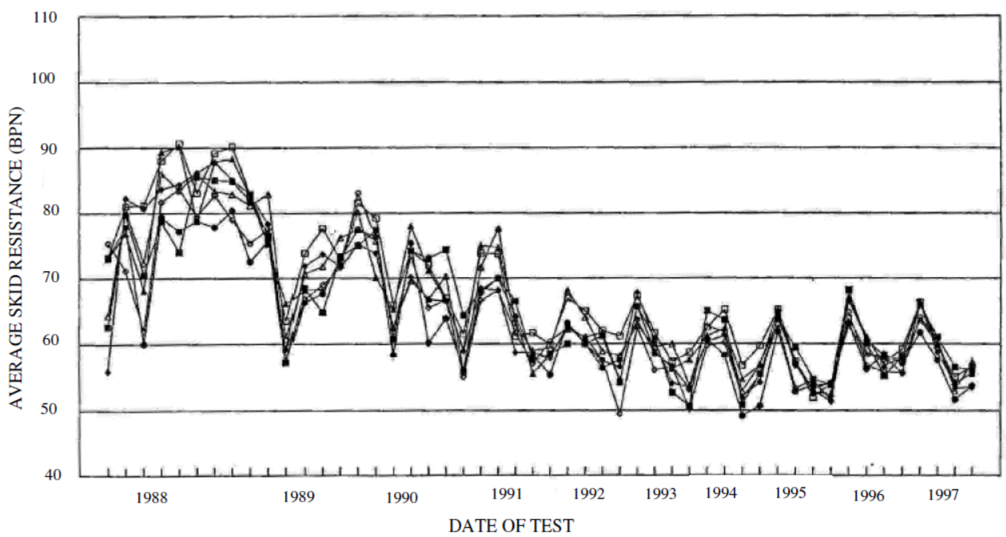


Fig. 5.34. Long-term skid resistance performance in seven roads, measuring by a BPT (Cenek et al., 1999).

As stated previously, the seasonal variations in the equilibrium phase has been modelled following a sinusoidal shape. In the northern hemisphere, the fluctuation through the year has been represented in Fig. 5.35 for a section with a mean summer SCRIM value of 0,50.

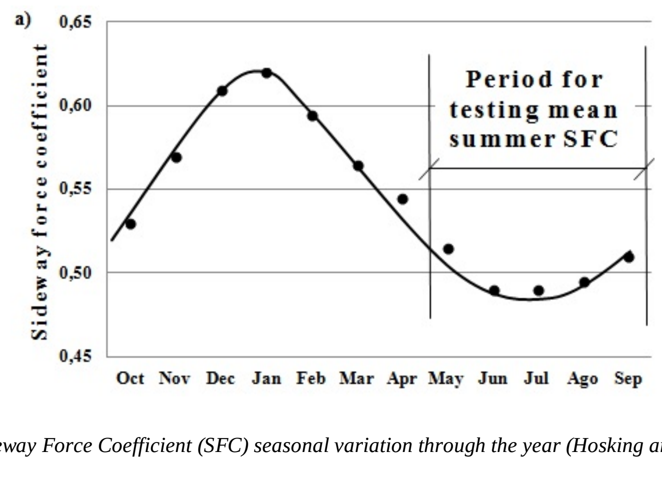


Fig. 5.35. Ideal Sideway Force Coefficient (SFC) seasonal variation through the year (Hosking and Woodford 1976a).

Road agencies must assure a minimum friction in their road network. Therefore, knowing the minimum level of skid resistance available is a vital issue in their pavement management system. Consequently, highway administrations are concerned about the importance of predicting the minimum friction. Thus, the British Highway agency has employed the Mean Summer SCRIM coefficient, MSSC, to determine the network and project level skid performance (Hosking and Woodford 1976b). The MSSC is calculated from the mean of 3 SC values for each region (called location) in summer months. It is obtained at its lowest and also when variation is the least, with measurements in summer every 3 years, in roads in the equilibrium polished state (Hosking and Woodford, 1976b). Hence, the "worst case" skid resistance value is expected to be given.

At present, un UK the *Characteristic SCRIM Coefficient*, *CSC*, is proposed as standardized value obtained in a month from May to September, every year in a different month and adjusted according to the observed variation in the previous 3 years in that area (Brittain, 2015). For example, a route tested in the early part of the season in year 1, could be tested in the late part of the season in year 2 and in the middle part of the season in tear 3. In year 4, it will be tested in the early part of the season again, etc. (Highways Agencies, 2015). Fig. 5.36 explains the differences in data collection between both plans.

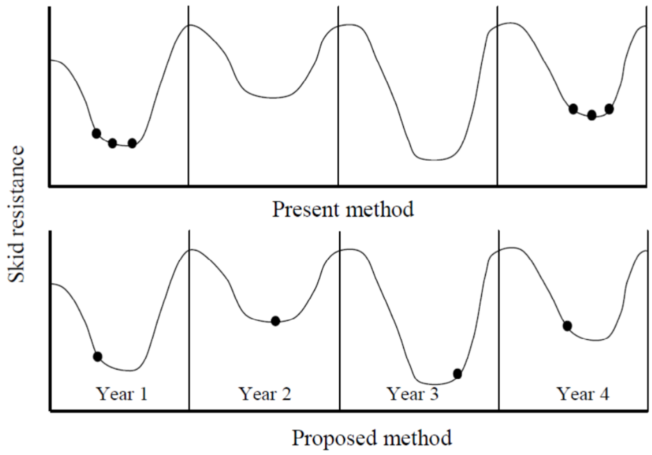


Fig. 5.36. Data collection plan for calculation of MSSC and CSC (Highways Agencies, 2015)

For calculating the CSC, a location is defined. A location is a collection of road sections or routes for which a correction factor is determined. It must be small enough so that similar weather conditions are normally experienced within it and large enough so that a stable value can be calculated to represent the long-term skid resistance. The *Local Equilibrium Correction Factor* (*LECF*) is the correction factor for each locality to bring the present year data to a level consistent with the long-term average, by removing the within-year seasonal variations and the variation between years (Fig. 5.33). The calculation has three stages (Highway Agencies, 2015).

First, *The Local Equilibrium SC* (*LESC*) is the average *SC*, calculated for all valid 10 m sub-section measurements in the locality overt the 3 years preceding the present test section. It represents the average skid resistance level of the locality over recent years. Secondly, the *Local Mean SC* (*LMSC*) is determined from the current survey. It is the average of all valid sub-sections in the locality in the current year survey. Finally, *LECF* is obtained by Eq. 5.55:

$$LECF = \frac{LECS}{LMSC} \qquad [5.55]$$

Them the CSC for each 10 m sub-section is calculated by multiplying the corrected SC (defined in Eq. fvgb) by the LECF (Eq. 5.56) (Highway Agencies, 2015).

$$CSC = LECF \cdot SC \qquad [5.56]$$

A valid measurement is the one that was meade in ther required part of the season (early, middle, late), on road surfaces that were at least 12 months old at the time of testing. This implies that if a length of road was resurfaced within the last 4 years, then that length should be excluded from the LEFC calculation (Highway Agency 2015). Consequently, as observed, the initial friction improvement stage and the polishing phase are supposed to be concluded after one year. Even a benchmark site approach, where MSSC is determined in that benchmark site, is proposed for estimating the CSC for sites with a single annual survey of the network.

A similar procedure is followed in New Zealand (Transit New Zealand 2002a; 2002b), where the Equilibirum Skid Coefficient (ESC) is calculated. It is obtained by means of yearly network surveys. The ESC is obtained by averaging the SCRIM coefficient measured within the testing season over a period of three consecutive years. Every year is surveyed in a different timing within the testing season, similar to the UK procedure. It results on a more stable and comparable parameter for multi-year comparison and other pavement management purposes (Feighan, 2006).

On the other hand, as it can be observed in Fig. 5.33 and Fig. 5.34, the minimum values in summer and the sinusoidal shape of the fluctuation varies from year to year due to the predominant climatic pattern of the year (Hosking and Woodford, 1976b). Even differences between the highest values of winter can be appreciated. However, both figures show that seasonal amplitude of the wavelength remains very similar and only seems to be moved slightly upwards or downwards every year. There is general consensus among researchers about the reasons for the seasonal variations (Wilson, 2006), that was explained adequately by Jayawickrama and Thomas (1998):

> *"Prolonged periods of dry weather in the summer allow the accumulation of fine particles that assist in polishing of the pavement surface. The combination of polishing and particle accumulation, together with the contamination from vehicles such as oil drippings and grease, results in a loss of microtexture and macrotexture during the summer months. In winter, the aggregate surface is rejuvenated with chemical reactions from the rainwater exposing new particles. The increased rain flushes out the finer particles responsible for polishing and other debris increasing macrotexture. The coarser aggregate surface and the increased macrotexture in turn lead to an increased skid resistance of the pavement."*

Other researchers also introduce the idea that the water film that covers the pavement surface for long periods in winter acts a lubricant and the polishing effect of traffic is reduces. Nevertheless, Oliver *et al.* (1988) reported that not all the climate regions followed this sinusoidal pattern. A weekly measurement by BPT was carried out in six cities of Australia (Fig. 5.37). The fluctuation of Queensland, situated in a sub tropical climate, is very small and is not so related to the season.

5.4.5.5.1. Proposed models for seasonal variations of skid resistance

Generally, the models proposed in the literature about seasonal variations of skid resistance employs a similar mathematical approach, a sinusoidal model. Some authors based the decision of applying a sinusoidal oscillation to the friction value due to the sinusoidal oscillation of the pavement temperature and rainfall, to which skid resistance variations are said to be related.

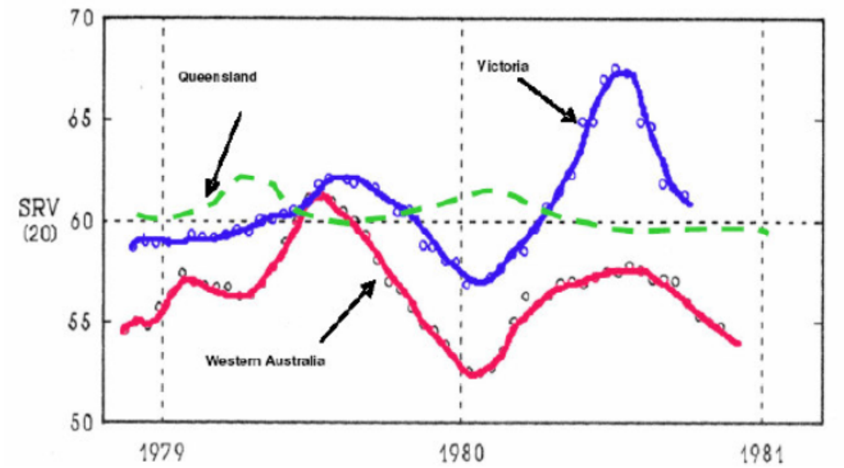


Fig. 5.37. Seasonal variation of friction in Western Australia, Victoria and Queensland (Oliver et al., 1988)

One of the first models was developed by Burchett and Rizenburgs (1980). It describes the friction behaviour as an oscillation around a trend value, expressed as Skid Number (*SN*), measured with an ASTM trailer. They correlated it with the Julian calendar by means of Eq. 5.57:

$$SN_p = SN_a + \Delta SN \cdot \sin\left[360 \cdot \left(\frac{D - D_1}{365}\right)\right] \qquad [5.57]$$

Where SN_p is the predicted Skid number, SN_a is the Skid Number about which the friction varies, ΔSN is the largest variation of *SN* about SN_a, D is the day of year and D_1 is the day of the year at which *SN* is lowest. As observed, the difference between *D* and D_1 is the wavelength and ΔSN is the oscillation amplitude. Burchett and Rizenbergs (1980) calibrated the values of SN_a, equation after measuring several types of pavements and traffic levels in Kentucky and by means of linear regressions (Fig. 5.38). They found that bituminous surfaces had greater amplitudes than concrete pavements. Moreover, they found that the higher the traffic, the lower was the skid resistance variation.

The lowest values predicted by Eq. 5.57 was between August and September.

Similarly, Diringer and Barros (1990) proposed a model that only depended on the Julian calendar day (JDAY) (Eq. 5.58). The skid resistance values were obtained with an ASTM trailer at 64 km/h (40 mph).

$$SN40 = SN_{ter\min al} + 3{,}0 \cdot \sin\left[360 \cdot \left(\frac{JDAY_1}{365}\right)\right] \qquad [5.58]$$

Where $SN_{terminal}$ is the terminal skid resistance of the pavement, in the equilibrium phase, and the seasonal variations fluctuate around it. The model predicts values of *SN40* from the terminal value, $SN_{terminal}$, and an oscillatory factor dependent on the day of the measurement. Since the $SN_{terminal}$ is difficult to obtain from in situ data, Diringer and Barros (1990) proposed an indirect procedure to estimate it. The procedure is based on a correlation between $SN_{terminal}$ (obtained with an ASTM trailer in sections with high traffic volume) and the PSV obtained by means of a polishing stone machine. The proposed method is useful for evaluation the long-term seasonal oscillation, but a good estimation of $SN_{terminal}$ is necessary.

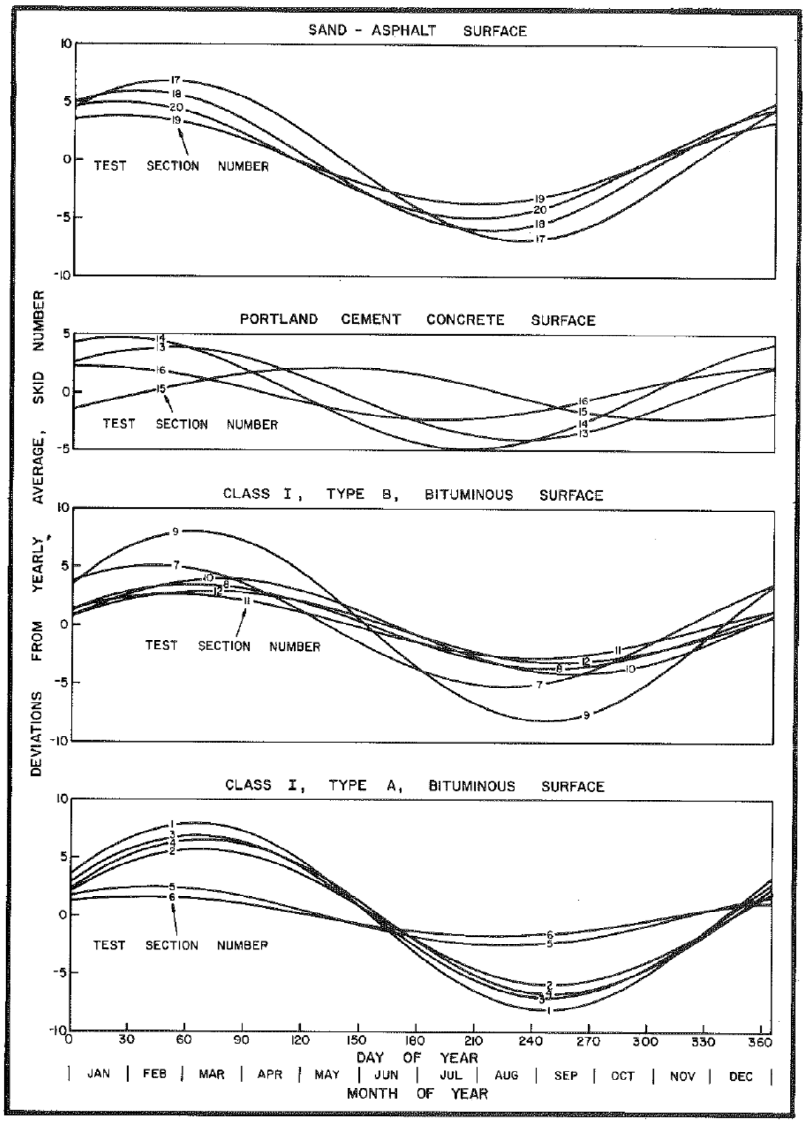


Fig. 5.38. Best-fit curves for Skid Number variation vs. Day of the year, in different test sections by pavement types (Burchett and Rizenbergs (1980).

Another skid resistance model was developed, after considering 31 test sections in Texas (USA) and 18 months of monitoring, including skid resistance, temperature, rainfall and traffic data (Jayawickrama and Thomas, 1998). The model, expressed by Eq. 5.59, had terms that were significant at the 0,05 level.

$$SN_{64} = a_0 + a_1 TEMP_5 + a_2 RF_5 + +a_3 \sin[(2\pi/365)\cdot JD] + \sum_{r=1}^{5} B_r \cdot I_r \qquad [5.59]$$

Where SN_{64} is the skid number at 64 km/h (40mph), $TEMP_5$ represents the average of daily temperatures for the 5 days before the measurement, RF_5 represent the cumulative rainfall over the 5-day period before the measurement. is the variable that represent the amount of rainfall received at the site of the section before skid measurement, JD is the Julian calendar day of the measurement, I_r are the indicator variables that identify the pavement section and A_i and B_r are regression coefficients. Their values are shown in Table 5.19.

Table 5.19. Regression coefficients and corresponding p-values(Jayawickrama and Thomas, 1998).

Coefficient	a_0	a_1	a_2	a_3	b_1	b_2	b_3	b_4	b_5
Magnitude	32,28	-0,14	0,031	-0,66	13,53	-3,12	-2,78	9,52	7,43
p-value	0,0001	0,0001	0,0079	0,0309	0,0001	0,0001	0,004	0,0001	0,0001

The model can consider short-term conditions, like rainfall and temperature effect over skid resistance and long-term conditions through the seasonal variations of the skid resistance.

Cenek *et al*. (1999) conducted a research to develop a procedure for adjusting values of wet road skid resistance measured by GripTester and British Pendulum Tester on a given weather conditions to a different day with different environmental conditions. About yearly seasonal oscillations, they concluded that the most appropriate model to estimate seasonal variations in skid resistance about a terminal values was obtained by:

$$C1 \cdot \cos(2\pi \cdot y) + S1 \cdot \sin(2\pi \cdot y) \qquad [5.60]$$

Where y is the Julian calendar day / 365,25 and y = 0 corresponds to January 1. The constant terms *Ce1* and *S1* are a function of aggregate type, climatic region and skid tester. If *S1* is set to zero, the model is simplified, losing a negligible precision. Moreover, an additional simplification is made if the amplitude of the seasonal adjustment (*C1*) is hold at a conservative value. These simplifications led to the following models. For the British Pendulum Tester, Eq. 5.61:

$$BPN = BPN_{terminal} - 5 \cdot \cos\left(\frac{2\pi \cdot JDay}{365{,}25}\right) \qquad [5.61]$$

Where *BPN* is the British Pendulum Number at any day, and $BPN_{terminal}$ is the value around which the skid resistance value oscillates in the equilibrium phase. *JDay* is the Julian calendar day.

For the GripTester, Eq. 5.62:

$$GN = GN_{terminal} + 0{,}002 \cdot \cos\left(\frac{2\pi \cdot JDay}{365{,}25}\right) \qquad [5.62]$$

Where *GN* is the Grip Number obtained in tow mode, $GN_{terminal}$ is the friction value in the equilibrium phase.

An alternative approach was employed by McDonald *et al*. (2009). They deployed structural equations modelling of energy transfer to evaluate seasonal variations of the skid resistance in situ data obtained from the Long Term Pavement Performance (LTPP) data. Instead of a seasonal model, an estimation of monthly changes of skid resistance was proposed, by relating air temperature variation in each month with skid resistance variation. As example, the model for Richmond, in Virginia (USA) is shown in Eq. 5.63:

$$\Delta SN40 = -0{,}138 \cdot \Delta MaxTemp \qquad [5.63]$$

Where $\Delta SN40$ is the skid number at 40 mph (64 km/h) and $\Delta MaxTemp$, in degrees Fahrenheit, is the variation in the monthly maximum temperature.

Echaveguren and de Solminihac (2011) proposed a model of the form of Eq. 5.64 for skid resistance oscillation from data collected in Chile.

$$SR(SI) = SR_B + A_0 \cdot \cos(\omega \cdot SI + \varphi) \quad [5.64]$$

Where *SR* is the skid resistance, measured by means of SCRIM device, SR_B is the reference skid resistance that allow the amplitude and period of the oscillation to be define; A_0 is the wave amplitude; ω is the wavelength; and φ is the starting gap of the wave. *SI* is a time index defining the season of the year in which data were collected: *SI* = 0 for spring; *SI* = 1 for summer; *SI* = 2 for autumn; and SI = 3 for winter. It terms of time, each *SI* represents a group of 3 months of the year and is located in the middle of each season. Models were proposed by adding factors affecting the friction. Firstly, the pavement surface (asphalt concrete, surface dressing or cement concrete) were obtained; secondly, the macrotexture level, high (sensor mean texture depth (SMTD > 0,6 mm) or low (SMTD < 0,6 mm); and finally, traffic level. It was a way of clustering the effects. Table 5.20 shows the coefficients of final models.

Table 5.20. Calibration of seasonal models of skid resistance for each type of pavement surface, macrotexture levels and traffic levels Echaveguren and de Solminihac (2011).

Type of pavement surface	**Traffic level**	**Macrotexture level**	**Calibration coefficients**				**error**
			SRB	A_0	ω	φ	
Asphalt concrete	High	High	0,585	0,105	1,2	-3,6	0,059
Asphalt concrete	High	Low	0,507	0,083	1,1	-3,1	0,074
Asphalt concrete	Medium	High	0,608	0,167	1,1	-3,3	0,088
Asphalt concrete	Medium	Low	0,598	0,112	1,1	-3,3	0,080
Asphalt concrete	Low	High	0,633	0,249	1,1	-3,2	0,026
Asphalt concrete	Low	Low	0,621	0,097	1,0	-2,9	0,069
Surface dressing	Medium	High	0,638	0,133	1,1	-3,1	0,072
Surface dressing	Medium	Low	0,608	0,167	1,1	-3,2	0,088
Surface dressing	Low	High	0,644	0,107	1,1	-3,1	0,066
Surface dressing	Low	Low	0,679º	0,112	1,0	-2,9	0,115
Cement concrete	High	Low	0,462	0,042	0,9	-2,3	0,037
Cement concrete	Medium	Low	0,487	0,067	0,9	-2,2	0,038
Cement concrete	Low	Low	0,496	0,110	0,9	-2,3	0,064

5.4.5.6. Short-Term variations

Bird and Scott (1936) were the first to mention that skid measurement varied significantly (Fig. 5.29) with variable weather condition, indicating that a friction drop occurs just after a rainfall had started. Hill and Henry (1981), from the result of a three-year research program, concluded that there were both long-term cyclical seasonal variation, identified as primary effect, and short-term friction variations, referred as secondary effect, and, consequently, a skid resistance management programme was difficult to establish. Both effects were superimposed on the seasonal annual cycle. Moreover, Henry (2000) pointed out that the main reasons for the short-term variations are the effect of the rainfall and the contaminants present on the road surface. If a friction measurement is made in a dry period, the water applied is mixed with dust and oil, reducing the friction value. If it is made after a rainfall, contaminants have been removed from the surface and higher values are obtained. On this basis, after long dry periods, the first rainfall can create a slippery road surface. Generally, drivers know that first rain makes road very slippery and reduce their speed. Hill and Henry (1981) explained that, while reasons for seasonal variations are polishing properties of aggregates and traffic volume, the causes for short-term variations can be summarized as:

- Rainfall effect
- Temperature effects (section 5.4.5.3)
- Errors in the measurement of skid resistance by the device

However, Cenek *et al.* (1999) indicated that although the main effect of rainfall is generally accepted for short-term variations in friction, the mechanisms or procedure to explain how it works are not yet sufficiently understood so as to model them reliably.

5.4.6. Effect of traffic loads

As stated in previous sections, the skid resistance performance after a new road is opened to traffic or a resurfacing activity is carried out varies according to the polishing action of vehicles circulating on the road. Vehicles replicate polishing effect described in section 5.4.5.4 and consequently, the three phases appear. Additionally, due to different environmental factor, seasonal and short-term variations occur.

At the beginning, the aggregates of a hot mix asphalt have good microtexture and sharp edges, but, under the polishing action of vehicles tyres, the edges become worn, microtexture is reduced and friction falls. It is generally assumed that the extent to which a road surface becomes polished is directly related to the traffic intensity, and especially with heavy traffic intensity (Wilson, 2006). A transversal profile of skid resistance will show lower friction levels in the wheel tracks. As a consequence, some devices like SCRIM measure skid resistance in the wheel paths.

With regard to the effect of traffic loads on the friction, Keneddy *et al.* (1990) indicated that, if other conditions were equal, a road with the highest heavy traffic volume would have the lowest skid resistance. If traffic loads are higher, road surface is polished quicker, and equilibrium phase is reached sooner. Then, if seasonal variation are not considered, skid resistance equilibrium is maintained if the surface is not deteriorated further or the heavy traffic volume changes. The arrival of the equilibrium phase depends on the number of equivalent single axle loads and the type of surfacing and the petrologic properties of the aggregates. Heavy traffic is said to be responsible of polishing away the fine-scale microtexture, and a higher heavy traffic volume means a lower skid resistance (Rogers and Gargett, 1991). Fig. 5.39 shows these ideas based on a research carried out in a standard UK freeway surfacing consisting of rolled asphalt with pre-coated chipping, PSV 58-60 (Salt 1977a, Salt 1977b).

As shown, the initial drop of the SCRIM coefficient value is due to the polishing phase, but it does not continue after equilibrium phase is reached. Therefore, the heavy traffic effect must not be considered accumulated year after year, being only dependent on the heavy traffic intensity (and the aggregates properties), if weather conditions remain inalterable (Szatkowski and Hosking, 1972; Achútegi Viada 2005; Wilson, 2006; Pérez-Acebo *et al.*, 2017a).

However, if traffic load conditions change, i.e. heavy traffic intensity, available skid resistance also changes, even increasing its value. In Colnbrook (UK), the skid resistance of the A4 road got better when a stretch of the M4 freeway was opened to traffic in the surrounding and the last one got part of the traffic volume from the former (Fig. 5.40) (Szatkowski and Hosking, 1972).

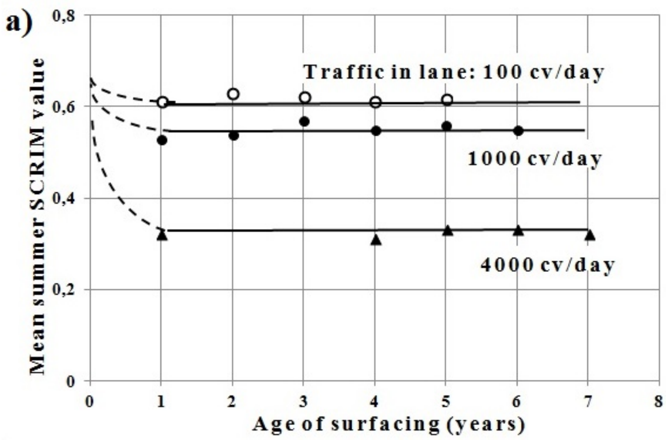


Fig. 5.39. Sideway force coefficient variation with heavy traffic volume, expressed in commercial vehicles (weight over 1.500 kg) (Szatkowski and Hosking, 1972).

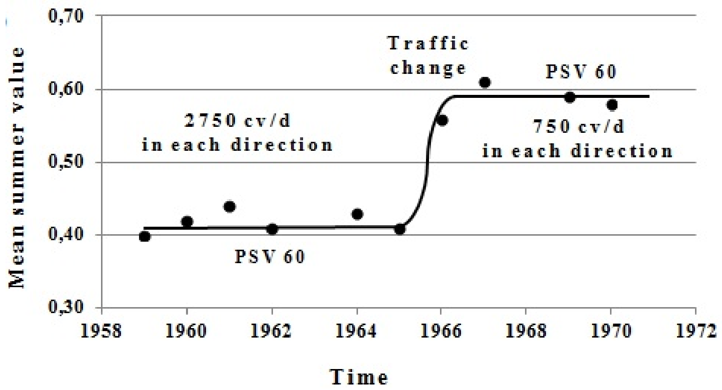


Fig. 5.40. Skid resistance variation with changing traffic volume (Szatkowski and Hosking 1972),

This phenomenon, which was identified by the TRRL in UK, it has also verified in Spain. A higher SFC was observed in the national road N-IV (Madrid – A Coruña) in the province of León, when the new motorway was constructed, A-6, and took the majority of the heavy traffic and the national only collected local traffic (Achútegi Viada, 2005).

5.5. Skid resistance performance prediction models

Taking into account the factors affecting skid resistance when tyre and factors related to vehicle are constant, some skid resistance performance prediction models have been developed. They all try to predict the available friction as a function of some variables that really have influence on the values.

5.5.1. Research in the United Kingdom

One of the first models was developed at the UK Transport and Road Research Laboratory (TRRL), currently named as Transport Research Laboratory, by Szatkowski and Hosking (1972). Before 1970 a large number of measurements of skid resistance were made on road experiments in order to obtain a better understanding of the relationship between PSV and SFC. From 13 sites, 20 analyses were made due to the division in different traffic lanes or types of surfacing at some of the sites. Results of the measurements showed that the relation between PSV and SFC had the form (Eq. 5.65):

$$SFC = a + b \cdot PSV \qquad [5.65]$$

Where *a* and *b* are constants determined by the extent to which polishing takes place at the site. SFC is expressed in a range from 0 to 1, and PSV in a range from 0 to 100. Analysis showed that coefficient *b* does not vary much and had an approximated value of $1 \cdot 10^{-2}$, giving a practical way to obtain the relationship between PSV and SFC. A change of unit of PSV would mean a corresponding change in SFC of $1 \cdot 10^{-2}$. Moreover, as traffic can be regarded as the skid resistance reducing factor, equations of the form of Eq. 5.66, may be obtained:

$$SFC = C_1 + C_2 \cdot f(T) + C_3 \cdot PSV \qquad [5.66]$$

Where $f(T)$ is a function of traffic conditions, and C_1, C_2 and C_3 are constants. Nevertheless, Szatkowski and Hosking (1972) also derived a more complex function. For any given PSV, it can be expected that the SFC depends on the intensity of the polishing action at the site. The lower polishing action, the greater would be the expected SFC for a specific PSV and vice versa. This idea is shown graphically in Fig. 5.41, where lines are drawn to represent the different degrees of polish expected for different traffic conditions.

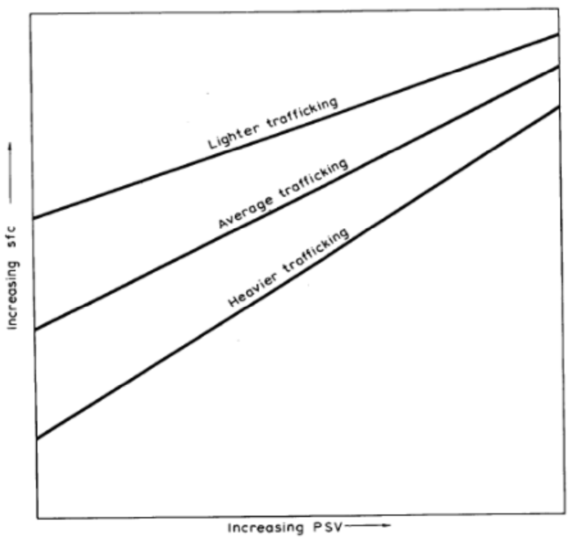


Fig. 5.41. Diagrammatic relation between SFC and PSV for different degrees of traffic (Szatkowski and Hosking, 1972).

Those lines are made to converge on a point representing a material of very high PSV that is not polished at all and, hence, the SFC would be the same for all traffic conditions. In this relationship, constant *a* and *b* of Eq. 5.67, would be related to traffic conditions (and, hence, to each other) and the general equation would become:

$$SFC = K_1 + K_2 \cdot f_1(T) + K_3 \cdot PSV \cdot f_2(T) \qquad [5.67]$$

Where K_1, K_2 and K_3 are constants and $f_1(T)$ and $f_2(T)$ are functions of traffic. Attempts were made to obtain an equation of this last form, but the lack of high-PSV aggregates resulted in anomalies. Obtained relationships showed a convergence to a point where PSV is about 55 and SFC about 0,51. It was concluded that the available data were not suitable for the reliable estimation of the relation between coefficient *b* and constant *a* and a simpler approach, as expressed in Eq. 5.68, was employed. The coefficient C_3 could be obtained by:

- Averaging the value of *b* obtained for all section. It resulted a value of $1{,}03 \cdot 10^{-2}$.
- Imposing that *b* corresponds to a line passing through the origin (If PSV and SFC is zero, *a* is also zero). It gave a value of $0{,}93 \cdot 10^{-2}$.

A value of $1 \cdot 10^{-2}$ was adopted because it was a more practical value to use.

With these ideas in mind, Szatkowski and Hosking (1972) carried out a regression analysis in order to correlate the three variables simultaneously. They conducted a survey of the SFC measurements taken by TRRL in the period 1960-1970. A site was considered if the surfacing was rolled asphalt or surface dressing and the PSV of the aggregates were known. SFC values were an average value of mean summer values, registered on 3 consecutive years. One hundred and thirty-nine sections were considered. Traffic was expressed as commercial vehicles (vehicles with a weight over 1500 kg). After a statistical analysis, this correlation was found with a correlation coefficient of 0,92:

$$SFC = 0{,}033 + 0{,}664 \cdot 10^{-4} \cdot Q_{CV} + 0{,}98 \cdot 10^{-2} \cdot PSV \qquad [5\ 68]$$

Where Q_{cv} is the traffic flow in commercial vehicles per lane and day and the other variables have been defined. As it can be observed, the estimate of the regression coefficient for PSV ($0{,}98 \cdot 10^{-2}$) agrees with the estimate ($1 \cdot 10^{-2}$) presented before, made for the site where a range of aggregates were used, showing that the data were representative with respect to PSV. They decided to round off the coefficient to the more convenient value of $1 \cdot 10^{-2}$, and the significance of the correlation was only slightly reduced (a correlation coefficient of 0,91), and the equation became:

$$SFC = 0{,}024 + 0{,}663 \cdot 10^{-4} \cdot Q_{CV} + 1 \cdot 10^{-2} \cdot PSV \qquad [5.69]$$

Where SFC is the Mean Summer SCRIM Coefficient measured by the SCRIM device at 50 km/h, PSV is the Polished Stone Value of the aggregates and Q_{cv} is the number of Commercial Vehicles (CV) per lane per day. In the UK, a CV was defined as a vehicle exceeding 1500 kg (15kN) mass.

Szatkowski and Hosking (1972) also proposed an equation (Eq. 5.70) for correlation with total traffic flow, expressed as total vehicles per lane and day (Q_{tv}).

$$SFC = 0{,}024 + 0{,}15 \cdot 10^{-4} \cdot Q_{tv} + 1 \cdot 10^{-2} \cdot PSV \qquad [5.70]$$

In this case, the coefficient of correlation was only 0,84 and, hence, this equation was not recommended.

The Eq. 5.69, from Transport and Road Research Laboratory Report LR504, was regarded as "*major advancement in the field of skid-resistance*" (Salt, 1977a; Cenek *et al.*, 2012). It was possible to predict the level of skidding resistance available, at its minimum from the value of PSV of the aggregates and the traffic flow (Fig. 5.42) (Szatkowski and Hosking, 1972). If identical surfacing materials are employed in different places, the level of SFC was found to be inversely related to the volume of heavy traffic (Salt, 1977a).

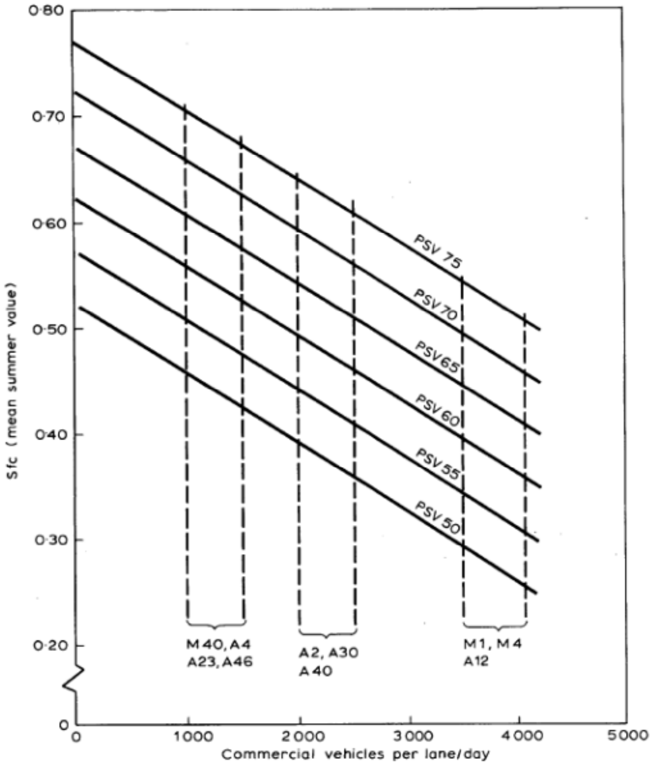


Fig. 5.42. Skidding resistance achievable on bituminous surfacing (surface dressing or rolled asphalt with aggregates of known PSV) under different traffic conditions (Szatkowski and Hosking, 1972).

Moreover, it also provided a method of nominating at the design stage the properties of the stone required to provide an ultimate skidding resistance given that the commercial traffic flow can be estimated (Salt, 1977a).

One conclusion extracted from commented researches was that the way that SFC and traffic were related. Unlike other processes that take place at the road, the effect of traffic on SFC is not cumulative from year to year. Consequently, concepts usually adopted, for example, in fatigue studies or in IRI progression do not apply to skid resistance. As shown, SFC is simply related to traffic volume for any aggregates of given PSV.

A useful explanation is provided by Salt (1977a) about this phenomenon, which does not imply a cumulative effect of the traffic:

> *"At the same time as traffic is tending to polish surface, other factor, usually identified as complex physico-chemical phenomena described as 'weathering', are acting in the opposite way, restoring microtexture of the exposed aggregate. Thus the resultant resistance to skidding represents equilibrium between the effects of certain naturally occurring conditions on the one hand and those of traffic on the other"*

As stated and explained in Fig. 5.40, if traffic flow changed, a corresponding change in SFC arises.

Eq. 5.69 was used as the basis for the standards for construction of new roads. The relationship was used to specify the required PSV of the aggregates to be used in surfacing in trunk roads in the Departmental Standard HD 16/76 in the Design Manual for Roads and Bridges (DMRB) of the United Kingdom. It was presented in the form of a table, where minimum PSV was indicated for given levels of traffic on different categories of sites. Later, following the introduction of in-service skid resistance standards, some requirements were modified in the Design Manual for Roads and Bridges (Highway Agency, 1999), but these requirements were still based on the original formula (Eq. 5.69).

Later, with the aim of providing information about skid resistance in correspondence with PSV in more heavily trafficked roads, a project was carried out by the Department of Transport of the UK (Roe and Hartshorne, 1998). For the skid resistance parameter, the Mean Summer SCRIM Coefficient (MSSC) was selected assuming that it would provide a reasonable approximation to the Equilibrium SCRIM Coefficient (ESC). Firstly, MSSC predicted by Eq. 5.69, was compared to the values recorded in practice. Fig. 5.43 shows the measured MSSC of different places, plotted against commercial vehicles flow per day and line (Q_{cv}). The values predicted from Eq. 5.69 are shown in line form for comparison.

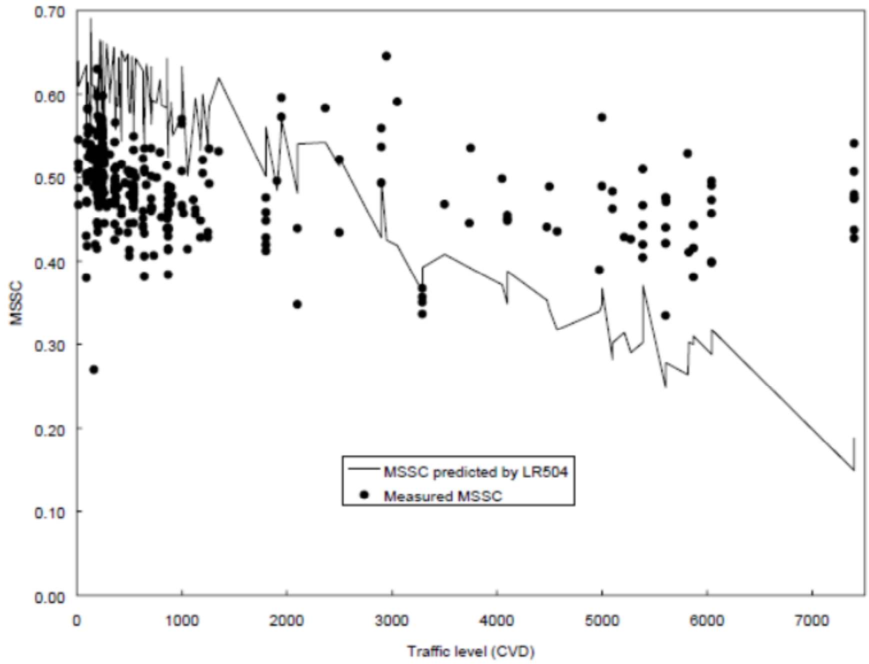


Fig. 5.43. Comparison of measured MSSC with predictions using Eq. 5.69 (LR504 model) (Roe and Hartshorne, 1998).

As observed in Fig. 5.43, Eq. 5.69 underestimates the MSSC achieved in practice at higher traffic levels, while, at lower traffic levels, it predicts consistently higher values that are actually found. In the central part of the traffic range, the model worked better, but there was still a wide spread of actual measured values. As a consequence, a new general relationship was tried to find. It was clear that the same aggregate could produce different skidding resistance for the same traffic level but under slightly different site conditions. Hence, it was probable that different polishing regimes were in operation at sites in different categories of road and it should be considered. Each site category defined in Table 3.1 of chapter of HD28 in the DRMB is assigned a risk rating and a corresponding investigatory level (IL) for MSSC. The investigatory level is defined as the MSSC below which investigation of the site and assessment of any need for remedial work is initiated. Several categories have the same investigatory level and can therefore be grouped into corresponding bands. Multiple regression analysis was carried out to explore alternative models, based on each investigatory level band separately. The IL bands are defined in Table 5.21.

Table 5.21. Definition of investigatory level bands (Roe and Hartshorne, 1998).

IL Band	Site categories	Investigatory Level
I	A, B	0,35
II	C, D	0,40
III	E. F. G1, H1	0,45
IV	G2	0,50
V	J, K	0,55
VI	H2	0,60 (at 20 km/h)
VII	L	0,55 (at 20 km/h)

As Szatkowski and Hosking (1972) had made, Roe and Hartshorne (1998) adopted *PSV* and Q_{CV} as independent variables and MSSC as dependent variable. They found that more of the variation could be explained transforming the Q_{CV}, and they employed the natural log of Q_{CV}, $\ln(Q_{CV})$. They proposed formulae of the form of Eq. Eq. 5.71 for each IL band:

$$MSSC = A \cdot PSV - B \cdot \ln(Q_{CV}) + K \qquad [5.71]$$

Where *A*, *B* and *K* are coefficients. The proposed coefficient for each IL band are summarized in Table 5.22. There were insufficient data for bands VI and VII.

Table 5.22. Coefficients for the Eq. 5.71 for individual IL bands (Roe and Hartshorne, 1998).

Band	IL	A (x 10^{-2})	B (x 10^{-2})	K	No of sections	R^2
I	0,35	6,18	2.25	0,252	2431	0,11
II	0,40	3,90	1,95	0,377	4073	0,23
III	0,45	2,94	1,70	0,407	1749	0,09
IV	0,50	5,81	1,46	0,193	82	0,11
V	0,55	4,73	0,98	0,231	43	0,08

As observed, PSV and traffic volume are still a key feature but, from the R^2 values, these factor can only predict about the 10 % of the total variation. Nevertheless, these coefficients and formulae were regarded as appropriate to be deployed in models because they involved the 2 factors that were available when designing a roads. The constant *K* is the dominant factor of the models. It comprises the influence of different factors that are not identified. The way in which these factors affect the skid resistance is unknown. The models created with Eq. 5.71 for each band (with PSV values ranging from 70 to 50) were compared graphically to Eq. 5.69 with a PSV of 60 (Fig. 5.44).

These results confirmed that both polishing resistance of the aggregates and the level of high traffic influence the equilibrium skid resistance. The results also underlined that the polishing action of traffic is a cause for skid resistance falling and that higher PSV of aggregates provide a higher skid resistance for the same traffic level. Nonetheless, a generalised relationship is not established that can reflect present-day conditions (Roe and Hartshorne, 1998). Other conclusions pointed out that in some situations, lower PSVs than specified could be used, while in other situations requirements were inadequate. It was also highlighted that materials were over-specified in regulations and a limited number of types of aggregates were used (Roe and Hartshorne, 1998).

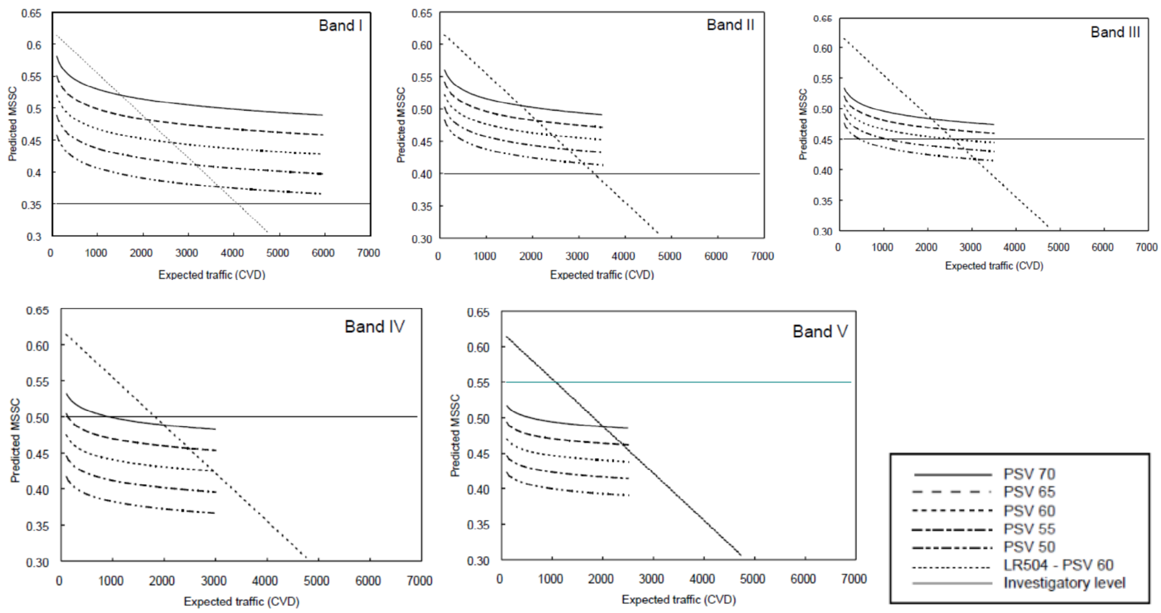


Fig. 5.44. Predicted MSSC for different PSV levels and IL bands and comparison with Eq. 5.71 with a PSV value of 60 (Roe and Hartshorne, 1998).

5.5.2. Research and standards in New Zealand

When the road agency of New Zealand, New Zealand Transport Agency (NZTA), formerly called Transit New Zealand (TNZ) was preparing a predictive specification policy for skid resistance (Transit New Zealand, 2002a), a SCRIM survey on national state highways was conducted in 1995 (WDM Ltd., 1998). Variables used in this study were traffic data, aggregate quarry source, PSV and site location, and it was reported a similar equation in New Zealand (Eq. 5.72) to that one obtained in the UK (5.69), which was the main reference in that moment:

$$SFC_{50} = 0{,}018 - 0{,}311 \cdot 10^{-4} \cdot CVD + 0{,}637 \cdot 10^{-2} \cdot PSV \qquad [5.72]$$

Where SFC_{50} is the Mean Summer SCRIM Coefficient measured at 50 km/h (in decimal fraction), CVD is the number of commercial vehicles per lane and day (in New Zealand, a commercial vehicle is considered when its mass is over 35 kN mass (3.500 kg), and PSV is the Polished Stone Value (from 0 to 100). This equation had a determination coefficient (R^2) of 0,28, which was considerably lower than the TRRL model (Eq. 5.69). In the research, it was noted that R^2 coefficient could be improved including the chip size in the prediction, up to 0,43, but still far from $R^2 = 0{,}83$. Even more, when comparing values predicted by the TRL equation (Eq. 5.69 to real SFC values, it was found that the present values were lower than the predicted ones (Fig. 5.45). The mean value of real data was 0,453 and the mean of the predicted values was 0,562.

Different reasons were stated for the poor correlation of the Eq. 5.72 and the differences with Eq. 5.69 (Fig. 5.45), such as errors in commercial vehicles values, variation in PSV and geological properties of different aggregates, climatic variations, differences of masses in commercial vehicles.

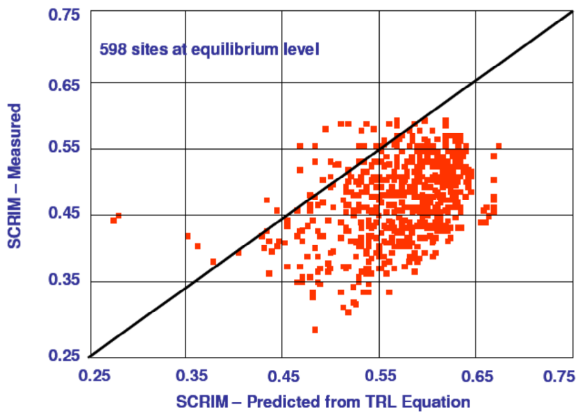


Fig. 5.45. Predicted SCRIM values from TRL equation (Eq. 5.69) vs. Real SCRIM data from New Zealand (Haydon, 2005).

Moreover, UK data was obtained in straight segments, whereas NZ data included curves and stressed sections. Consequently, in order to compensate the additional polishing that suffered NZ aggregates, in the research an increase of 5 PSV units was recommended. Therefore, Eq. 5.73 was incorporated as the final predictor equation of the NZ skid resistance policy in the TNZ T/10:2002 (Transit NZ, 2002a).

$$PSV = 100 \cdot ESC_{50} + 0{,}00663 \cdot CVD + 2{,}6 \qquad [5.73]$$

Where PSV is the Polished Stone Value of aggregate (from 0 to 100), CVD is the flow of commercial vehicles (as defined in NZ) per lane per day, and ESC_{50} is the Equilibrium Skid Resistance Coefficient. The last one is the sideway force coefficient as measured by SCRIM device corrected for Mean Summer SCRIM Coefficient (MSSC) and for yearly variations, at a measuring speed of 50 km/h. SCRIM Readings in New Zealand are corrected by the Index of SFC (0,78) like in the UK. In the original formula (Transit NZ, 2002a), instead of ESC_{50}, it says SR, which is the Investigatory Level (IL) for the site, i.e. the minimum required value for that place. There are 2 levels; the Investigatory Level (IL) and the Threshold Level (TL) (Table 5.23). The IL is a skid resistance warning level. If the skid resistance is below this value and investigation is required. If the ESC_{50} is bellow TL, measures must be taken (Table 5.24). The TL is defined as sites that have a value of 0,1 below the IL.

ESC_{50} values are expected to be over IL specifications, and, therefore, this value should be introduced in Eq. 5.73. As conducted research was supposed to predict ESC, the skid resistance policy established the required minimum PSV of aggregates in order to obtain a desirable ESC.

Moreover, it is established that the CVD used in Eq. 5.73 is the expected flow at the end of the surfacing's life (Transit NZ, 2002b), which can be calculated by Eq. 5.74.

$$CVD_F = \left(1 + \frac{i}{100}\right)^n \cdot CVD_P \qquad [5.74]$$

Where CVDF is the future CVD (the expected flow of commercial vehicles per lane per day at the end of the surfacing's life, CVDP is Present CVD, the current flow of commercial vehicles per lane per day *i* the expected percent traffic growth and n the number of years expected life.

Table 5.23. Investigatory Skid Resistance Levels (Transit New Zealand, 2002a)

Site category	Site definition	Investigatory Level (IL)	Threshold Level (TL)
1	Approaches to: • Railway level crossing • Traffic lights • Pedestrian crossing • Roundabouts • Stop and Give Way controlled intersections (where the State Highway is required to stop or to give way) • One lane bridges (including bridge deck)	0,55	0,45
2	• Curve < 250 m radius • Down gradients > 10 %	0,50	0,40
3	• Approaches to road junctions (on the State Highway or side roads) • Dow gradients 5 – 10 % • Motorway junction are including On/Off Ramps	0,45	0,35
4	• Undivided carriageways (event – free)*	0,40	0,30
5	• Divided carriageways (event – free) *	0,35	0,25

* Event-free: Where no other geometrical constraint, or situation where vehicles may be required to brake suddenly, may influence the skid resistance requirements

Table 5.24. ESC values and actions to perform according to the IL and TL (Transit New Zealand, 2002a).

ESC value	Definition	Action
High	Values of ESC above the IL	No action required in terms of this specification
Medium	Values of ESC between IL and the threshold level (TL)	Inspect and prioritise for future maintenance within the annual programme
Low	All values of ESC below the TL	These sites must be investigated, the cause determined, the appropriate treatment designed and programmed as soon as practicable as part of routine highway maintenance

Moreover, transformation formulae were displayed for friction measurement conducted with other testers (Transit NZ, 2002b). For GripNumber (GN) measured using 0,25 mm water film depth at a 50 km/h survey speed Eq. 5.75 may be used:

$$ESC = 0{,}42 \cdot GN + 0{,}20 \qquad [5.75]$$

For Norsemeter ROAR data, where μ is measured using fixed slip ratio of 34 % and 0,5 mm of water film depth at a 50 km/h survey speed, Eq. 5.76:

$$ESC = 0{,}55 \cdot \mu + 0{,}12 \qquad [5.76]$$

For British Pendulum Tester, where British Pendulum Number (BPN) is measured according to TRL note 27 (TRRL, 1969), Eq. 5.77.

$$ESC = 0{,}0071 \cdot BPN + 0{,}033 \qquad [5.77]$$

A later research conducted by Cenek *et al.* (2003) found that Eq. 5.73 predicted in-flied measured skid resistance very poorly, with a coefficient of determination (R^2) of 0,08, noting that Eq. 5.73 overestimated expected levels of skid resistance. Consequently, results showed that aggregates were not performing as expected and safety levels on road networks were lower than expected. Cenek *et al.* (2003) included seal chip grade in the analysis and R^2 raised up to 0,35 (Eq. 5.78).

$$MSSC = 0{,}0013 \cdot PSV + 0{,}1 \cdot e^{CHCV} - 0{,}007 \cdot ALD + 0{,}44 \qquad [5.78]$$

Where ALD is the average least dimension of the sealing chip in mm and CHCV is the cumulative heavy commercial vehicle traffic per lane in millions, calculated by Eq. 5.79:

$$CHCV = 0{,}0003 \cdot HCV \cdot AGE \qquad [5.79]$$

Where HCV is the commercial vehicle traffic (as described per NZ) per lane per day and AGE is the surface age in years

Since it found that Eq. 5.73 was not suitable to select aggregates in sections with higher polishing stress demand, further research were carried out, where statistical modelling studies were undertaken to establish whether or not a categorical parameter, representing the quarry from which the aggregates were sourced, could improve the model (Cenek *et al.*, 2012). Selected variables for analysis are listed in Table 5.25.

Considered road surface types are only single and two chipseals, 1CHIP and 2CHIP, respectively, which are the prevalent seal types used on the New Zealand state highway network.

Table 5.25. Variables selected from the Road Assessment and Maintenance Management (RAMM) database used in the national analysis (Cenek et al., 2012).

Variable	Restrictions	Comments
SCRIM coefficient	none	Not MSSC or ESC
Age (months)	24 < age < 240	The minimum was established to analyze Equilibrium Phase
Average Daily Traffic (ADT)	100 < ADT < 30.000	It was not used explicitly as a variable.
Traffic	Traffic < 1.260.000	Calculated as Age x ADT
Pavement source	> 1000 observations	At least 1000 observations from the same quarry to be included in the analysis
Gradient	Non	Uphill are positive and downhill negatives
Curvature	10 < curvature < 32000	Radius of the horizontal curvature (m)
PSV		Not included in the analysis as a variable. Only for comparing different pavement sources
Skid site	-	Site categories established in T10:2002 (Transit NZ, 2002a)
Surface function	-	1st coal, 2nd coal or reseal
Surface material	1CHP or 2 CHIP	

A single coat (1CHIP) is a single application of sealing binder follower immediately with a single application of chip, which is spread and rolled into place (Fig. 5.46a). A two cota (2CHIP) is a chipseal with two applications of binder and two applications of chip (Fig. 5.46b).

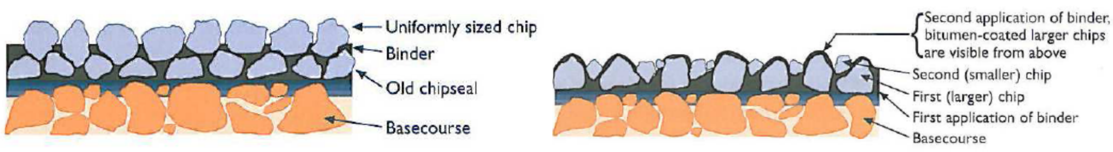


Fig. 5.46. a) A single coat seal shown as a reseal, b) Two coat seal shown as first coat (Transit NZ, 2005).

From the statistical analysis some conclusions were extracted:

- The statistical modelling performed on SCRIM coefficient (SC) of sealed section identified that pavement aggregate source, horizontal curvature and traffic (calculated as ADT x surfacing age= have the largest influence on in-service resistance, followed by sealing chip size.
- Pavement aggregate source, i.e. the quarry, showed to have the main parameter altering the skid resistance.
- The correlation between SC and PSV was not very strong. The results suggested that there was at least another factor that is accounted for by the categorical variable “source” but not by the quantitative variable PSV. It was suggested that this factor may be related to the shape/abrasion resistance of the sealing chip.
- It was recommended to use a spreadsheet to calculate the required PSV. The spreadsheet is called “Aggregate selection for skid resistance” and is available at www.nzta.govt.nz/resources/research/reports/470/index.html. Cenek *et al.*, 2012

As a consequence of this research, skid resistance policy was modified. In the new T10:2013 (NZTA, 2013a), there some changes.

With regard to the investigatory levels, they are exposed in Table 5.26, according to sites descriptions.

Table 5.26. Skid resistance investigatory levels (NZTA, 2013a)

Site category	Skid site description	Investigatory level (IL), units ESC					
		0.35	0.40	0.45	0.50	0.55	0.60
1	Approaches to: a) Railway level crossings b) Traffic signals c) Pedestrian crossings d) Stop and Give Way controlled intersections (where state highway traffic is required to stop or give way) e) Roundabouts. One lane bridges: a) Approaches and bridge deck.						
2	a) Urban curves <250m radius						
	b) Rural curves <250m radius			L	M	H	
	c) Rural curves 250-400m radius		L	L	M	H	
	a) Down gradients >10%. b) On ramps with ramp metering.						
3	a) State highway approach to a local road junction. b) Down gradients 5-10% c) Motorway junction area including on/off Ramps d) Roundabouts, circular section only.						
4	Undivided carriageways (event-free).						
5	Divided carriageways (event-free).						

Minimum macrotexture is required in order to minimise the progressive loss of skid resistance with increasing speed on wet roads and to prevent or minimise the loss of skid resistance due to contact between vehicle tyres and bitumen. Table 5.27 list the required MPD (in mm) according to speed limit

Table 5.27. Required minimum macrotexture requirements (NZTA, 2013a)

Minimum macrotexture – mean profile depth (MPD mm)						
Permanent speed limit	**Chipseal**		**Asphaltic concrete, ESC ≥ 0,4**		**Asphaltic concrete, ESC < 0,4**	
	ILM	**TLM**	**ILM**	**TLM**	**ILM**	**TLM**
50 km/h or less	1,0	0,7	0,4	0,3	0,5	0,5
Less than or equal to 70 km/h but > 50 km/h	1,0	0,7	0,4	0,3	0,7	0,5
Greater than 70 km/h	1,0	0,7	0,9	0,7	0,9	0,7

The fundamental requirement for new surface selection continues being the same. Aggregates must be selected with the objective of maintaining the skid resistance over the IL during all the design life of the surfacing. However, in the new version, there are two methods for selection of surfacing aggregate

1) Aggregate performance method: a more rigorous method that ensures the most economic choice of aggregate for surfacing (the preferred one)
2) PSV method: the simplest design method.

The first method consists of the following steps:

1) Using data about the polishing of aggregate (*SC* or *ESC* achieved during its life) from the survey data, produce a matrix of aggregate performance in a variety of polishing stress situation normalised for heavy traffic.
2) Produce a list of aggregates commonly used in the area, ordered by their polishing resistance and select aggregates according to requirements of Table 5.28.
3) Assess the traffic and polishing stress and select appropriate aggregates.

On the contrary, the polished stone value method only implies using Eq. 5.80 to indicate the PSV required for the aggregate.

$$PSV = 100 \cdot PSV + 0{,}00663 \cdot HCV + PSF \qquad [5.80]$$

Where *SR* is the investigatory level for site (Table 5.26), *HCV* is the estimated heavy commercial vehicles per lane per day at the end of the surfacing life, *PSV* is the Polishes Stone Vale required for aggregates and *PSF* is the polishing stress factor selected for site according to Table 5.28.

As seen, in the new equation (Eq. 5.80), apart from the polishing effect of heavy traffic (incorporated in Eq, 5.73 and in 5.69), a new factor dependent on various is introduced. Eq. 5.73 was derived from reasonably low polishing stress sections of surfacing and was found unsuitable for higher stress section. To overcome it, the constant 2,4 of the precedent equation 5.73 was replaced by a polishing stress factor that ranges from 3 to 9.

On the other hand, when conducting the aggregate performance, a polishing stress factor (*Polishing Stress*) must be calculated, following Eq. 5.81:

$$Poloshing\ Stress = 0{,}00663 \cdot HCV + PSF \qquad [5.81]$$

Table 5.28. Polishing stress factors (NZTA, 2013a)

Polishing stress factor	Site description
3	Site category 5, event-free: where no other geometrical constraint, or situations where vehicles may be required to brake suddenly, could increase the skid resistance requirements. Site category 4, straight level road, less than 5,000VPD, very seldom any congestion and few low-volume access points.
4	Site category 4, greater than 5,000VPD, very seldom any congestion, grades < 3% Site category 3a, state highway approach to a local road junction. Low-risk curves.
5	Site category 4, where congestion may occur or grades ≥ 3% Site category 3b, down gradients 5–10%. Site category 3c, including 200m before off ramps. Urban site category 2 curves.
6	Site category 1, with average approach speeds and infrequent emergency braking. Rural site category 2, curves medium risk.
7	Site category 1, with average braking Rolling country with frequent curves requiring frequent acceleration and deceleration
8	Site category 1, with frequent heavy braking Site category 2, curves that are high risk. Any site with frequent heavy braking, e.g. curve requiring braking at end of down grade.
9	Sections of the network with highest stress due to braking or cornering eg curve requiring braking at end of steeper down grade (≥8%).

As observed, the polishing stress is composed of the effect of heavy traffic and a factor dependent on curve radius, grades, probability of congestion and braking, situation of the road, etc (Table 5.27). The PSV method is easier to implement that the aggregate performance method, but it is not as accurate (NZTA, 2013b).

5.5.3. Research in Texas (USA)

At the Texas A&M University, it was developed a model that combines previous skid resistance prediction in laboratory as a function of aggregate characteristics and gradation and later, it was verified in situ (Mahmoud and Masad, 2007; Masad *et al.*, 2007; Rezaei *et al.*, 2009; Rezaei *et al.*, 2011; Rezaei and Masad, 2013; Kassem *et al.*, 2013; Masad *et al.*, 2009a; Masad *et al.*, 2009b). Hence, the models were based on both laboratory and field measurements in correspondence with surface characteristics. Rezaei and Masad (2013) related the complete process of the model, which includes two phases.

The first phase consisted of two steps for obtaining a skid resistance model that predicts the performance based on polishing cycles in the laboratory based on aggregate characteristics and aggregate gradation (Rezaei *et al.*, 2009). Aggregate characteristics were based on the following tests:

- Aggregate image system (AIMS). As previously explained, AIMS is an automated imaging system that can determine the angularity, shape and texture of coarse aggregates and the shape and texture of fine aggregates by means of a scanning system and digital image processing. It has been shown that AIMS is able to measure aggregate texture less than 0,5 mm down to 10 microns (Al-Rousan, 2004). Rezaei *et al.* (2009) showed that AIMS can be used before and after polishing in the Micro-Deval and describe the skid resistance of asphalt pavement and aggregate polishing susceptibility.

- The Micro-Deval test following the ASTM D6928 procedure (ASTM, 2017a) was applied to polish aggregates for two different time periods: 105 and 180 minutes. Aggregate texture was measured after those periods by AIMS.
- The Dynamic Friction Test (DFT) was used according to the ASTM E1911-98 procedure (ASTM, 2002). It was used at 20 km/h for microtexture measurements (Hall *et al*., 2009).
- The Circular Texture Meter (CTMeter) as employed following the ASTM E2157 procedure (ASTM, 2009b). It calculated the mean profile depth (MPD) dividing the data into eight arcs.
- Slabs from different combinations of aggregates and mixtures were polished in the NCAT polishing machine, originally developed by the National Center for Asphalt Technology (NCAT). DFT and CTMeter measurement were carried out after each polishing cycle (105 and 180 minutes).

Rezaei *et al*. (2009) employed these tests after also analyzed the sand patch test and British Pendulum Test, which were discarded. Sand Patch test was not able to detect changes in macrotexture before and after polishing and the British Pendulum provided a high variability and did not allow detecting the frictional differences for the analyzed mixtures. They used the equation proposed by Mahmoud and Masad (2007), Eq. 5.82, to describe texture as a function of polishing time (*t)* in minutes,

$$Texture(t) = a_{agg} + b_{agg} \cdot \exp(-c_{agg} \cdot t) \quad [5.82]$$

Where a_{agg}, b_{agg} and c_{agg} are regression constants, they can interpreted as the terminal, initial and rate of texture change, respectively.

Aggregate gradation is a key factor that affects macrotexture, and, hence, Rezaei *et al*. (2009) employed a two-parameter Weibul distribution (Eq. 5.83) to describe the aggregate gradation, fitting the standard aggregate size distribution:

$$F(x;\lambda,\kappa) = 1 - e^{-(x/\lambda)^{\kappa}} \quad [5.83]$$

Where x is aggregate size in mm, and κ and λ are referred to as the shape and scale parameters of the Weibull distribution, respectively. These two parameters change according to the mixture gradation.

Using the DFT and CTMeter data, the International Friction Index (*IFI*) can be obtained based on ASTM E1960-07 (ASTM, 2007) with their corresponding coefficient by means of Eq. 5.84 and 5.85.

$$IFI = 0{,}081 + 0{,}732 \cdot DF_{20} \cdot \exp\left(\frac{-40}{S_p}\right) \quad [5.84]$$

$$S_p = 14{,}2 + 89{,}7 \cdot MPD \quad [5.85]$$

Where DF_{20} is the measured dynamic friction at 20 km/h and *MPD* is the mean profile depth value by CTMeter and S_p is a value adopted in the IFI model. Obtained values for different cycle time are exposed in

Fig. 5.47. A formula (Eq. 5.86) is proposed to relate IFI to skid resistance measured using a smooth tyre skid trailer.

$$IFI = 0{,}045 + 0{,}925 \cdot 0{,}01 \cdot SN(50) \cdot \exp\left(\frac{20}{S_p}\right) \qquad [5.86]$$

Where *SN*(50) is the skid number measured by a smooth tyre tow trailer at 50mph (80 km/h).

Then, as a second step of this phase, a similar equation to the one that was developed by Mahmoud and Masad (2007), Eq. 5.82, used to describe aggregate texture loss was employed to describe the changes in IFI values and Rezaei and Masad (2013) obtained Eq. 5.87.

$$IFI(N) = a_{mix} + b_{mix} \cdot \exp(-c_{mix} \cdot N) \qquad [5.87]$$

Where a_{mix}, b_{mix} and c_{mix} are the terminal, initial and rate of change in IFI, and N is the number of polishing cycles, expressed in thousands (e.g. N = 50 for 50.000 polishing cycles). The regression coefficients of Eq. 5.87 were calculated and Eq. 5.87 was found to accurately predict the IFI functions for all mixes. Regression coefficients are listed in Table 5.29. (Rezaei *et al.*, 2009)

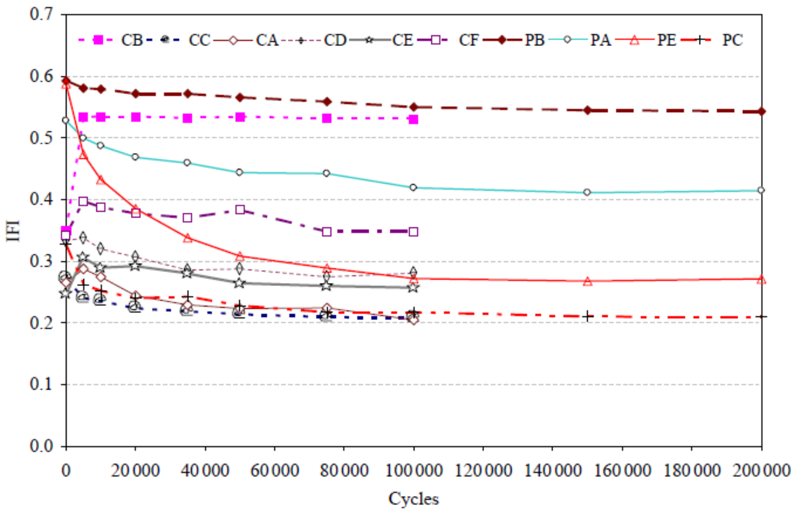


Fig. 5.47. Calculated IFI values for different mixes (Rezaei and Masad, 2013).

Table 5.29. Regression coefficients for the model developed by Rezaei et al. (2009).

Mix type	a_{mix}	b_{mix}	c_{mix}	R^2
CA	0,213	0,097	0,050	0,96
CB	0,474	0,061	0,001	0,98
CC	0,212	0,058	0,102	0,96
CD	0,275	0,065	0,038	0,95
CE	0,250	0,060	0,023	0,95
CF	0,225	0,170	0,003	0,87
PA	0,279	0,0288	0,055	0,97
PB	0,539	0,048	0,013	0,97
PC	0,221	0,117	0,155	0,90
PE	0,416	0,101	0,025	0,95

The coefficients of Eq. 5.87 were related to the aggregate texture coefficients (Eq. 5.82) and the gradation parameters (Eq. 5.83) and the obtained statistical model is described by Eq. 5.88, Eq. 5.89 and Eq. 5.90.

$$a_{mix} = \frac{18{,}422 + \kappa}{118{,}936 - 0{,}0013 \cdot (AMD)^2} \qquad R^2 = 0{,}96 \qquad [5.88]$$

$$a_{mix} + b_{mix} = 0{,}4984 \cdot \ln\left[5{,}656 \cdot 10^{-4} \cdot \left(a_{agg} + b_{agg}\right) + 5{,}846 \cdot 10^{-2} \cdot \kappa - 4{,}985 \cdot 10^{-2} \cdot \lambda\right] \qquad R^2 = 0{,}82 \qquad [5.89]$$

$$c_{mix} = 0{,}765 \cdot e^{\left(\left(-7{,}297 \cdot 10^{-2}\right)/c_{agg}\right)} \qquad R^2 = 0{,}90 \qquad [5.90]$$

Where *AMD* is aggregate texture measured by AIMS after 105 min of polishing in Micro-Deval, $a_{agg}+b_{agg}$ and c_{agg} are the initial and rate of texture change for corresponding aggregate, and κ and λ are Weibull distribution shape and scale factor, respectively.

The steps described in this first phase can be summarized in Fig. 5.48.

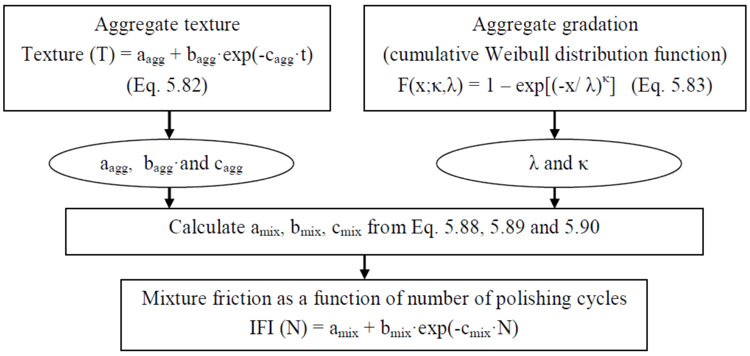


Fig. 5.48. IFI calculation procedure. (Rezaei et al., 2009)

In a second phase, Rezaei and Masad (2013) developed a skid resistance prediction model, which depends on the aggregate texture and gradation and traffic volume from field measurements, i.e. the IFI is expressed as a function of traffic volume instead of polishing cycles. Field measurements were performed on 64 pavements sections, covering a wide range of aggregates and mixtures types, similar to the ones studied in the laboratory phase. Selected sections had a complete skid resistance database. Various traffic volumes were also represented in those road segments.

Rezai and Masad (2013) employed 3 steps. Firstly, they substitute the CTMeter and DFT field measurements in Eq. 5.84 and 5.85 so as to obtain the IFI for the in situ section. Secondly, IFI must be expressed function of number of polishing cycles using Eq. 5.88 through 5.90. Finally, the skid resistance model must predict IFI as a function of traffic volume, instead of polishing cycles. They defined the Traffic Multiplication Factor (TMF) as shown in Eq. 5.91.

$$TMF = \frac{AADT\left(for\ outer\ lane\right) \cdot years\ in\ service \cdot 365}{1000} \qquad [5.91]$$

Where *AADT* is the annual average daily traffic for the mist critical lane in the highway, the outer lane. Contrary to the formulae proposed in the United Kingdom, it does not consider the effect of heavy traffic, supposing a similar effect of all the vehicles.

After carrying out the first two steps, they found the correlation between TMF and number of polishing cycles, N, Eq. 5.92, by means of a non-linear least-square regression analysis.

$$N = TMF \cdot 10^{\left(1/\left(A+B\cdot c_{mix}+\left(C/c_{mix}\right)\right)\right)} \qquad R^2 = 0{,}74 \qquad [5.92]$$

Where A, B and C are regression coefficients and have these value of -0,452, -58,95, and 5,834 · 10^{-6}, respectively. It must be highlighted that he relationship between N and TMF is dependant of mixture polishing characteristics, c_{mix}. Consequently, IFI can be predicted as a function of TMF by Eq. 5.93.

$$IFI\,(TMF) = a_{mix} + b_{mix} \cdot \exp(-c_{mix} \cdot TMF \cdot 10^{\left(1/\left(A+B\cdot c_{mix}+\left(C/c_{mix}\right)\right)\right)} \qquad [5.93]$$

Moreover, a most highway agencies, especially in the USA, prefer to use skid number instead of IFI, Rezaei and Masad (2013) calculated skid number at 50 mph (80 km/h) from Eq. 5.86. Given the IFI values of the pavement sections, they compared them with the SN(50) values reported by the Texas Department of Transportation (TxDOT) for the same sections. It was found that the measured values of SN(50) are correlated with but greater than those values calculated using the equations and coefficients proposed by PIARC (1995) (Eq. 5.84, Eq. 5.85). Therefore, the data were modified to the equality line as observed in Fig. 5.49, and Eq. 5.94, obtained by regression analysis, was recommended instead of Eq. 5.86.

$$SN(50) = 1{,}41 + 143{,}19 \cdot (IFI - 0{,}045) \cdot e^{(-20/S_p)} \qquad R^2 = 0{,}75 \qquad [5.94]$$

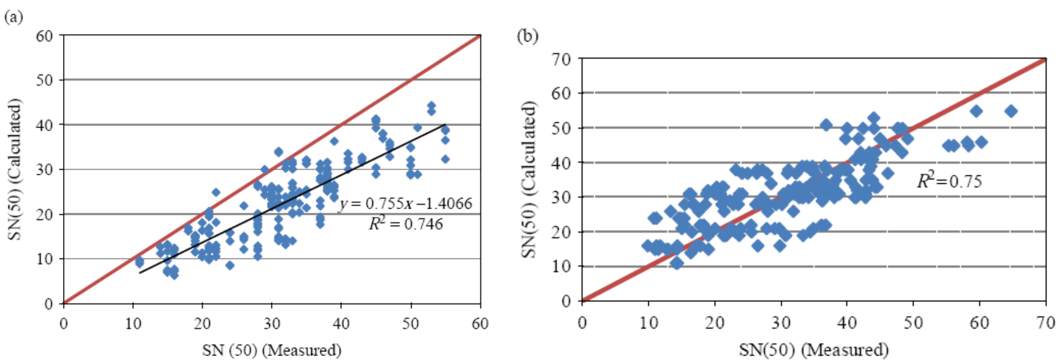


Fig. 5.49. Relationship between back-calculated SN(50) and measured SN(50), a) before shifting, b) after shifting (Rezaei and Masad, 2013).

Eq. 5.94 requires S_p, which is dependent on macrotexture. The macrotexture can be measured using the CTMeter. Nevertheless, it is simpler if the index is estimated from aggregate gradation and not using CTMeter field measurements. After statistical analysis, it was found that MPD, and consequently S_p, can be predicted by means of Eq. 5.95:

$$MPD_0 = 0{,}139 \cdot \lambda + 0{,}086 \cdot \kappa - \frac{0{,}041}{\kappa^4} \qquad R^2 = 0{,}79 \qquad [5.95]$$

Where κ and λ are Weibull distribution function coefficients. Therefore, the skid number, $SN(50)$ can be calculated using Eq. 5.94 and merging Eq. 5.85 and 5.95 as shown in Eq. 5.96:

$$S_p = 14{,}2 + 12{,}468 \cdot \lambda + 7{,}714 \cdot \kappa - \frac{3{,}678}{\kappa^4} \qquad [5.96]$$

As a conclusion, this model can be employed by road administrations to estimate the traffic levels at which the skid resistance of the pavement section could drop below acceptable levels (Rezaei and Masad 2013).

In the same research group of the Texas A&M University, an alternative model was developed, which was able to predict the loss of skid resistance as a factor of aggregate shape characteristics, mainly the aggregate angularity, the aggregate resistance to abrasion and polishing and gradation (Kassem *et al.*, 2013). Following the equation of Mahmoud and Masad (2007), Eq. 5.82, for describing the change in the texture, they presented a similar equation to describe the loss of aggregate angularity, Eq. 5.97:

$$GA(t) = a_{GA} + b_{GA} \cdot e^{(-c_{GA} \cdot t)} \qquad [5.97]$$

Where a_{GA}, b_{GA} and c_{GA} are regression constants and t is the polishing time in the micro-Deval test. In Fig. 5.50, some examples of these regression constants for the change in angularity are shown. The angularity of the aggregates was measured by means of the AIMS. It is an automated system that determines aggregate characteristics through image processing and analysis methods. The AIMS software provides aggregate shape measurements of the form, angularity and surface texture (Fig. 5.51). Angularity is quantified by measuring the irregularity of a particle surface from black-and-white images. The micro-Deval test, as in previous research was applied during 105 and 180 minutes.

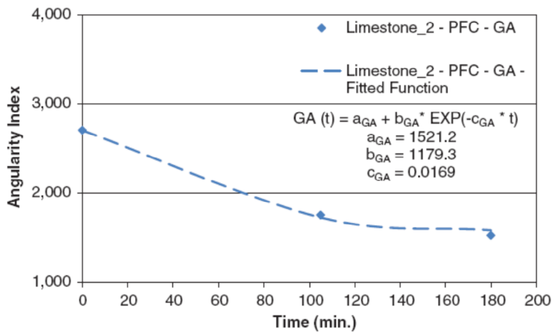


Fig. 5.50. Examples of regression constants for aggregate angularity versus micro-Deval time (Kassem et al., 2013).

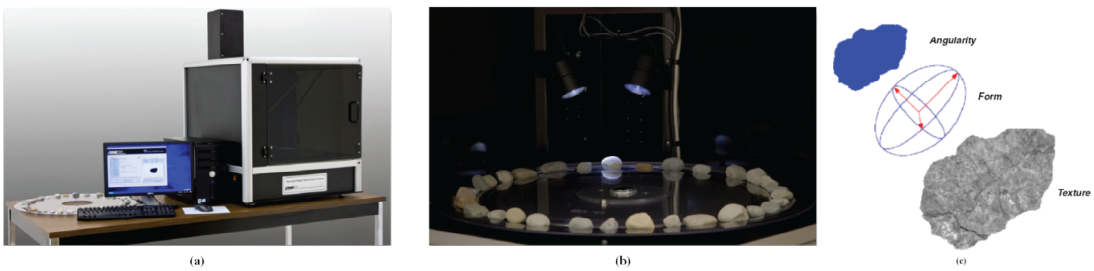


Fig. 5.51. AIMS analysis process of aggregate properties (Kassem et al., 2013).

Kassem *et al*. 2013 also employs the two-parameter Weibull distribution for describing the aggregate gradation, Eq. 5.83. Finally, similar to Eq. 5.87 (Rezaei and Masad, 2013; Masad *et al*., 2009a; Masad *et al*., 2009b), from the results of change in IFI values, obtained with the polishing cycles based on the MPD and

DFT20 measurements, Eq. 5.98 was proposed:

$$IFI(N) = a_{mix} + b_{mix} \cdot \exp(-c_{mix} \cdot N) \quad [5.98]$$

Where a_{mix}, b_{mix} and c_{mix} are regression coefficients which represent the terminal, initial and the rate of change in IFI, respectively, and N is the number of polishing cycles (in thousands) using the polisher.

This time, the regression coefficients were calculated by nonlinear regression analysis to the aggregate texture coefficients (Eq. 5.82), to the aggregate angularity coefficients (Eq. 5.97) and Weibull distribution parameters that describe the aggregate gradation (Eq. 5.83). Proposed equations are:

$$a_{mix} = \frac{47.493 + \lambda}{307{,}071 - 0{,}003 \cdot (AMD)^2} \quad [5.99]$$

$$(a_{mix} + b_{mix}) = 0{,}308 \cdot \ln\left(\frac{1{,}438 \cdot (a_{agg} + b_{agg}) + 46{,}893 \cdot \lambda + 333{,}491 \cdot \kappa}{2{,}420 \cdot (a_{GA} + b_{GA})}\right) + 1{,}008 \quad [5.100]$$

$$c_{mix} = 0{,}052 + 2{,}284 \cdot 10^{-14} \cdot e^{\left(\frac{0{,}523}{c_{agg}}\right)} + 2{,}008 \cdot 10^{-47} \cdot e^{\left(\frac{1{,}708}{c_{GA}}\right)} \quad [5.101]$$

Where AMD is the aggregate texture after 105 min of the micro-Deval test. The measured IFI value has a coefficient of determination (R^2) with the predicted IFI after applying exposed equation of 0,92 (Fig. 5.52). Coefficient of determination (R^2) of the coefficients a_{mix}, b_{mix} and c_{mix} obtained values from 0,89 to 0,96.

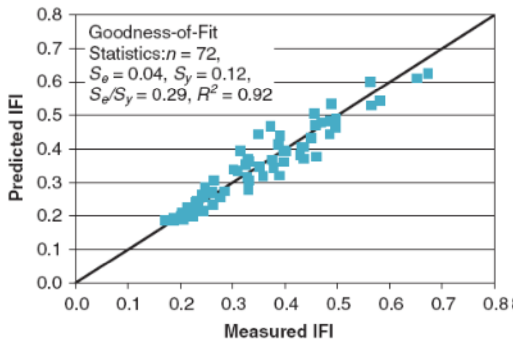


Fig. 5.52. Measured IFI versus predicted IFI (Kassem et al., 2013).

5.5.4. Models listed in the COST Action 354

As explained in section 5.3.1.2.3, COST Action 324 an extensive inventory was made about the pavement performance models that were in use in PMS in participating countries. Table 5.10 listed the number of models existing in each country for each pavement characteristic. As observed, only Hungary and the United Kingdom used performance models for skid resistance.

The skid resistance prediction model included in Table 5.10 refers to the Eq. 5.69 and hence, it is not further commented. In Hungary, there were some models to predicte the microtexture and the macrotexture:

$$MICRO = a - b \cdot AGE \quad [5.102]$$

$$MICRO = a - b \cdot FORG \quad [5.103]$$

$$MACRO = a - b \cdot AGE \quad [5.104]$$

$$MACRO = a - b \cdot FORG \quad [5.105]$$

Where *MICRO* is the microtexture, *MACRO* is the macrotexture, *AGE* is the age of the coarse surface and *FORG* is the number of vehicle repetitions expressed in unit of passenger cars.

5.5.5. Other researches

Khasawneh (2017) analyzed the polishing behaviour of laboratory prepared Hot Mix Asphalt specimens made of eight different mix formulas in terms of friction values by means of the British Pendulum Number. The aim of the research was to test a new asphalt polisher, which showed a good degree of repeatability with small variations. With regard to the polish effect, it was stated that the decrease in polish number is maximum during the first hour of polishing. With the passage of time the drop in BPN decreases and stabilizes after roughly 5 or 6 hours of continuous polishing (Fig. 5.53). This behaviour is similar to the one observed in field (5.38) and obtained in laboratory in other researches (Fig. 5.54).

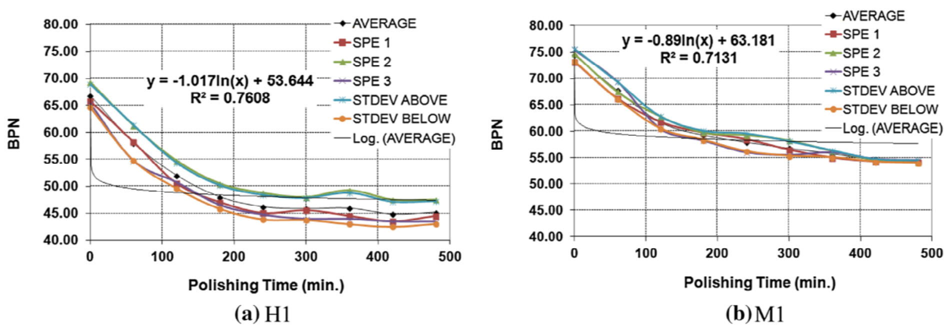


Fig. 5.53. Friction deterioration of asphalt mixes of different polishing susceptibility (Khasawneh, 2017).

5.6. Conclusions

Factor affecting roughness and skid resistance and proposed models to forecast its evolutions have been extensively commented in this chapter.

With regard to the roughness performance models, it is clear that its values and its progression are totally dependent on other pavement distresses. As stated by Paterson (1987), roughness progress due to deformation caused by traffic loading and rut depth variation; surface defects produced from cracking, potholes and patching and a combination of aging and environmental factors.

Therefore, Paterson (1987) developed a model that predicted the change in roughness as a function of the following factors: current condition, pavement strength and age characteristics, environment, incremental time and incremental traffic. From this idea, the HDM-III model for IRI progression was developed, and later, the HDM-IV. Other models also take into account these factors. In more simple modelling, some of this factors are eliminated an in others, they are calculated in different ways, but the essence of this idea remains in all of them, not being related to the model type. All types of models have included the majority of these factors, although in different ways. Deterministic models, both empirical and mechanistic-empirical models have principally deployed these factors, introducing them. It has been observed that better predictions are obtained when more factors are introduced, but more intense data collection is needed, increasing the cost.

Probabilistic models, which have also included these factors, can also capture the variability of the pavement behaviour and therefore, they are said to better represent their behaviour. However, this kind of models lack of the ability to be adapted to different circumstances or changes in some affecting factors' value and they are usually developed as family models. Bayesian, Artificial Neural Network and subjective models are also utilized for modelling roughness evolution and they can be as accurate as other models, but their main problem could be that they cannot broadly used in other regions or countries and need an intense calibration.

As observed, most of the times, the model selection depends on the available information and according to collected data one or other model is chosen.

Regarding to skid resistance, the performance is totally different. As observed from different researches, it does not get worse with time. After an initial friction improvement, a quick polishing phase arrives and after it, whose time interval can range from some mouths to 1 or 2 years, the pavement section arrives to an equilibrium phase where the values are said to be constant and only seasonal variations are registered, with greater values in winter and lower values in summer. This trend has been checked both in real pavements and in laboratory tests. Various models have been developed to model the seasonal variation, but the main interest is on predicting the equilibrium phase value. It seems reasonable that the peak values obtained in winter are not useful for road agencies as the values describe the situation when the friction is at its maximum. On the contrary, it seems logical that road-agencies want to predict the values at their minimum, to know the situation at its worst. Consequently, values obtained in summer, called differently in each country, are aimed to be predicted. If those values do not meet required standards, actions must be taken.

The most important model for skid resistance prediction was developed in the UK by the TRRL. The selected equation was considered as a reference and it forecasted available minimum friction as a function of daily traffic of commercial vehicles per lane and the Polished Stone Value of the aggregates. Later it was modified, but the new equation maintained the necessary factors (independent variables) of the original model.

This model was also used in other countries like in New Zealand and was adapted and calibrated to the specific characteristics of the country. Adapted formula was employed to aggregate selection, but it was also modified to better reflect the reality. Different variations were conducted but the essence of the British idea remain, the main factors that impact skid resistance performance in the long term are Polishing Stone Value of aggregates and heavy traffic volume.

Experimental tests have also been carried out, especially in the Texas A&M University. By means of elaborate equations it was possible to predict available friction after a great number of polishing cycles as a function of different properties of the aggregates: gradation, texture, etc. All the developed equations still reproduce the trend observed in field, after a number of polishing cycles, the available friction is constant.**Continuation of this PhD thesis in English is provided in Annex III.**

II. ATALA. DATU ESKURAGARRIAK ETA METODOLOGIA

6. Kapitulua. Bizkaiko Foru Aldundiko Bide-zoruak Kudeatzeko Sistema

6.1. Sarrera

Kapitulu honetan, Bizkaiko Foru Aldundiak garatutako Bide-zoruak Kudeatzeko Sistemaren (BKS) ezaugarriak deskribatzen dira. Bizkaiko Foru Aldundiak (BFA) Bizkaiko probintziaren hiri-arteko ia bide-sare osoaren bide-administrazio arduraduna da eta bide-zoruak kudeatzeko sistema bat garatu da, “Agenda de Estado” (Egoera Agenda) izenekoa, bide-sarearen etorkizuneko egoera aurreikusteko eta funts eskuragarriak hobeto esleitzeko.

Lehenik eta behin, Espainian eta Bizkaian errepideen kudeaketaren berrikuspen historiko labur bat azaltzen da. Bigarrenik, lurraldean dagoen bide-sareari dagokionez Bizkaiko Foru Aldundiak gaur egun dituen eskumenen ikuspegi orokorra aurkezten da. Eskumen horiek nazio eta autonomia mailako legeen bidez esleitzen dira. Behin eskumenak esleitzen zaizkionean, lege bat indarrean sartu zen, zeinek Bizkaiko Foru Aldundiak bere eskumenak nola garatzen dituen azaltzen den. Horrez gain, lege horretan dagoen errepide-sailkapena deskribatzen da.

Azkenean, erabilitako bide-zoruak kudeatzeko sistemaren ezaugarriak komentatzen dira. BFAk erabilitako BKS 2. kapituluan aurkeztutako ideiekin konparatzen da, non BKSek izan behar duten oinarrizko informazioa azaltzen zen. Hori dela eta, Egoera Agendak bide-zoruak kudeatzeko sistemen beharrezko baldintzak betetzen diren aztertzea posible da.

6.2. Bizkaiko bide-sarearen berrikuspen historikoa

Iberiar Penintsulako erromatar inbasioaren aurretik bide batzuk egon ahal izan arren, lehenengo bide-sare osoa erromatar inperioaren garian garatu zela esan daiteke. Iberiar Penintsulan zehar zeuden ibilbideak hobeto azaltzen dituen dokumentua Antonine Ibilbidea da (Itinerarum Antonini Augusti, latinez, literalki Antoninus enperadorearen ibilbidea). Dokumentu famatua da, geltokien eta hainbat errepideren luzeren erregistroa. Dokumentu ofizialetan oinarritzen dela dirudi, agian, Augusto enperadorearen agindupean egindako azterlan batetik eta Erromatar Inperioaren galtzada deskribatzen ditu. Mota honetako erregistroak eta dokumentuak eskasak direla eta, balio historiko handia dauka. Data eta egileari buruzko informazio gutxi jakiten da. Jatorrizko edizioa 3. Mendearen hasieran prestatua izan zela uste da, nahiz eta tradizionalki 2. Mendeko Antoninus Pius enperadorearen babesari atxikitakoa izan, dagoen kopia zaharrena Dioklezianoren garaikoa dela ezarri da.

Hispaniari, Iberiar Penintsulako probintzi erromatarren izena, dagokionez, komentatutako dokumentuan agertzen diren 372 ibilbideetatik 34 ibilbide Hispanian kokatzen dira. Antonine Ibilbidea dokumentuan deskribatutako ibilbideak 6.1 irudian erakusten dira. Ikusten den moduan, gaur egun Bizkaiko lurraldea kokatzen den lekuan ez dago erromatar galtzadarik. Hala eta guztiz ere, Antonine Ibilbidea dokumentuak

“Praetor” baten erregistroan sartutako bide nagusiak baino ez zituen aipatzen, gaur egungo baliokideak herrialde baten errepide nagusiak izango liratekeenak. Horren ondorioz, beste autore batzuek (Estrabon, Plinius Zaharra, etab.) aipatzen dituzten bide lokalak ez daude barne (Pérez-Acebo, 2018).

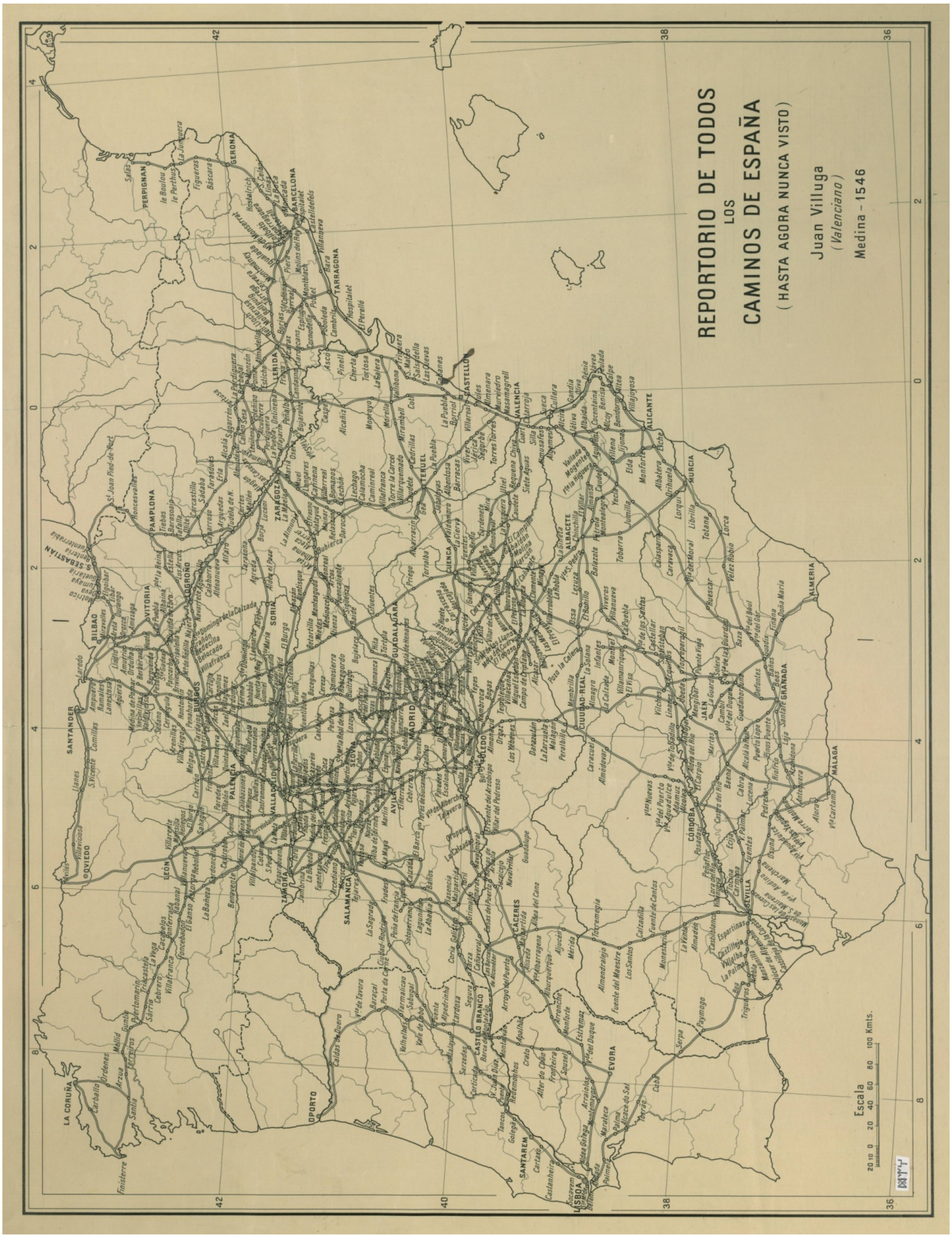
6.1 irudia.. Hispaniako erromatar galtzada nagusiak, Antonine Ibilbidean bilduta. Blázquez (1892)-etik egokitua.

Erromatar Inperioaren jaitsieraren ondoren, erromatar galtzadak bertan behera utzi ziren. Ez zen errehabilitazio edo mantentze-lanik egin eta 10 mendez bide-sare bakarra izan zen Erdi Aroan.

XVI. mendean, austriar dinastiapean bide-sarearen berreraikuntza jarduerak burutu baziren ere, oinarria Erromatar Inperioko galtzadak ziren (Pérez-Acebo, 2018). 6.2 irudiak 1546 urteko errepide-sarea erakusten du.

Borbonen dinastiapean (1714), Felipe V.ak Madrilen ezarri zuen hiriburua, eta errepide-sare modernoa eraiki nahi zuen. Hori gauzatzeko dokumentu bat erabili zen, *Reglamento General para la Dirección y Gobierno de los Oficios de Correo Mayor y Postas de Españas*” (Ariztia, 1720). Dokumentu horretan Madrilen zentratutako errepide-sare bat proposatzen zen. Proposatutako ibilbide nagusiak Madriletik irteten ziren:

1. Baionara (Frantzia), Irun eta Iruñetik
2. Bartzelonara eta Frantziako mugari
3. Valentziara
4. Murtzia eta Cartagenara
5. Sevilla eta Cadizera
6. Badajozera
7. Galiziara Medina del Campotik



6.1 irudia. 1546an Espainian existitzen ziren errepideak (Villuga, 1546).

Sarea Mediterraneo itsasoari paraleloa zen Bartzelonatik Alacantera zihoan errepide batekin osatzen zen, Portugalgo mugari paralelo zen ibilbide batekin Benaventetik Sevillara eta konexio bat Burgosetik Medina del Campora, Valladolidetik pasatuz.

Posta sare hau monarkiak erabiltzen zuen gortetik aginduak bidaltzeko eta informazioa jasotzeko. 1761 urtean, Carlos IIIak finantzatutako azterlan batetik errepide-plan orokorra burutu zen. Bertan, sei errepide erradialak gomendatzen ziren Madriletik, Erreinuko hiriburutik, Coruñara, Badajozera, Cadizera, Alacantera eta Frantziako mugara bai Baionatik bai Perpignanera (Ward, 1779; 1787). Warden proposamena 1761eko ekainaren 10ean lege zehatz baten bitartez gauzatu zen, aurrekontu zentralaren finantzaketa sartuta zeukana Andaluziara, Kataluniara eta Valentziara zihoazen errepideetarako (Carlos III, 1761). Eraikuntza finantzatzeko 20 urterako gatza merkaturatzeko zerga berria ezarri zen, 1801 urtera arte luzatu zena (Bel, 2010). Hala, Wardek planeatutako bide-sarea finantzatu zen, Bizkaiko, Gipuzkoako, Arabako eta Nafarroako probintzietako errepidetakoak izan ezik, haien Foru Ogasun propioek finantzatu zutelarik (Wais San Martin, 1963).

Handik aurrera, Espainiako errepide-sarea garatzeko hainbat plan burutu dira, eta Euskal probintziek errepideei buruzko kudeatzeko autonomia mantendu dute, egoera ezberdinak egon direlarik, Espainiako gobernuaren arabera.

XX. mendean, Francoren diktaduran (1939-1975) Euskal Herriko probintzietako pribilegio ezabatu egin ziren eta gobernu zentralizatuak eskumen guztiak hartu zituen.

Demokraziaren iristearekin, lurralde bakoitzeko errepideei buruzko pribilegioak eta eskumenak berreskuratu egin ziren Autonomia Erkidekoaren Estatutuaren arabera (BOCG del País Vasco, 1980) Oraingo egoera 6.3 atalean azaltzen da.

6.3. Bizkaiko Foru Aldundiaren eskuduntza gaur egun

6.3.1. Bizkaiko Foru Aldundiko errepideei buruzko eskumenak lortzeko prozesua

Euskal Autonomia Erkidegoko Autonomia antolakuntzari buruzko legean, Autonomia Estatutua (BOCG del País Vasco, 1980), 10 artikuluaren arabera, Euskal Autonomia Erkidegoak eskumen esklusibo guztiak mantenduko dira eta, errepideei dagokienez, Espainiako Konstituzioan (Cortes Generales, 1978) 148 artikuluan aipatzen diren eskumenez gain, Euskal Autonomia Erkidegoko lurralde historiko bakoitzaren Foru Aldundiek lehen zeuzkan eskumenak eta estatus juridikoa kontserbatuko dituztela aipatzen da, 3. artikuluaren arabera. 3. artikuluan lurralde historiko bakoitzak bere antolakuntza eta erakunde propioak kontserbatzeko, berriztatu eta eguneratu ahal izango du.

10. artikulua

Euskal Herriko Autonomia Erkidegoak bakarreko eskumena du honako gai hauetan:

3.4. Bide eta errepide arloan, lurralde historikoetako foru-aldundiek oso-osorik gordeko dituzte orain dituzte lege-araubidea eta eskumenak ala, hala dagokionean, Estatutu honen 3. Artikuluaren arabera berreskuratzeko dituztenak, Konstituzioaren 148. Artikuluko 1. Zenbakiko 5. Paragrafoan jasotako eskumenez gain.

3. artikulua

Euskal Herria osatzen duten lurralde historikoetatik bakoitzak ahal izango du, Erkidegoaren baitan, bere antolakuntza eta autogobernu-erakunde propioak gorde edota, hala dagokionean, biziberritu eta eguneratu.

Iturria:Euskal Herriko Autonomia Estatutua (BOCG del País Vasco, 1980).

Espainiako Konstituzioaren 148. artikuluan ezartzen du Erkidego Autonomoek har ditzaketela Erkidego Autonomoaren lurralde barrutian zehar bakarrik igarotzen diren trenbide eta bideak eta era berean trenbidez, bidez eta kablez egiten den garraioari buruzko eskumenak.

Erkidego Autonomoen konpetentziak

148. artikulua

1. Erkidego Autonomoek har ditzazketen konpetentziak alor hontakoak dira:

1.° Autogobernurako Erakunde antolaketa.
2.° Bere lurralde barneko Udalen barruti aldaketa, eta orokorki Estatuaren administrazioari tokiko gorporazioetan dagozkiona, eta tokiko erregimen legeak onartzen duen aldaketa maila.
3.° Lurralde, hirigintza, eta etxegintza eraketa.
4.° Erkidego Autonomoaren lurralde barrutian zehar igarotzen diren eta interesgarri zaizkion Herri Lanak.
5.° Erkidego Autonomoaren lurralde barrutian zehar bakarrik igarotzen diren trenbide eta bideak eta era berean trenbidez, bidez, eta kablez egiten den garraioa.

Iturria: Espainiako Konstituzioa (Cortes Generales, 1978)

Gainera, Azaroaren 25eko 27/1983 LEGEA, Autonomia-Elkarte Osorako Erakundeen eta bertako Kondaira-Lurraldeetako Foruzko Ihadutze-Erakundeen Arteko Harreman"ei buruzko legeak onartzen ditu espresuki lurralde historiko bakoitzeko Foru Aldundiaren eskumen esklusiboak errepideei buruz (BOPV, 1983). Hala eta guztiz ere, hiru lurraldeetako Foru Aldundien eta beste administrazioekin beharrezko koordinazioa egon behar dela ezartzen du.

Bigarren Atalburua.-Kondaira-Lurrandeen Agintepideez

7garren Atala

a) Kondaira-Lurraldeetako Foru-Erakundeek, bakoitzaren berarizko lege-jaurpideen arauera erabiliko dute bakarrean agintepidea izango dute honako gai hauetan:

8. Errepide eta bideekiko egitamuketa, egitasmoak, egitea, hoiei eustea, aldatzea, hoien dirubideak, erabilpena eta ustiaketa.

Autonomia-Elkarteko errepide-sareen arteko behar bezalako egokitasuna bermatzeko, Kondaira-Lurraldeek, bere bide saretan, Euskal Herri Osorako Erakundeek onartutako Orotariko Errepide-Egitamuan jarri daitezen teknikazko eta bide seinaleentzako arauka jarriko dituzte indarrean, eta Erresumako Errepide-sarearen jarraipen izan edo elkarteaz besteko beste Herri-Erakundeenei edo Kondaira-lurraldeetakoei heurei atxiki dakizkienetan berriz, esandako Orotariko Errepide-Egitamu horretan jarri daitezen aintzin-neurriak, helburuak, lehentasunak eta hobekuntzak egingo dituzte gutxienez.

Autonomia-Elkartearen, Erresumaren, Eñkarteaz besteko beste Herri-Erakunde batzuen edo Kondaira Lurraldeen egitamuetako taiuketetan Kondaira-Lurraldeak edo hoien mugakideak har ditzaten elkartzepide berriak egitea oharteman dedinean, egitamu hoien bakoitzaren ahalmen eta agintapideen arauera elkarri egokitiko zaizkio.

Hori guzti hori, Autonomia-Estatutoaren 10.34 Atalean agintzen denaren kaltetan gabe.

Iturria: Azaroaren 25eko 27/1983 LEGEA, Autonomia-Elkarte Osorako Erakundeen eta bertako Kondaira-Lurraldeetako Foruzko Ihadutze-Erakundeen Arteko Harreman"ei buruzkoa (BOPV, 1983) (Idazkera originala errespetatu da).

Horren ondorioz, Euskal Autonomia Erkidegoan, lurralde bakoitzeko, Araba, Bizkaia eta Gipuzkoako, Foru Aldundiek bere lurraldearen barruan dauden autobide, autobia eta errepideei buruzko eskumen esklusiboak dauzkate, nahiz eta errepideak beste Autonomia Erkidegoak zeharkatzen dituen ibilbide luzeko parte bat izan. Salbuespen bakarrak dira bide-sariko bi autobideak, Espainiako Gobernua jabea delarik eta kontzesio baten bidez ustiatzen ari direnak. Autobide horien jabetasunak ezin dira Foru Aldundiei pasatu kontzesioa amaitu arte. Euskal Autonomia Erkidegoko Foru Aldundiek kudeatzen ez dituzten bide-sariko autobideak honako hauek dira.

- AP-68 autobidea, Bilbo (Bizkaiko hiriburua) eta Zaragoza lotzen dituena Logroñotik. Kontzesioa 2026ko azaroaren 10ean amaitzen da (Ministerio de Fomento, 2000). Bizkaiko eta Arabako probintziak zeharkatzen ditu.

- AP-1 autobidea, Burgos eta Arminon (Araba) lotzen dituena. Kontzesioa 2018ko azaroaren 30ean amaitzen da (Ministerio de Fomento, 2005). Arabako lurraldean 5,7 km baino ez ditu.

6.1 eta 6.2 taulak Espainiako Gobernuak Araban eta Bizkaian kudeatzen dituen autobideen kilometroak erakusten ditu. Gipuzkoan ez dago Espainiako Gobernuak dituen errepiderik. Gainera, udalerri barruan dauden errepide lokalak udalenak dira eta udalek kudeatzen dituzte (Pérez-Acebo, 2018).

6.1 taula. Estatuaren jabetasuna daukan errepide-sarea Araban. Iturria: Catálogo provincial de la Red de Carreteras del Estado (Ministerio de Fomento, 2017)

Errepideak	**Hasierako PK**	**Amaierako PK**	**Hasierako puntua**	**Amaierako puntua**	**Luzera**	**Errepide mota**
						Bide-sariko autobidea
AP-1	77+0300	82+1060	P.L. Bizkaia - Araba	Arminongo lotunea	5,74	5,74
AP-68	22+0390	77+0470	P.L. Bizkaia - Araba	P.L. Araba - Burgos	55,15	55,15
Errepide-kopurua :			**2**		**60,89**	**60,89**

Nota: P.L: probintziaren muga

6.2 taula. Estatuaren jabetasuna daukan errepide.sarea Bizkaia. Iturria: Catálogo provincial de la Red de Carreteras del Estado (Ministerio de Fomento, 2017)

Errepideak	**Hasierako PK**	**Amaierako PK**	**Hasierako puntua**	**Amaierako puntua**	**Luzera**	**Errepide mota**
						Bide-sariko autobidea
AP-68	0+0000	22+0390	Bilbo	P.L.. Bizkaia - Araba	22,40	22,40
Errepide-kopurua			**1**		**22,40**	**22,40**

Nota: P.L: probintziaren muga

6.3.2. Bizkaiko Foru Aldundiko errepide eta bideei buruzko eskumenak garatzea

Aurreko atalean azaldu den moduan, Bizkaiko Foru Aldundia Bizkaiko hiri arteko errepide guztien jabea da eta kudeatzen ditu, AP-68 bide-sariko autobidea eta udal-errepide eta kaleak izan ezik. Euskal Autonomia Erkidegoko Autonomia Estatutuan ezartzen diren eskumenen arabera eta lehen komentatutako legearen arabera, Azaroaren 25eko 27/1983 Legea, Autonomia-Elkarte Osorako Erakundeen eta bertako Kondaira-Lurraldeetako Foruzko Ihadutze-Erakundeen Arteko Harremanei buruzkoa (BOPV, 1983), autonomia estatutua garatzen dituena; Bizkaiko Lurralde Historikoko foru erakundeek bakarreko eskumena dute Bizkaiko errepide eta bideen arloko plangintza egiteko, hain proiektuak egiteko, errepide-bideak eraiki, kontserbatu, aldatu, finantzatzeko, zen errepide-bideen erabilera kudeatzeko eta haiek ustiatzeko. Beraz, bere eskumenez baliatuz, Bizkaiko Batzar Nagusiek otsailaren 18ko 2/1993 foru araua onartu zuten Bizkaiko foru errepideen plangintza, proiekzioa, aldaketa, eraikuntza, kontserbazioa, finantziazioa, erabera eta ustiapena arautu zituena, bai eta alboko lurzoruaren erabileraren baldintzak eta mugak ere (BOB, 1993).

Araua onartu zenetik hamabost urte igaro ondoren, komeni zen arauaren alderdi batzuk berrikustea, batetik, haren edukia hobetzeko, eta, bestetik, arau hori gertatutako aldaketetara zein lurraldearen eta herritarren beharretara egokitzeko. Horren ondorioz, Bizkaiko errepideei buruzko martxoaren 24ko 2/2011 Foru-araua onartu zen (BOB, 2011).

Foru arau honek Bizkaiko foru errepideen plangintza, proiektua, aldaketa, eraikuntza, kontserbazioa, finantziazioa, erabilera eta ustiapena nahiz alboko lurzoruaren erabileraren baldintzak eta mugak arautuko dituzten xedapenak ezartzen ditu. Funtzionaltasunaren arabera, hierarkizazio hau ezartzen du errepide-sareen artean:

a) Lehentasunezko sarea (gorria)

b) Oinarrizko sarea (laranja)

c) Sare osagarria (urdina)

d) Eskualde-sarea (berdea)

e) Sare lokala (horia)

Errepide bakoitzak bete behar dituzten ezaugarriak hierarkizazio horretan errepide-sare bakoitzean sailkatzeko, haien funtzionaltasunaren eta eremuaren arabera, 6.3 taulan aurkezten dira.

Bizkaiko Foru Aldundiak kudeatzen duen errepide-sarea 1.200 km baino luzeagoa da, errepideen enborrak baino ez kontuan hartu eta adarrak, konexioak eta lotuneak kontuan hartu barik. 6.3 taulan deskribatutako errepide-sare mota bakoitzaren luzera 6.4 taulan azaltzen da. Hala eta guztiz ere, taula horretan Bizkaian dauden hiri arteko errepide guztiak agertzen dira. Errepide batzuk ez dira Foru Aldundiarenak eta beste batzuk, haien jabea izan arren, ez Foru Aldundiak ez ditu kudeatzen eta mantentzen. Errepide hauek kontzesio moduan daude, AP-8 bide-sariko autobidea bezala, Interbiak erakundeak kudeatua, BFAko sozietate publiko bat. N-629 errepide nazionala, Burgos eta Colinders (Cantabria) lotzen dituena, Foru Aldundiarena da baina Espainiako Sustapen Ministerioak mantentzen du Bizkaiko lurraldearen muturrean dagoelako eta 4 km-ko luzera baino ez duelako. Lehen aipatutako AP-68 bide-sariko autobidea, Espainiako Gobernuaren dena eta kontzesiopean kudeatzen duenaz gain, beste errepide bat dago, BI-711, Bilboko Portua dela bere jabea. Bilboko Portua Espainiako Sustapen Ministerioaren sozietatea da. Informazio guztia 6.5 taulan azaltzen da.

Bizkaiko Foru Aldundiak jabetasuna eta kudeatzen duen errepide-sare osoa 1. eranskinean erakusten da (Pérez-Acebo, 2018). Errepideak errepide-sareen arabera sailkatuta daude, lehen erakutsi direnak, eta errepide bakoitza osatzen dituen tarteak identifikatzen dira. Hasierako eta amaierako Puntu Kilometrikoak (PK) eta tartearen luzera sartuta daude. Gainera, errepide-mota ere aipatzen da. Errepide-motak, *Ley 37/2015, de 29 de septiembre, de carreteras* (BOE, 2015) legearen arabera ezartzen dira, Espainiako Errepideren legea dena. Arauan ezartzen diren errepide-motak honako hauek dira:

- **Autobideak**. Autobide izateko bereziki proiektatu, eraiki eta seinaleztatutako errepideak, ibilgailu automobilentzat bakarrik eginak, errepidearekin mugakide diren jabetzetarako sarbiderik ez duena, ez du beste bide edo trenbide zehartzen eta ez da zeharkatua eta zirkulaziorako noranzko bakoitzak galtzada bat minimoz duena, elkarrengandik bereiziak.

- **Autobideak**, autobideen antzekoak dira baina edozein ibilgailuk erabil ditzake eta ez daukate mugakide diren jabetzetarako sarbide zuzenik baina zerbitzu-bideren bitartez egin daiteke.
- **Errei anitzeko errepideak**, noranzko bakoitzean bi errei baino gehiago dituzten banandutako galtzadak dituzten errepideak dira, beste bideekin maila berean gurutzatu ahal dira eta ez daukate sarbide zuzenik mugakide diren jabetzetara.
- **Errepide konbentzionalak edo arruntak**, beste errepide-moten beharrezko ezaugarriak betetzen ez dituzten errepideak.

6.3 taula. Bizkaiko errepide-sareen hierarkizazio-mailak, haien funtzionaltasunaren eta eremuaren arabera. Iturria: Bizkaiko errepideei buruzko martxoaren 24ko 2/2011 Foru-araua (BOB, 2011)

Metropoli eremua	Gainerako eremua
a) Lehentasunezko sarea	
• Ibilbide luzeko bidaiak bideratzen ditu, bertatik igarotzeko direnak edo metropoli-eremua dutenak sorburu edo jomuga. • Garraio-terminal handietarako sarbideak eskaintzen ditu: portuetara, aireportuetara, garraio-aldaguneetara eta merkantziak garraiatzeko zona logistikoetara	• Ibilbide luzeko bidaiak bideratzen ditu: bertatik igarotzeko direnak, edo sorburu edo jomuga hauetan dutenak, metropoli-eremuan, gainerako eremuan edo, garraio-terminal handietan.
b) Oinarrizko sarea	
• Metropoli-eremuaren zona orbital eta anularren arteko fluxuak bideratzen ditu, errepide bidezko garraio publiko zein pribatuenak • Edukiera handiko ibilbide-ehunaren euskarria da, eta aukera ematen du sarea kudeatzeko	• Eskualdeetatik lehentasunezko interesa duten ardatzetara joaten laguntzen du • Lurraldea egituratzen laguntzen du
c) Sare osagarria	
• Oinarrizko eta lehentasunezko sareen edukiera handiko ibilbideak hiri-arteriekin lotzen ditu • Metropolia sortzen laguntzen du, loturarik gabeko lurzoruak eta aglomerazioak integratzen ditu eta. • Aukerako lurzoruak jartzen ditu eskuragarri • Mugikortasun sortzaile handietarako sarbidea da.	• Ez da existitzen
d) Eskualde sarea	
• Zona orbitalen arteko joan-etorriak errazten ditu, hain urbanizaturik ez dagoen eremu batean. • I Oinarrizko sarearen ordezko bidea da, ezohiko pilaketak arintzen baititu • Udalez gaindiko ekipamenduetarako sarbideak eskaintzen ditu • Edozein lurzoru motatara heltzen laguntzen du • Hiri barruko garapen etengabeak kale bihurtuko ditu sare honetako errepideak.	• Eskualde mugakideak lotzen ditu • Arrantza-portuetarako sarbidea ematen du. • Errepide-ehuna sortzen laguntzen du • Lurzoruetara iristen laguntzen du • Udalerri hurbilen arteko lotura errazten du.
e) Sare lokala	
• Aurreko sareen barruan ez dauden foru errepideek osatzen dute sare lokala	• Aurreko sareen barruan ez dauden foru errepideek osatzen dute sare lokala.

6.4 taula. Bizkaiko Foru Aldundiko errepide-sarearen luzera errepide-sare mota eta errepide-motaren arabera 2015, km-tan (1). Iturria: Errepideen inbentarioaren atala. Bizkaiko Foru Aldundia.

Errepide-sarea	Bide-sariko autobidea	Debaldeko autobideak	Autobiak	Errei anitzeko errepideak	Errepide konbentzionalak	Guztira
Lehentasunezko sarea (Gorria)	68,881	30,009	27,295	13,854	106,794	246,833
Oinarrizkoa (Laranja)			39,437	19,470	151,570	210,477
Osagarria (Urdina)				4,803	24,080	28,883
Eskualdekoa (Berdea)				1,070	208,847	209,917
Lokala (Horia)				1,217	609,685	610,902
TOTAL	**68,881**	**30,009**	**66,732**	**40,414**	**1.100,976**	**1.307,012**

(1) Errepidearen enborra baino ez, adarrak, konexioak eta lotuneak kontuan hartu barik.

6.5 taula. Bizkaiko errepide-sarearen jabetasunaren eta kudeaketaren laburpena. Iturria: Errepideen inbentarioaren atala. Bizkaiko Foru Aldundia

Jabetasuna	Kudeaketa	Errepide-kodea	Errepidearen izena	Puntu Kilometrikoak	Luzera (km)
Bizkaiko Foru Aldundia (BFA)	**Bizkaiko Foru Aldundiak**	***Beste errepide guztiak***	-		**1.223,649**
	Kontzesioa	*AP-8*	*Kantauriko autobidea (Bide-sarikoa)*	*74+0905 – 129+ 504*	*46,531*
	Kontzesioa	*BI-625*	*Urduna Bilbo (Salbe – Ugazko tunela)*	*395+0075 – 396+0115*	*1,009*
	Kontzesioa	*BI-626*	*Artxanda – Salbe tunela (Bide-sarikoa)*	*1+0757 – 3+0718*	*1,961*
	Kontzesioa	*BI-627*	*Artxanda - Ugazko tunela (Bide-sarikoa)*	*2+0109 – 3+0621*	*1,512*
	Besteak (Sustapen Ministerioa)	*N-629*	*Burgos - Colindres (N-629)*	*60+0810 – 64+0675*	*3,870*
				BFAk ez kudeatua guztira	**54,883**
				BFAk kudeatua, guztira	**1,278.532**
Ez da BFA-rena	**Sustapen Ministerioa (Kontzesioa)**	*AP-68*	*Bilbo - Zaragoza bide-sariko autobidea (Ez da BFAkoa)*	*0+0000 – 22+0350*	*22,350*
	Bilboko Portua	*BI-711*	*Bilbo - Getxo (Ez da BFAkoa)*	*7+0200 – 13+0330*	*6,130*
				BFAren jabetasunik ez	**28,480**
			Errepide-sarea guztira (enborra, adarrik gabe)		**1.307,012**

6.4. “Egoera Agenda”, BFAko Bide-zoruak Kudeatzeko Sistema

6.4.1. Sarrera

Bizkaiko Foru Aldundiak errepideen egoerari buruzko datuak biltzen ditu 2000 urtetik adierazle eta indize desberdinen bidez. Kudeatu beharreko errepideekin erlazionatutako informazioa zerrendatu da, honako datu hauek sartuta: errepideen luzera, errepide-mota (errepide konbentzionala, errei anitzeko errepidea, autobia edo autobidea), ezaugarri geometrikoak (errei kopurua, erreien zabalera, kurbetako erradioak, etab.), trafiko bolumenak, egituren inbentarioa, etab. Gainera, BFAk finantzatutako eta kudeatutako proiektu guztiak ere sartu ditu. Hala eta guztiz ere informazio guztia barreiatuta zegoen hainbat tokitan, fisikoki eta datuak datu-tegi informatikoetan sartuta zeudenean ere, artxibo mota ezberdinak zituzten datuak. Beraz, informazioa bilatu behar zen artxibo batzuetan, normalean ez bateragarriak.

Horren ondorioz, mantentze eta birgaitze-jarduerei buruzko erabakiak ingeniarien esperientziaren arabera hartzen ziren, metodo arrazionalik gabe. BFAko ingeniarien esperientziagatik, aukera onak aukeratzen ziren, baina ez ze ezagutzen nola bide-zoruak eboluzionatzen ziren aplikatutako soluzioaren arabera. Labur esanda, baliabideak, bai gisakoak bai ekonomikoak, subjektiboki inbertitzen ziren.

2010 urtetik aurrera, ulertu zen Bide-zoruak Kudeatzeko Sistema bat beharrezkoa zela informazioa kudeatzeko eta metodo arrazionalago bat aplikatu behar zela aukerarik onena hautatzeko, optimizatu ostean. Horren ondorioz, lizitazio bat zabaldu zen eta hiru ingeniaritza-enpresen proposamena aukeratu zen kontratua aurrera eramateko. Enpresa horiek Dair Ingenieros, Euskontrol, S. A. eta Ingeplan Consulting dira.

Handik aurrera, Bizkaiko Foru Aldundiaren Bide-zoruak Kudeatzeko Sistema garatu dute, Egoera Agenda izenekoa eta Bizkaiko Foru Aldundiak emandako informazioarekin eguneratzen jarraitzen dute.

Errepidearen administratzaileak errepidea kontserbatu eta eraberritzerakoan egin beharreko jarduera eta lehenespenen inguruan erabakiak hartu ahal izateko oinarrizko kudeaketa-tresna da Egoera Agenda.

Egoera Agendan jasotzen dira defizitak, kalteak edo nahikotasun funtzionala eta azpiegituraren egoera neurtzen duten parametroekiko egokitzapen-ezak; halaber, zer-nolako hobekuntza diren beharrezkoak, berehalakoak eta kostu baxukoak ere jasotzen ditu, prestazio hobeak lortu eta bideko segurtasuna hobetu ahal izateko.

Egoera Agendak eginkizun hauek beteko ditu:

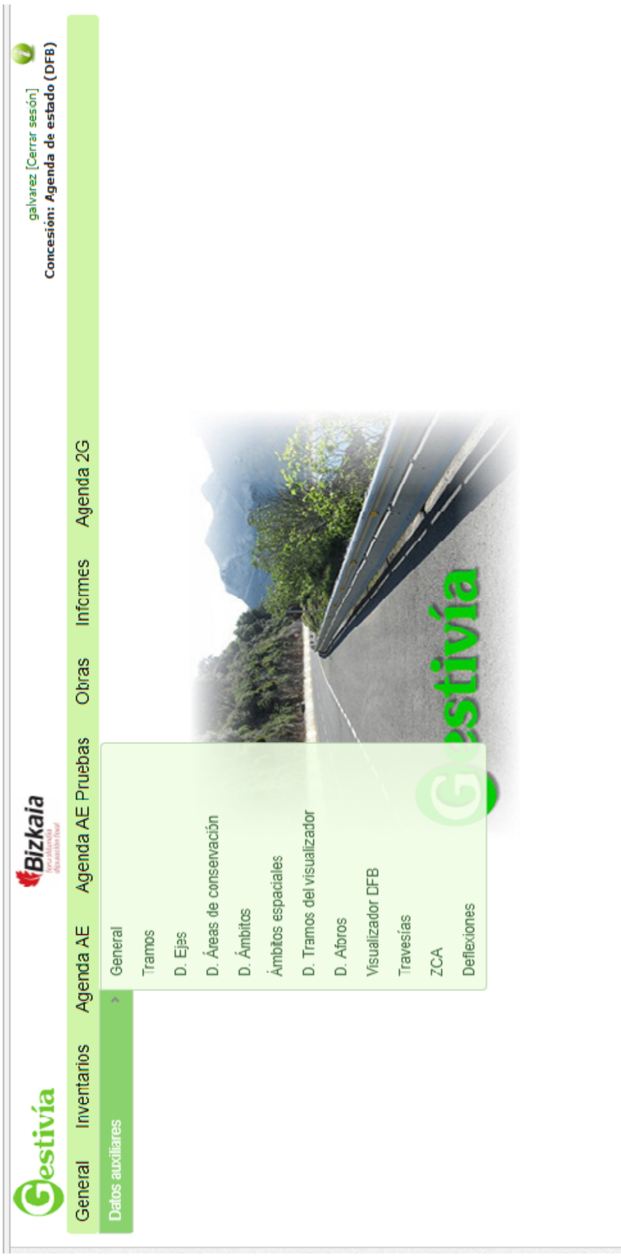
- DIAGNOSIA EGINGO DU: auskultazio edo inbentarioen emaitzak, denbora-estadioak kontuan hartuta.
- ZEHAZTUKO DU: zein aukera dauden, haien eraginkortasuna zein den, eta irtenbide bat proposatuko du.
- EGOERA EBALUATUKO DU: ez haren balio ekonomikoa bakarrik, baizik eta irtenbidearekin lotutako muga funtzionalak eta tenporalak ere bai.
- ADIERAZLEAK PROPOSATUKO DITU: kaltetutako parametroen bilakaera ebaluatzeko.

Administrazioak errepide-sarea indarrean dagoen araudira egokituta edukitzeko, zehazki zer inbertitu beharko lukeen irudikatzen du Agendak, eta, horrez gain, bideko segurtasuna hobetu eta inguruaren araberako funtzionaltasuna egokitzen du.

Inbentarioan sartzen den informazio guztia datu-tegi informatiko batera igotzen dira, datu guztiak biltzen dituena. Software hau Gestivía deitzen da (Bide-kudeaketa) eta BFAn bide-zoruak kudeatzeko lan egiten duten pertsona guztientzat irisgarria da.

Programaren hasierako itxura 6.3 irudian agertzen da. 6 atal nagusi ditu, bistaratu daitezkeenak, eta azpiatalak erakusten dira ere. Atal nagusian dira: Orokorra, Inbentarioak, Egoera Agenda, Egoera Agenda probak (ez da atal bezala hartzen, aurrekoa bezalakoa baita, baina aldaketak egitekotan nolako aldakuntzak sortzen diren ikusteko balio du), Lanak, Txostenak eta Agenda 2G (2. belaunaldiko agenda). Atal nagusi bakoitzaren azpiatalak 6.6 – 6.11 tauletan erakusten dira.

Hurrengo azpi-sekzioan, Egoera Agendan jasotako datuak deskribatzen dira eta konparatzen dira normalean Bide-zoruak Kudeatzeko Sistema batean jaso beharreko datuekin, 2. Kapitulua deskribatzen ziren irizpideen arabera. Beraz, datu eskuragarriak BKS batean egon behar diren datuekin egiaztatzen dira..



6.2. irudia. Gestiviaren itxura, Bizkaiko Foru Aldundiko Bide-zoruak Kudeatzeko Sistemaren softwarea.

6.6.taula Orokorra atalaren azpi-atalak Gestivía software eta bakoitzean sartutako datuak.

1) OROKORRA

Datu laguntzaileak

Orokorra: Errepidearen hasierako eta amaierako PK eta distantziak Bide-sare mota.

Tarteak: Errepidea zatitzen den tarteak.

Ardatzen datuak: Errepide bakoitzean dauden ardatzak: Enbor nagusia (errepide konbentzionala, galtzada bakarrekoa), gorantz edo beherantz doan galtzada (banandutako galtzadetan) eta adarrak (haien PK-ekin).

Kontserbazio-eremuak: Errepidea dagoen kontserbazio-eremua. Kontserbazio-eremuak; 1, 2, 3 eta 4

Eremuak: errepidea zatitzen den azpi-tarteen izenak edo erreferentziak. Normalean, izen popularrak dira, ezagutzen den ezaugarri bat erabiltzen (bide-gurutzea, gasolindegia, etab.).

Eremu berezia : Errepidea zein tartetan sailkatzen da Errepideei buruzko Plan Orokorraren arabera

Ikuslearen tarteak: Tartearen kodeak BFAko web-orriko ikuslean agertzen diren tarteen kodeen arabera

Trafiko bolumenak: Eguneko Batez Besteko Intentsitateak, bai trafiko totalaren bai trafiko astunena errepide bakoitzean 2000tik

BFAko ikuslea: BFAko web-orrian dagoen ikuslean agertzen diren datu geometriko guztiak, 10 m-tan behin**.**

Zeharbidea: Azaltzen da zein errepideren tarteak zeharbide bat bezala sailkatzen dira, bere kodearekin

Istripu ugariko tarteak: Bizkaiko errepide-sarearen Istripu Ugariko Tarteen zerrenda, istripu motak eta leku-mota adieraziz.

Deflexioak: GEOCISA enpresak burututako txosten zahar baten koefiziente batzuk

6.7 taula. Inbentarioak atalaren azpi-atalak Gestivía software eta bakoitzean sartutako datuak

2) INBENTARIOAK

Azpiegitura

Orokorra: Errepidearen zatiketa egituren, lubeten eta abarren arabera

Balizamendua: Hormigoizko New Jersey hesiak, biondak

Bide-zoruak: Zerbitzuan jarri zen urtean. Erreien zabalera, bazterbideen zabalera..

Seinaleztapen bertikala: Errepide bakoitzean daude seinale bertikal guztiak bere kodearekin

Bide-zoruen auskultazioa

Zeharkako marruskadura koefizientea: Errepide bakoitzean, 20 m-tan behin, zeharkako marruskadura koefizientea (SCRIM aparatuaren bidez lortuta) eta bide-zoruaren egitura, datuak bildu diren urteetan

IRI International Roughness Index: Errepide bakoitzean, 100 m-tan behin, IRI balioak erakusten dira, datuak bildu diren urteetan

Deflexioak Deflectografoa: Deflektografoarekin lortutako eskuineko eta ezkerreko deflexio balioak, datuak bildu diren urteetan

Deflexioak Dynatest: Dynatest (FWD) aparatuarekin lortutako deflexioak

Deflexio karakteristikoa: Errepide tarte bakoitzerako kalkulatutako deflexio karakteristikoa

Kalkuluzko deflexioa: Errepide azpi-tarte bakoitzerako lortutako kalkuluzko deflexioa, Ministerio de Fomento (2003a)-ren arabera

Gurpil arrastoak: Lortutako ezkerreko eta eskuineko gurpil arrastoak, datuak bildu ziren urteetan

Pitzadura Indizeak: Pitzadura Indizeak, pitzadura akatsak dituen errepidearen proportzioa bezala, dezimaletan edo portzentajeetan azalduta, urte ezberdinetan

Bide-zoruen inbentarioa

Hasierako bide-zorua: Hasierako bide-zoruaren artxiboa daukaten azpi-tarteak

Proiektuak: Gauzatutako proiektuak eta lanak. Sailkapen mota ezberdinak ezaugarri batzuen arabera**.**

Gainazaleko akatsak: Batxeak, pitzadura eta bide-zoruaren gainazalean ikusitako beste akatsak, bere dimentsioak ezarriz, errepide bakoitzean. Konponketa data ere.

DESU [Gainazaleko akatsak]: Gainazaleko akatsak kudeatzeko sistema garatzeko saiakera. Ez garatua.

Zundaketak: Zundaketaren lekua eta urtea, eta lortutako datu guztiak

Inbentarioa: Hasierako Bide-zorua artxiboa erakusten du (hasierako bide-zorua ezaguna den tarteetan, proiektuan sartuta den bezala), burututako errehabilitazio proiektuak, Bide-zoruaren Egitura artxiboa, eta Gainazaleko geruza artxiboa, errepide bakoitzean

Bide-zoruak Kudeaketa - GEFI: Bide-zoruaren egituraren sailkapena aurreko BKSen saiakeren arabera. Gaur egun ez da erabiltzen

Istripuak

Istripuaren txostena: Istripuari buruzko datu guztiak

Plubiometria: 2000 urtetik 2016ra hilabete bakoitzeko plubiometria

Euskalmet-eko plubiometria: aurrekoaren antzekoa, Euskalmet (Euskadi Autonomia Erkidegoko Meteorologia Agentzia), 2011tik 2016ra eta kontserbazio-eremuaren arabera bananduta

Lan bereziak: errepide bakoitzean gauzatutako lanak

Puntu bereziak: Puntu altuak edo baxuak, bide-gurutzeak, etab.

6.8 taula. Egoera Agenda atalaren azpi-atalak Gestivía software eta bakoitzean sartutako datuak

3) EGOERA AGENDA

Egoera Agenda

Euspen lokaleko elementuen algoritmoa: Dauden euste-elementuak eta proiektu-abiadura

Motor-gidariak euspen elementuen algoritmoa: Motor-gidariak eusteko dauden elementuak

Bide-zoruen algoritmoa: Kalkulatzen du nolako eremua konpondu behar da hainbat aldagairen arabera (SCRIM koefiziente, Pitzadura indizeak, etab.), Konfigurazio atalean eraldatu ahal dena

Bide-zoruen bizitza kalkulatzeko algoritmoa: GEOCISAk proposatutako algoritmoa bide-zoru bakoitzari geratzen zaion bizitza aurreikusteko deflexioen arabera

Gainazaleko akatsen algoritmo Zeharkako Marruskadura koefizienterik gabe. Gainazaleko akatsen larritasuna SCRIM koefizientea kontuan hartu gabe

Hesiak: Errepideen zehar dauden hesien zerrenda

Zeharkako drainatzea: Zeharkako drainatzerako dauden elementuen zerrenda

Luzetarako drainatzea: Luzetarako drainatzerako dauden elementuen zerrenda

Gainazaleko drainatzea: Gainazalerako drainatzerako dauden elementuen zerrenda

Drainatze sakona: Drainatze sakonerako dauden elementuen zerrenda

Uraren laminaren altuera: Uraren altuera areketan

Seinaleztapenaren iraungitze-data: Seinaleztapen bertikalaren iraungitze-data

Dezeleratzeko seinaleztapena]: Dezeleratzeko dauden seinale bertikalak

Aurreratzeko seinaleztapena: Aurreratzeko dauden seinale bertikalak

Altuera egokitzeko seinaleztapena: Altuera egokitzeko dauden seinale bertikalak

Balizamendua: Balizamenduen zerrenda

Balizamendua 2. Itzulia: Balizamenduaren 2. zerrenda

Balizamenduaren egoeraren kontrola: Balizamenduaren egoeraren kontrola

Egiturak]: Dauden egiturak

Ezpondak: Dauden ezpondak

Ezegonkortasun puntuak: Dauden ezegonkortasun puntuak

Abiaduraren analisia: Abiaduraren analisia

Errepideen garrantzia: Errepide bakoitzari emandako garrantzia dagoen errepide-sarearen arabera

Algoritmoak konfiguratzeko*

Bide-zoruak – Zeharkako Marruskadura Koefizientea: Inolako konponketarik, prebentzio-lana eta beharrezko lanak ezarri ahal dira errepideen kategoriaren arabera

Bide-zoruak – sasoiko aldakuntza: SCRIM koefizientean gauzatzen diren zuzenketak neguan hartutako datuak eraldatzeko gainazaleko geruza eta trafiko kategoriaren arabera

Bide-zoruak - prezioak: Errehabilitazio eta mantentze-lan bakoitzerako ezarritako prezioak m2 bakoitzeko

Bide-zoruak - Lehentasunak: Errehabilitazio-lanak haztatzeko koefizienteak ezaugarri batzuen arabera: errepide-sarea, kurbako erradioa, istripu ugariko tartea, hiri-barnekoa, etab.

Bide-zoruak – Akatsen portzentajeak: Akatsen portzentajetako mugak sartu ahal dira errehabilitazio lana beharrezkoa den ezartzeko mugak

Errepideren garrantzia: Trafiko bolumen minimoa eta errepide-kategoria errepidea kontuan hartzeko

Geratzen diren bizitza kalkulatzeko aldagaiak: Deflexio, trafiko bolumen, astunen portzentajearen trafikoa, hazkuntza tasa, erreien arteko banaketa, astunen pisa, ibilgailu mota bakoitzerako ardatz bakoitzeko pisua eta pisu banaketaren baliok

* Konfigurazio-algoritmoa aldatu ahal duten faktore guztiak ez dira komentatzen, bide-zoruekin erlazionatuta daudenak baino ez

6.9 taula. Lanak atalaren azpi-atalak Gestivía software eta bakoitzean sartutako datuak

4) LANAK
Lanak
Gauzatzen ari diren lanak: Momentuan gauzatzen ari diren lanak aukeratu ahal dira irizpide batzuen arabera
Gauzatutako lanak: Amaitutako lanak aukeratu ahal dira irizpide batzuen arabera

6.10 taula. Txostenak atalaren azpi-atalak Gestivía software eta bakoitzean sartutako datuak

5) TXOSTENAK
Batzarren artxibo historikoa: Bide-zoruen Kudeaketa Sistemari buruzko egindako batzarren artxibo historikoa

6.11 taula. Agenda 2G atalaren azpi-atalak Gestivía software eta bakoitzean sartutako datuak

6) 2G AGENDA
Istripu-tasak: BKSren parte berri bat, istripuekin erlazionatuta

6.4.2. Datuen inbentarioa

Bide-sarearen datuen inbentarioak datu kopuru handia dauka, eskualdean daude bide-zoruak identifikatzeko eta deskribatzeko beharrezkoak direnak. BFAko BKSren informazio eskuragarria 2.7.1 azpi-atalean adierazitako elementuak jarraituz komentatuko da.

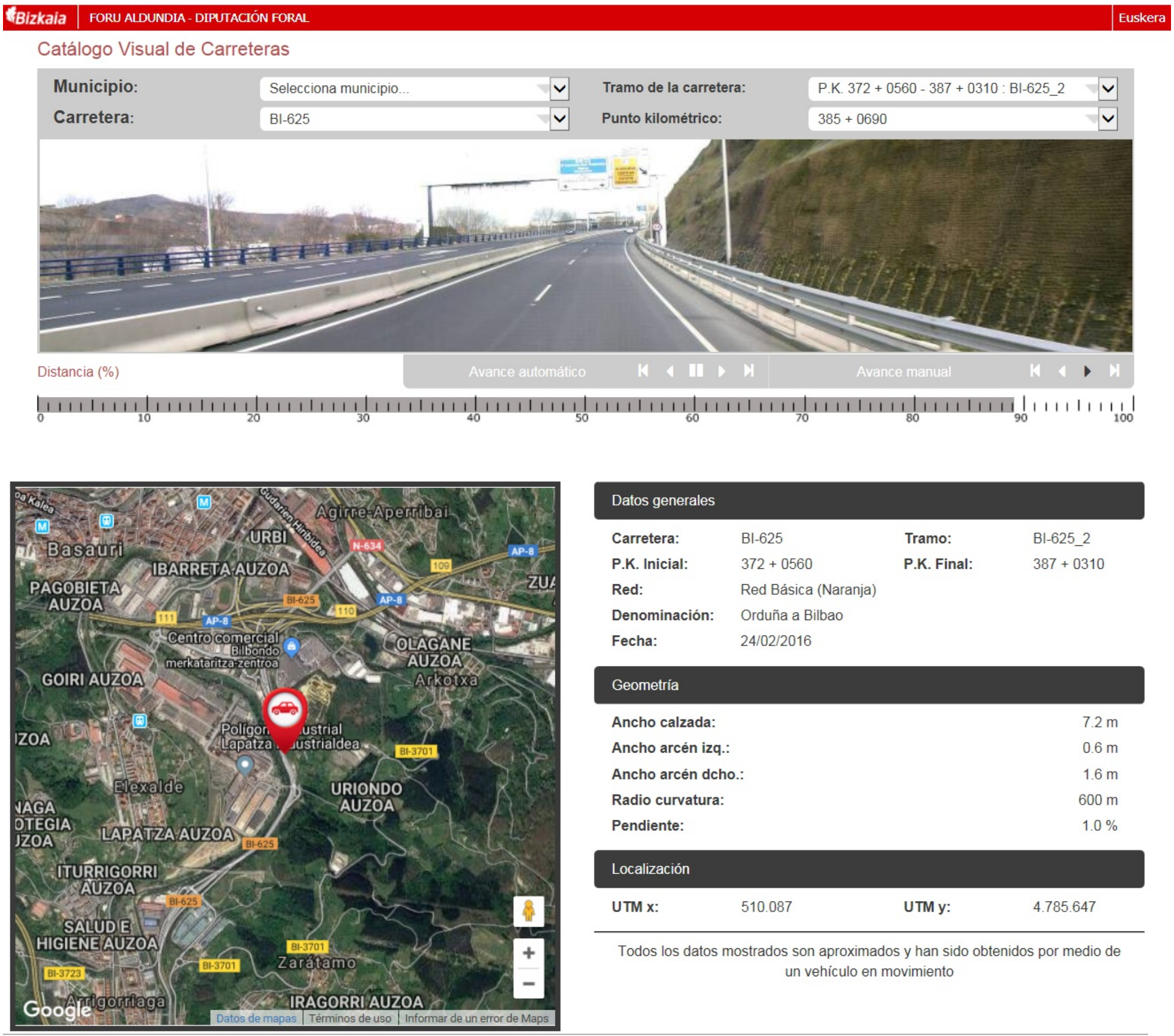
6.4.2.1. Errepidea edo tartearen identifikatzeko datuak

a) **Errepidearen izena eta identifikazioa eta tartearen identifikazioa**. Errepideak identifikatzeko, errepideen izenak (kodeak) erabiltzen dira. Errepide bakoitzak duen kodea da, 1. eranskinean azaltzen direnak, N-634, AP-8, BI-625 bezala. Batzuetan, errepide osoa izen Berea izan arren, luzera osoa ez da Bizkaiko Foru Aldundiak kudeatua. Normala da errepide-tarte batek herri bat zeharkatzen duenean, tarte hori udalari transferitzen da. Beraz, luzera hori udal administrazioak kudeatzen du eta ez BFAk. Beste batzuetan, errepideak beste probintziak zeharkatzen ditu, eta horren ondorioz, tarte horiek ez daude Bizkaiko lurraldean. Honelako egoerak agertzen direnean, izendatze berea daukan errepidea tarte batzuetan banatzen da, eta tarte horiek elkarren segidako kodifikazio batekin izendatzen dira, hau da, BI-624-1, BI-625-2, etab. Gainera, udalei edo beste administrazioei transferitutako tarteei kodeak ere jartzen zaizkie, adibidez, BI-625-C1. Horrez gain, softwarek beste kodeak ere erabiltzen ditu, baina guztiek erreferentzia egiten diote daude tarte espezifiko bati.

b) **Hasierako eta amaierako puntuak, tartearen luzera eta kokapen erreferentzia sistema**. Tarte bakoitzaren hasierako eta amaierako puntuak sartuta daude. BFAk garatutako kokapen erreferentzia sistema **Poste Kilometrikoaren** metodoa (edo **Puntu Kilometrikoa, PK**) da. Poste kilometrikoak kokatuta daude errepidean zehar, normalean beraien artean 1 km gutxi gorabehera dagoelarik. Metodorik onena dela uste da trazatu berriak eraikitzekotan tarte berriak eragiten duen luzeraren poste kilometrikoak baino ez direlako aldatu behar eta errepide osoan ez delako aldaketarik egin behar. Beraz, eraldatuta izan ez diren tarteak mantentzen dituzte haien erreferentziak eta ez da haiekin erlazionatutako datuak eraldatu behar. Poste kilometrikoaren erreferentziak 0+0000 bezala ezartzen dira. Lehenengo zenbakiak (+ ikurra baino lehen) kilometroa adierazten du eta hurrengo 4 zifrek aurretik aipatutako Poste Kilometrikotik dagoen distantzia, metrotan. Lau zifra behar dira elkarren segidako bi Poste kilometrikoen arteko distantzia 1.000 m baino luzeagoa izan daiteke. Beste batzuetan, elkarren segidako bi PK-ren arteko distantzia 1.000 m baino laburragoa da. Tartearen luzera totala ere adierazten da, distantzia partzialen batuketa egin ondoren.

c) **Koordenatu geografikoak**. Errepide-tarte bakoitzeko hasierako eta amaierako puntuen koordenatu geografikoak ere eskuragarri daude. Konkretuki, UTM koordenatuak erabiltzen dira. Gainera, UTM koordenatuak ere eskuragarri daude tarte bakoitzean 10 m-tan behin. Informazio hau ere eskuragarri dago BFAko web-orrialdean (www.bizkaia.eus), non BFAk kudeatzen dituen errepide guztien zerrenda ikus daitekeela (6.4 irudia). Ikusten den bezala, errepidearen kodea, izendatze eta PK-ak ere eskuragarri daude.

d) **Errepide-sailkapena, funtzioa, administrazio-eremua eta jabetasuna**. BFAk kudeatzen dituen errepideak 6.3 taulan adierazitako irizpideen arabera sailkatzen dira. Errepide bakoitzeko funtzioa errepide-sarearen sailkapenean sartzen da. Errepide-sare osoaren kudeaketa 4 kontserbazio-eremuetan burutzen da. 1. eremuak Uribe-Kosta eta Gernikaldea hartzen ditu. 2. eremuak Lea-Artibai eta Durangaldea hartzen ditu. 3. eremuak Nerbioi-Arratia eta Enkarterriak hartzen ditu. Azkenik, 4. eremuak Bilboko metropoli-eremua hartzen du. Tarte bakoitzerako, dagokion kontserbazio-eremua (1., 2., 3., edo 4. eremua), bere kontserbazioa gauzatu behar duen bide-administrazioa (Interbiak, Sustapen Ministerioa, Bilboko Portua edo udal administrazioak) ezartzen da, 6.5 taulan erakusten den bezala

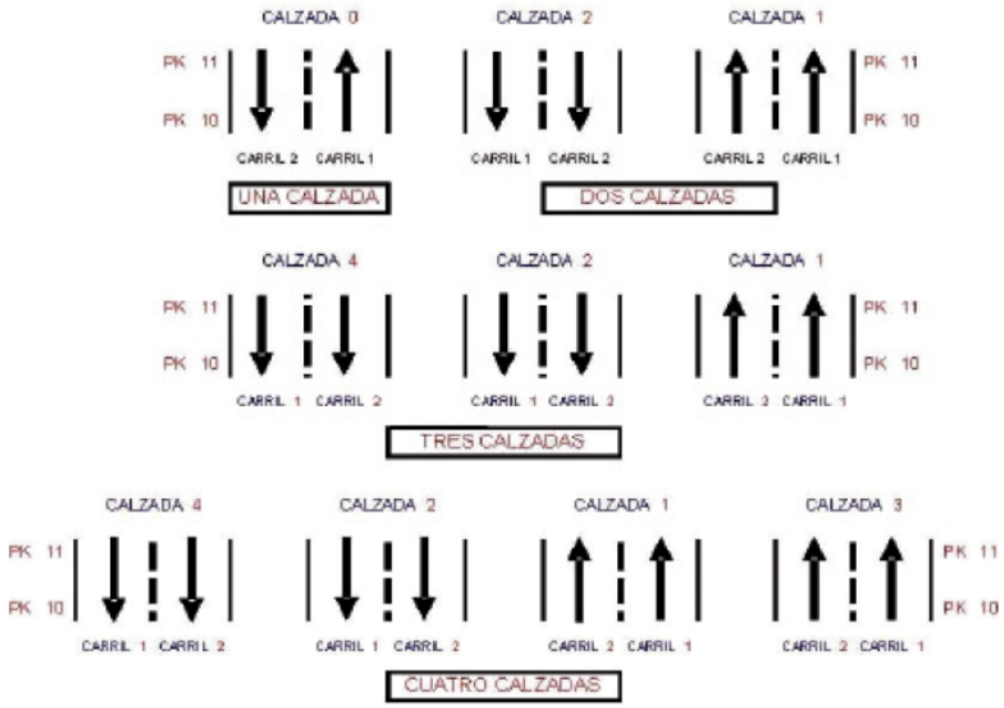


6.3 irudia. BFAko web-orrian errepideren ikuslearen adibidea. Iturria: www.bizkaia.eus

6.4.2.2. Galtzadaren datu geometrikoak

Errepidearen datu geometrikoak ere sartzen dira BFAko Bide-zoruak Kudeatzeko Sisteman. Errepide-tartea galtzada bakarrekoa edo galtzada bananduetakoa den ezartzeko “*Guía para la actualización del inventario de firmes de la Red de Carreteras del Estado*” gida (Ministerio de Fomento, 2011a) jarraitzen da.

Errepideak galtzada bakarrekoa bada, galtzadari 0 ezartzen zaio. Bi galtzada daudenean, PK-ak kontuan hartuta gorantz doan errepidearen eskuineko galtzadari 1 ematen zaio eta ezkerraldean dagoenari 2a. Bi galtzada baino gehiago daudenean, 1 eta 2 zenbakiak galtzada nagusientzat erreserbatzen dira. Galtzada guztiek garrantzia berdina baldin badaukate, 1 eta 2 zenbakiak erdian daudenei aplikatzen zaizkie, 2 banandutako galtzadentzat ezarritako irizpidea jarraituz. Gorantz doazen galtzadaren noranzko berdineko galtzadek zenbaki bakoitiak hartzen dituzte (1, 3, 5, etab.). Beste noranzkoan dauden galtzadek zenbaki bikoitiak erabiltzen dituzte (2, 4, 6, etab.) Azaldutako kasuak 6.5 irudian laburtzen dira.



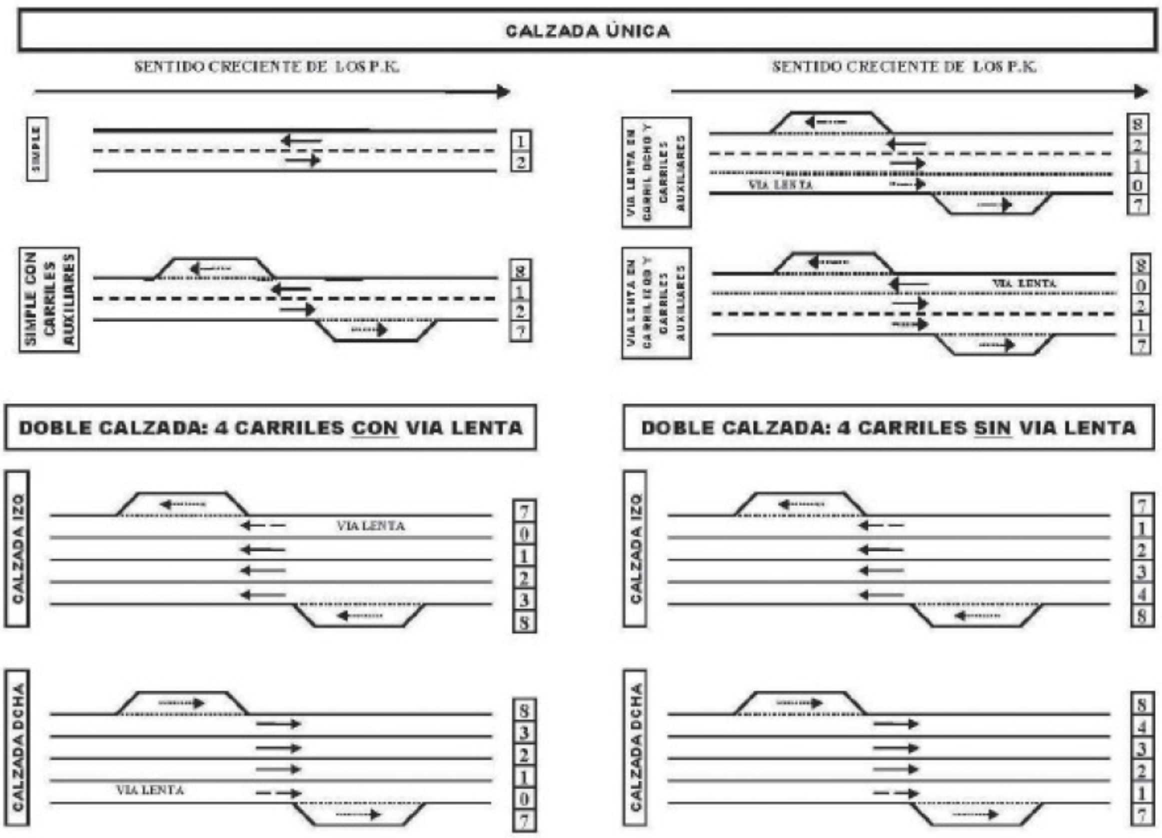
6.4 irudia. Galtzadak izendatzeko arauak (Ministerio de Fomento, 2011a).

Galtzada bakoitzean erreiak identifikatzeko, honako irizpide hauek erabiltzen dira.

- **Galtzada bakarreko errepide konbentzional batean**, PK-ak kontuan hartuta gorantz doan eskuineko erreiak 1 zenbakia hartzen du eta ezkerraldeko partean dagoenak 2a. Errei laguntzaileak egotekotan, 7 zenbakia erabiltzen da eskuineko erreiarekin bat egiten duen errei laguntzailerako, eta 8a erabiltzen da ezkerraldean dagoenarentzat. Ibilgailu astunentzat erreiak badaude, 0 zenbakia hartzen dute.

- **Galtzada banandutako errepideetan**, erreiak elkarren segidako zenbakiekin izendatzen dira, kanpoko erreitik hasita barneko erreirantz (0, 1, 2, 3, 4, etab.). 0 zenbakiarekin hasten da kanpoko erreia ibilgailu astunentzat erreserbatutako erreia bada, edo 1 zenbakiarekin errei konbentzionala bada. Errei laguntzaileak egotekotan 7 zenbakia ematen zaio kanpoko errei laguntzaileari eta 8 zenbakia barneko erreiari. 6.6 irudiak erakusten du azaldutako egoetarako eskemak.

Erreiaren zabalera, galtzada osoaren zabalera eta bazterbidearen zabalera ere sartzen dira BKSn. Datuak errepidearen 10 m-tan behin eskuragarri daude, 6.4 irudian ikusi den moduan. Kurbaduraren erradioak eta luzetarako sestraren inklinazioa ere eskuragarri daude BFA-ko web-orrian (6.4 irudia). Datu geometriko gehiago ere badaude, hala nola ikuspen-distantziak eta alderantzizko ikuspenak.



6.5 irudia. Erreiak izendatzeko eskema hainbat egoeratan (Ministerio de Fomento, 2011a).

Azkenik, errepide-tartearen natura ere sartu da. Tartea egitura baten gainean, tunel batean edo herri zeharbide batean dagoen ere aipatzen da. Tarte hauek aipatzeko 200 m baino luzeagoak izan behar dute. Tarteak izan ahal dituen natura posiblearen adibideak honako hauek dira: tunela, hiri-barneko eremua, saihesbidea, hiri-arteko tartea, portu-eremua, industria-gunea edo eremu esperimentala. Beste datuak ere sar daitezke..

6.4.2.3. Bide-zoruaren egitura, mantentze- eta errehabilitazio-lanak eta proiektuak.

Sustapen Ministerioaren aipatutako gidak, *"Guía para la actualización del inventario de firmes de la Red de Carreteras del Estado" izenekoa* (Ministerio de Fomento, 2011a), bide-zoruaren sekzioak datu-tegian sartzeko metodologia bat proposatzen dut. Metodologiak bi motako artxiboak erabiltzen ditu: hasierako bide-sekzioaren artxiboa eta mantentze- eta errehabilitazio-artxiboa. Lehenengoak bide-zoruaren egitura osoa sartzen du, dagoen bide-zorua Espainiako Bide-zoruak neurtzeko gidetan agertzen direnetako bat den aipatuz [*Norma 6.1. IC "Firmes flexible" and Norma 6.2. IC "Firmes rígidos"* (Ministerio de Obras Públicas, 1976a); *Secciones de firme en autovías* (Ministerio de Obras Públicas y Urbanismo, 1986b); *Instrucción 6.1 y 6.2 Secciones de Firme* (Ministerio de Obras Públicas y Urbanismo, 1989a); *6.1-IC Secciones de Firme* (Ministerio de Fomento, 2003b)]. Horrekin batera, geruza bakoitzean erabilitako material guztiak eta geruzen lodierak ere sartzen dira. Era berean, Mantentze- eta Errehabilitazio-artxiboak dagoen bide-zoruan gauzatutako errehabilitazio eta mantentze-lanak sartzen ditu. Beraz, metodologia horrek tartearen zeharkako azken sekzioa erakusten ditu, aurrekoak eta hasierakoa.

Bizkaiko Foru Aldundiko Bide-zoruak Kudeatzeko Sistemak aipatutako gidaren ideia batzuk erabiltzen ditu.

Hala eta guztiz ere, errepidearen tarte bakoitzean dagoen bide-zoru egitura eta jasandako mantentze-historia sartu ordez, datu hauek errepidean gauzatutako proiektu eta lanen bitartez sartzen dira datu-tegian. Erabilitako metodologia oinarrizkoa da bide-zoruen hondatze-modeloak garatzeko, gehiago azaltzen da 6.5 atalean.

6.4.2.4. Egiturari buruzko beste datuak

Bide-korapiloak, zubiak eta drainatzeko egiturak ere aipatzen dira softwarean. Errepide bakoitzean dauden ezaugarri guztiak zerrendatzen dituen sekzio bereziak daude. Hondatze-modeloak garatzeko beharrezkoa ez denez informazio hau, ez da gehiago komentatzen.

6.4.3. Trafikoaren datuak

Bizkaiko Foru Aldundiak errepideei buruzko eskumenak transferitu zizkiotenetik trafiko datuak bildu ditu. Gainera, BFAk urtero argitaratu du errepide-sarean neurtutako trafiko bolumenei buruzko txosten bat, “Trafikoaren bilakaera Bizkaiko errepideetan” izenekoa, non dagokion urtea aipatzen delarik (Pérez-Acebo, 2018). Dokumentu garrantzitsua eta hartutako datu guztiak erakusten ditu, komentatzen dira eta irudiak ere agertzen dira datuak hobeto erakusteko.

2016ko Edukiera Planak kontrolatu zituen lurraldeko errepide-sareen tarten guztien luzera 1.309,2 km-koa izan zen, guztira. Horietatik 1.304,1 km 2016ko abenduaren 31n katalogatu zen Sare osoari dagozkio ia-ia (%99,6), eta gainerakoak (56,2 km) honako honi dagozkio: sailkatu gabeko forupeko tarteak (lotura-adarrak), udalei emandako tarteak eta Arabako Lurralde Historikoari dagokion A-625 errepidearen tarte bat, 6.12 taulan erakusten den moduan

6.12 taula. Errepidearen luzera sare motaren arabera banatuta. Iturria: Trafikoaren bilakaera Bizkaiko errepideetan 2016 (Diputación Foral de Bizkaia, 2017).

Sare mota	Sarearen luzera (km)			Neurketa gune kopurua.
	Kontrolatuta	Katalogatuta	%	
Lehentasunezko interesa (gorria)	248,6 *	245,7	100,0	99
Oinarrizkoa (laranja)	210,4	210,4	100,0	91
Osagarria (urdina)	32,5 **	28,9	100,0	22
Eskualdekoa (berdea)	209,5	209,5	100,0	75
Lokala (horia)	603,1	314,7	98,1	183
Sare katalogatu osoa	**1.304,1**	**1309,2**	**99,6**	**470**
Sailkatu gabeko sarea (S/G)	5,6			16
Emandako tarteak	50,6			30
Sare ez-katalogatu osoa	**56,2**			**46**
Sare kontrolatuta osotara	**1360,3**			**516**

* Kantauriko autobidearen tarte bat sartu da,. Jada ez agertzen katalogoan, baina neurtu da.

** Jada lagatako BI-737 errepidearen zati bat sartu da, korridoreko trafikoa ez hutsaltzeko.

BFAk 516 estazio erabili zian ditu 2016an. Estazio finko batzuk finkoak dira. 1990. urtera arte, horrelako hiru estazio baino ez zeuden Bizkaian. Beraz, bigarren mailako estazioen eta estazio osagarrien EBBI, gehienetan, horiei zegozkien antzeko lehen mailako estazioen neurketan oinarritzen zen (Pérez-Acebo, 2018).

1991. Urtean, BFAk 97 estazio indukzio magnetikozko begiztez hornitu zituen, eta, horri esker, beharrezkoa izanez gero, estazio horiek estazio finko bihur zitezkeen. Ordutik gaur arte, begiztak leku gehiagotan jartzen joan dira eta, egun, horrelako 124 estazio daude, metropoli-eraztunean daude kontrol telematikoko sareko 22 estazioez gain. 2016. Urtean, dauden 516 estazioetatik, 18 estazio finkoak dira, 41 lehen mailakoak, 139 bigarren mailakoak, 265 osagarriak, 29 ernakida erregimenekoak eta 24 telematikoak (Pérez-Acebo, 2018).

2000. urtetik honako trafiko bolumenak sartuta daude datu-tegian. Aurreko urteetako datuak paperezko dokumentuetan daude, baina ez dira sartu softwarean. Estazio bakoitzerako, errepidearen tarten zehatz bata ordezkatzen duena, honako datu hauek eskuragarri daude 2000. urtetik orain arte. Eguneko Batez Besteko Intentsitatea (AADT) trafiko osoarena bi noranzkoetan, astunen portzentajea, ibilgailu astunen Eguneko Batez Besteko Intentsitatea (H.AADT) proiektuko erreian eta trafiko kategoria (6.13 taula). Lehen adierazi den bezala, Espainiako legeen arabera (Ministerio de Fomento, 2003b), ibilgailu astuna kontsideratzen da 3.500 kg baino gehiago pisatzen duenean. Hau da trafiko astunei buruzko informazio bakarra. Ez dago daturik Ardatz Bakarreko Pisu Baliokidea (*Equivalent Single Axle Load*) (*ESAL*).

6.13 taula. BFAko BKSn trafikoari buruz sartutako informazioaren adibidea.

Estazioa	**Errepidea**	**Galtzada**	**Hasierako PK**	**Amaierako PK**	**Tartearen luzera (m)**	**Errei mota**	**AADT 2000**	**Astunen portzentajea 2000**	**H.AADT proiektuko erreian, 2000**	**Trafiko kategoria**
115A	BI-625	BI-625Galtzada 0	382+0200	384+0050	2030	2 errei	18.892	18.4	1.739	T1
154A	BI-625	BI-625 Galtzada 0	384+0050	386+0050	850	2 errei	25.491	18	2.295	T0
153A	BI-625	BI-625 Gorantz(Galtzada 1)	386+0050	387+0310	1270	2 errei noranzko bakoitzean	29.062	19	2.761	T0

6.4.4. Ingurumen-datuak

Gestivían, Bizkaiko Foru Aldundiko Bide-zoruak Kudeatzeko Sisteman ez dira sartzen ingurumen-daturik. Bizkaia probintzia txikia da, 2.217 km^2 baino ez ditu eta klima ozeaniko homogeneoa dauka probintzia osoan, prezipitazio handiak (1.200 mm/urte) urte osoan zehar eta tenperatura moderatuak. Tenperatura baxuagoak izan daitezke barneko kota altuetan, elurra ohikoa izaten baita neguan.Bilboko batez besteko tenperatura maximoa urtarrileko 13° C eta abuztuko 26° C-ren artean dago.

Horren ondorioz, esan daiteke ingurumen-agenteak uniformeki eragiten dute probintzia osoan eta ez dago klima-diferentziarik errepide-sare osoan. Beraz, ez dago klima-zatiketarik, AASHTO (2012) gidak proposatzen duen bezala, Bizkaiko neurri txikiagatik, EEBBetako estatuekin konparatzean. 6.7 taulan azaltzen den moduan, prezipitazio datuak sartuta daude Gestivían. Badaude hileroko prezipitazioa 2000tik orain arte Espainiako Meteorologia Agentziaren arabera eta hileroko prezipitazioa 2011tik orain arte Euskalmet-en arabera (Euskadiko Meteorologia Agentzia), kontserbazio-eremuaren arabera zatikatuta (6.4.2.1 d). Datu hauek istripuei buruzko atalean sartuta daude..

Gainera, beste ingurumen-datu bat sartuta dago datu-tegian. 5.4.5.1 atalean komentatu zen moduan, euri zaparradek eragina daukate labainketarekiko erresistentziaren datuetan. Beraz, marruskadura datuak bildu baino lehen botatako euri zaparradak kontuan hartzeko 2016ko datu-biltzean aurreko 15 egunetan jasotako prezipitazioen datua estazio meteorologiko hurbilenean ere sartzen da sisteman.

6.5. Gestivían bide-zoruen egiturak sartzeko metodologia

6.5.1. Proiektuak sartzea

Adierazi den bezala, bide-zoruen sekzioei buruzko informazioa sartzen da Bizkaiko errepide-sarean burututako proiektuen bitartez. Metodologia honek datu errealak sartzen ahalbidetzen du, nahiz eta osorik ez izan. Informazioa osorik ez izan arrean, informazio guztia benetakoa da eta egiaztatu da. Proiektuen bidezko informazioa dela esan daiteke, eta ez da beharrezkoa errepide tarte guztiak aztertu, garestiagoa izango litzakeena.

1983. urtetik aurrera gauzatutako proiektuen gehiengoa, Bizkaiko Foru Aldundiak errepideei buruzko eskumena izan zuenetik, sartuta daude datu-tegian. Aurreko informazioa ere euskaragarri daude, Espainiako Gobernuak Bizkaiko errepideak kudeatzen zituenean, baina informazio hori paper formatuan gordeta zegoen eta 1983ko Bilboko uholdeetan galdu zen.

1983tik aurrera, proiektu garrantzitsuenak eskuragarri daude, batez ere trazatu berriak deskribatzen dituztenak. 1990etik aurrera, eraikuntza proiektu guztiak datu-tegian sartuta daudela esan daiteke. 2000. urtetik aurrera, errehabilitazio eta mantentze-lan guztiak ere sartuta daude Gestivían. 2000. urtea baino lehen burututako errehabilitazio- eta mantentze-lan guztiak ez dira sartuta. Beraz, 80ko hamarkadetan eraikitako tarteetan inolako errehabilitazio- eta mantentze-lanik ez dela egin ikusten bada, kontuz hartu behar dira datu horiek informazio-falta dagoela eta. Hala eta guztiz ere, ez da normala eta bide-zoruan egindako lan guztiak erregistratuta daude tarte gehienetan.

Bai errepide-tarte berri baten eraikuntza proiektuetarako bai errehabilitazio- eta mantentze-proiektuetarako, honako informazio hau sartzen da softwarean:

- **Kodea**. Proiektu bakoitzari kode bat ematen zaio, 1etik hasita (PROJ-1, PROJ-2, etab.). Dagoen ordenak Bizkaiko errepideak aztertzen hasi zirenean emandako ordena jarraitzen du. Normalean, errepide baten proiektu guztiak batera bilduta daude, elkarren segidako zenbakiekin. Proiektu guztiak sartu zeudenean, proiektu berriek hurrengo zenbakiak hartzen dituzte. 2017an, 2000 proiektu baino gehiago erregistratuta daude, mantentze- eta errehabilitazio-jarduerak barne.
- **Errepidea**. Honako informazio hau sartzen da:
    - **Errepidearen izendatzea**. Proiektua gauzatzen den errepide nagusiaren izendatzea.
    - **Galtzada**. Proiektua burutzen den galtzada: errepide konbentzionaletan dagoen bakarra, edo galtzada bananduetako errepideetan, gorantz edo beherantz doana PK-ak kontuan harturik, edo lotune-adarrak.
    - **Hasierako eta amaierako PK eta luzera**. Errepide nagusiaren hasierako eta amaierako PK-ak eta puntu horien koordenatuak. Hasierako eta amaierako puntuen arteko benetako

distantzia ere sartzen da.

- **Proiektuaren datuak**. Honako datu hauek erregistratzen dira:
  - **Proiektuaren titulua**. Proiektuaren izen osoa, tituluan adierazita dagoen bezala
  - **Egilea**. Proiektua idatzi duen egilea. Normalean ingeniaritza enpresa bat da. Proiektu txikietan, bide-administrazio bera egilea izan daiteke.
  - **Proiektu-mota**. Proiektua bide-zoruaren egituran aldaketa bat suposatzen duen adierazten da. Alde batetik, *Proiektua/Errodadura lana* bezala sailkatu ahal da, bide-zoruaren egitura eraldatua izan dela adieraziz. Beste aldetik, Batxeak bezala sailkatzen bada, konpontze puntualak gauzatu direla esan nahi du, bide-zoruaren egituran eraginik gabe. Batxe bezalako lana sartzen bada, informazio gehiago sartzen da: data, lan-mota eta galtzada osoa edo errei batzuetan baino egiten den.
  - **Iturria.** Proiektua non aurkitu zen adierazten da. Komentatu den bezala BKS martxan jarri baino lehen, proiektuak hainbat tokitan zeuden sail ezberdinek gauzatu zutelako. Atal honetan zein sailetan edo lekutan (ez-fisikoa ere) aurkitu zen proiektua aipatzen da. Sartutako sailak hauek izan ahal dira: BFAko artxiboa, Kontserbazio Saila, inbentarioaren atala, Interbiak (proiektu eta errepide batzuk kudeatzen dituen enpresa publikoa), Bide-zoruen Plana (1990. urtean gauzatutako plan garrantzitsu bat, non hainbat jarduera burutu ziren errepide-sare osoan), BFAko web-orrian (www.bizkaia.eus), EAO (Estatuko Aldizkari Ofiziala) edo EHAA (Euskal Herriko Agintaritzaren Aldizkaria). Azken kasu honetan, proiektu batzuk ez daude gordeta baina EAO-n ta EHAA-n informazioa ere aurkitu daiteke eta beraz datu batzuk sartu ahal dira
  - **Saila**. Proiektuaren sail arduraduna adierazten du. Sail batzuk hauek izan ahal dira: Eraikuntza saila, Kontserbazio Saila, Kudeaketa saila, Eguneratze saila eta beste batzuk.
  - **Kontserbazio-eremua**. Zein kontserbazio-eremutan kokatzen den proiektua adierazten du. Kontserbazio-eremua 1, 2, 3 edo 4, kontzesiopeko tartea edo transferitutako tartea izan daiteke, 6.4.2.1 atalean komentatu den bezala.
  - **Proiektu-mota**. Zein proiektu-mota den adierazten du, datu-tegian sartzeko. Hauek dira aukerak: Planifikazio proiektua (Plan Orokor batekin erlazionatutako proiektua), Eraikuntza proiektua (detailean deskribatzen den proiektua), proiektu osagarria (proiektu bat gauzatzen den bitartean idazten den proiektua jarduera gehiago sartzeko), eraldatutako proiektua (lanak dauden bitartean idazten den proiektua aldatzen diren gauzak idatzita izateko) eta amaierako proiektua (lanak amaitzen direnean, burutzen den proiektua, egindako guztia dagoen bezala deskribatzeko). Sar daitezkeen beste motak: aurre-proiektua, trazatu-azterlana, azterlan informatiboa, trazatu-proiektua, behin-behineko amaierako proiektua, etab.
  - **Eraikuntza-mota**. Nolako proiektua gauzatzen den adierazten du bide-zoruaren sekzioaren

aldetik. Honako aukera hauek daude: *Trazatu berria* (Trazatu berria gauzatzen denean, eta bide-zoru sekzio berria egiten da), *bide-zoruaren hobekuntza* (geruza berri batzuk gehitzen zaizkionean dagoen sekzioari), *bide-zoruaren hobekuntza eta trazatu-aldaketa* (trazatua partzialki eraldatzen denean eta geruza berri batzuk zabaltzen direnean). Beste batzuk ez dira hain ohikoak: *3. erreiko proiektua, bide-segurtasunaren proiektua, kalteak osteko konponketa* (uholdeak), etab.

  - **Espediente zenbakia**. BFAko aurreko datu-tegietan proiektuak zeukan zenbakia adierazten du.
  - **Proiektuaren data**. Egileak, normalean ingeniaritza enpresa batek, proiektua idazten noiz amaitu zuen adierazten du.
  - **Zerbitzuan jarri zen data**. Lanak noiz amaitu ziren adierazten du eta beraz, proiektua zerbitzuan jarri zen eguna. Data zehatza ez badago eskuragarri, proiektuaren data baino 2 urte geroago jartzen da. Datu garrantzitsua da.
  - **Burututako proiektua**. Proiektua benetan gauzatu zen ala ez adierazi behar du. Batzuetan, proiektuak idatzita egon arren, azkenean ez dira burutzen, eta beraz, ez dute eraldatzen bide-zoruaren sekzioa.
  - **Fikziozko proiektua**. Proiektua fikziozkoa den adierazi behar du. Proiektu gehienak ez dira fikziozkoak, baina batzuetan, ez dago errepide bati buruzko informaziorik baina errepidea existitzen da eta errodadura geruza ezaguna da. Orduan, kasu hauetan, fikziozko proiektua bezala sartzen da Gestivían, bertan dagoen errodadura kontuan hartzeko, nahiz eta informazio gehiago eskuragarri ez egon (adina, errehabilitazio-historia, etab.)

- **Bide-zoruaren informazioa**. Bide-zoruaren egiturari buruzko honako informazio hau sartzen da:
  - **Bide-zoruaren kodea**. Proiektuaren barruan, hainbat lan eta bide-zoru gauzatu ahal dira errepide berean edo beste errepideetan. Horren ondorioz, lan bakoitzak kode indibidualarekin erregistratzen da. Proiektuaren kodea erabiltzen du eta gidoi bat eta zifra batzuk. Adibidez: PROJ-30-1, PROJ-321-2020.
  - **Errepidearen izendatzea**. Proiektuak eragina duen errepide guztiak aipatzen ditu. Nahiz eta errepide nagusia adierazita dago proiektu-datuetan, proiektu batek normalean beste aldaketak eragin ditzake beste errepideetan, haien arteko loturak baitaude. Beraz, proiektuak eragindako errepide guztiak aipatzen dira
  - **Errepide-datua**. Proiektuak eragina daukan errepide bakoitzean datu batzuk adierazten dira: Lanak gauzatzen diren galtzadak (errepide konbentzionaleko errepide bakarra edo gorantz edo beherantz doan galtzada, galtzada banandutako errepideetan), tartearen hasierako eta amaierako PK-ak, hasierako eta amaierako PK-en UTM koordenatuak (X eta Y) eta tartearen luzera. Gainera, adierazten da galtzadaren zein erreitan proiektuak eragina daukan. Azkenik, tartearen natura ere adierazten da: egitura baten gainean, hiri-barnekoa,

hiri artekoa, saihesbidea, portu eremua edo ezer ez.

- **Bide-zoruaren akzio-mota**. Beharbada, datu-basean dauden datu garrantzitsuenetariko bat da. Atal honetan adierazten da proiektuan burututako bide-zoru sekzioa **<u>trazatu berria</u>** den (errepide berriko trazatu berrietan bezala, saihesbideetan bezala edo autobide baten tarte berrian) edo **<u>errehabilitazio- eta mantentze-lana</u>** den bide-zoruan. Azken kasuan, aurreko bide-zoru sekzioa kontsideratu behar da. Horren ondorioz, proiektuak berezitu ahal dira. Alde batetik, trazatu berriko proiektuetan hasierako bide-zoruaren sekzioa identifikatzen da. Beste aldetik, mantentze- eta errehabilitazio-lanetan independente kontuan hartzen dira tarte berrietatik. Batzuetan, mantentze- eta errehabilitazio-lanak aplikatzen dira bide-zoru ezezagun baten gainean. Orduan, errodadura geruzak baino ez dira ezagutzen eta ez sekzio osoa. Hala eta guztiz ere, dagoen bide-zoruaren parte bat ezaguna da eta aztertu ahal da.

- Fresaketa lodiera. Bide-zoruaren geruzaren bat fresatzen bada, mantentze- eta errehabilitazio-jarduеretan baino ez, fresatutako lodiera dagoen bide-zoruan sartu ahal da. Fresatutako lodierari buruzko informazioan lodiera, cm-tan baino ez da aipatzen, eta ez da Ministerio de Fomento (2011a)-k proposatzen duen bezain zehatza, geruzak fresatzea ez da hain ohikoa Bizkaian.

- Oinarrizko geruza bituminosoa. Oinarrizko geruza bituminosoan erabilitako materialaren izendatzea adierazten du, gaur egungo legearen arabera, aurreko izendatzea (6.14 taulatik), erabilitako betun mota (baina normalean ez da adierazten) eta geruzaren lodiera, cm-tan.

- Tarteko geruza bituminosoa. Tarteko geruza bituminosoan erabilitako materialaren izendatzea adierazten du, gaur egungo legearen arabera, aurreko izendatzea (6.15 taulatik), erabilitako betun mota (baina normalean ez da adierazten) eta geruzaren lodiera, cm-tan.

- Errodadura geruza bituminosoa. Errodadura geruza bituminosoan erabilitako materialaren izendatzea adierazten du (6.16 taulatik), erabilitako izendatze espezifikoa gaur egungo legearen arabera, aurreko izendatzea (6.17 taulatik), erabilitako betun mota (baina normalean ez da adierazten) eta geruzaren lodiera, cm-tan.

- Azpi-oinarrizko geruza. Azpi-oinarrizko geruzan edo geruzetan erabilitako materialak adierazten dira, erabil daitezkeen materialen artean (6.18 taula) eta bakoitzaren lodiera, cm-tan.

- Oharrak. Bide-zoruaren egiturari buruzko edozein ohar sar daitezke zalantzak argitzeko edo konkordatzen ez duen zerbait azaltzeko.

6.14 taula. Oinarrizko geruzan egon daitezkeen material bituminosoak (Ministerio de Fomento, 2011a)

Oinarrizko geruza bituminosoa	
Izendatzea UNE-EN 13108-ren arabera	**Aurreko izendatzea**
AC 32 base S	S25
AC 22 base G	G20
AC 32 base G	G25
AC 22 base S MAM	MAM

6.15 taula. Tarteko geruzan egon daitezkeen material bituminosoak (Ministerio de Fomento, 2011a)

Tarteko geruza bituminosoa	
Izendatzea UNE-EN 13108-ren arabera	**Aurreko izendatzea**
AC 22 bin D base S	D20
AC 22 bin SG	S20
AC 32 bin S	S25
AC 22 bin S MAM	MAM

6.16 taula Errodadura geruzan egon daitezkeen materialak (Ministerio de Fomento, 2011a)

Materialak errodadura geruzan	
AC	Hormigoi bituminosoa
BBTM	Beroan egindako nahaste etena
PA	Nahaste bituminoso dranaitzailea
LB1, LB2, LB3, LB4	Kare-esneak
TS	Gainazaleko tratamendua, geruza bakarra zabaltzea
DTS	Gainazaleko tratamendua, geruza bikoitza zabaltzea
HF	Hormigoizko bide-zoruak
HM	Hormigoi gihartsua
OM	Beste materialak

6.17 taula. Errodadura geruzan egon daitezkeen materialak (Ministerio de Fomento, 2011a)

Errodadura geruza bituminosoak		
Geruza bituminoso mota	**Izendatzea UNE-EN 13108-ren arabera**	**Aurreko izendatzea**
Nahaste bituminoso etenak	BBTM 8 A	F8
	BBTM 8 B	M8
	BBTM 11 A	F10
	BBTM 11 B	M10
Nahaste bituminoso dranaitzaileak	PA 11	PA 12
	PA 16	-
Hormigoi bituminosoa	AC 16 surf D	D12
	AC 16 surf S	S12
	AC 22 surf D	D20
	AC 22 surf S	S20

6.18 taula. Oinarri-azpiko geruzan egon daitezkeen materialak (Ministerio de Fomento, 2011a).

Oinarri-azpiko geruzen materialak	
GC	Hartxintxar eta zementua
SC	Zementu lurzorua
MG	Zagorrak
HF	Hormigoizko bide-zoruak
HM	Hormigoi gihartsua
GE	Zepa-pikortsua

Datu-basean proiektu guztiak sartuta, deskribatutako kategoria baten arabera sailkatu ahal dira. Errepide zehatz baten proiektuak aukera daitezke, trazatu berria daukaten proiektuak, etab. Aukera asko dituen tresna anitza da.

Datuak sartzen dira datu-basean Sustapen Ministerioaren gidaren irizpideak jarraituz (Ministerio de Fomento, 2011a): Hala eta guztiz ere, gidan ez da berezitzen hasierako bide-zoru egiturak eta mantentze- edo errehabilitazio-jarduera jasan duena. Bide-zoruari buruzko informazio guztia kasu bietan berdin sartzen da, baina Bizkaiko kasuan adierazten da bide-zoru sekzioa hasierako bide-zoru sekzioa edo konpondutako bide-zoru sekzio bezala kontsideratu ahal den.

6.5.2. Gestivían bide-zoruen egituraren ikuspena

Proiektuen informazio guztia Gestivían softwarean sartu denean, informazio hau bi modu ezberdinetan ikusi daiteke: **Bide-zoru Egitura** artxiboaren bidez edo **Errodadura Geruza** artxiboaren bidez. Modu egokia da, alde batetik, bide-zoruaren egitura osoa eta, beste aldetik, pneumatiko eta bide-zoruaren arteko errodadura geruzaren ezaugarriak aztertzeko, hurrenez hurren

6.5.2.1. Bide-zoru Egitura artxiboa

Bide-zoru Egitura artxiboak errepide osoa ikusten ahalbidetzen du, tarteetan zatitua, ezagunak diren bide-zoru egiturekin. Batzuetan, zeharkako bide-zoru sekzio osoa sartuta dago, eta beste batzuetan, ezagunak diren geruzak baino, proiektu eskuragarrien bidez. Artxiboak 6.19 taulan agertzen den antzeko taula ematen du.

6.19 taula. Bide-zoru Egitura Artxiboaren adibidea BI-2701 errepidean.

Errepidearen ardatza	Hasierako PK-a	Amaierako PK-a	PK-en arteko luzera (m)	Galtzada	Errei 1	Errei 2	Errei 3	Errei 4	Errei 5	Errei 6	Tartearen natura	Hasierako bide-zoruaren data	Zelaigunea
BI-2701 Nagusia	27+0000	27+0200	200	0	Bai	Bai					Hiri-artekoa	01/06/1994	
BI-2701 Nagusia	27+0200	27+0340	140	0	Bai	Bai	Bai	Bai			Hiri-artekoa	01/06/1994	
BI-2701 Nagusia	27+0340	27+0550	310	0	Bai	Bai					Hiri-artekoa	01/06/1994	
BI-2701 Nagusia	27+0550	27+0580	30	0	Bai	Bai					Hiri-artekoa	01/06/1994	
BI-2701 Nagusia	27+0580	27+0750	170	0	Bai	Bai					Hiri-artekoa	01/06/1988	
BI-2701 Nagusia	27+0750	27+0900	150	0	Bai	Bai					Hiri-artekoa	01/06/1988	
BI-2701 Nagusia	27+0900	27+0960	60	0	Bai	Bai					Hiri-artekoa		
BI-2701 Nagusia	27+0960	28+0050	150	0	Bai	Bai					Hiri-artekoa		
BI-2701 Nagusia	28+0050	28+0160	110	0	Bai	Bai					Hiri-artekoa		
BI-2701 Nagusia	28+0160	28+0260	100	0	Bai	Bai					Hiri-artekoa		
BI-2701 Nagusia	28+0260	28+0500	240	0	Bai	Bai					Hiri-artekoa		

Hasierako PK-a	Amaierako PK-a	Nahaste bituminosoak lodiera (cm)	Hormigoizko bide-zorua lodiera (cm)	Hormigoi-gihartsua lodiera (cm)	Hartxintxar eta zementua lodiera (cm)	Zementu eta lurzoru lodiera (cm)	Material pikortatua lodiera (cm)	Zepa pikortsua lodiera (cm)	Beste materialak lodiera (cm)	Bide-zoru sailkapena	Errodadura geruza Materiala	Errodadura geruza Materiala izendatzea	Errodadura geruza Betuna	Errodadura geruza Aurreko izendatzea	Errodadura geruza lodiera (cm)	Errodadura geruza Beste materialak	Errodadura geruza Data
27+0000	27+0200	19	0	0	0	0	25	0	0		LB2				0		01/12/2004
27+0200	27+0340	19	0	0	0	0	25	0	0		LB2				0		01/12/2004
27+0340	27+0550	19	0	0	0	0	25	0	0		LB2				0		01/12/2004
27+0550	27+0580	24	0	0	0	0	25	0	0		AC	AC 16 surfS		S12	5		01/12/2004
27+0580	27+0750	26	0	0	0	0	15	30	0		AC	AC 16 surfS		S12	5		01/12/2004
27+0750	27+0900	30	0	0	0	0	15	30	0		AC	AC 16 surfS		S12	5		01/12/2004
27+0900	27+0960	15									AC	AC 16 surfS		S12	5		01/12/2004
27+0960	28+0050	9									AC	AC 16 surfS		S12	5		01/12/2004
28+0050	28+0160	0									LB2				0		01/12/2004
28+0160	28+0260	5										AC 16 surfS		S12	5		01/06/2009
28+0260	28+0500	3										BBTM 11 A		F10	3		01/06/2012

Agertzen den datu bakoitza azaltzen da:

- **Errepidearen ardatza**. Galtzada bakarra errepide konbentzionaletan, hau da, enborra; edo gorantz edo beherantz doan galtzada, galtzada banandutako galtzadetan, PK-ren arabera.
- **Hasierako eta amaierako PK-ak**. Ezaugarri berdinak dituen tartearen hasierako eta amaierako PK-ak.
- **PK-en arteko luzera (m)**. Hasierako eta amaierako PK-en arteko benetako distantzia, Puntu Kilometrikoen arteko benetako distantzia kontuan hartuz.
- **Galtzada.** 0, 1 edo 2 adierazten da, Ministerio de Fomento (2011a)-ren irizpideen arabera.
- **Errei 1, 2, 3, 4, 5, 6**. Tartearen ezaugarriak dituzten erreiak adierazten dira. Tartean dagoen errei kopurua adierazteko ere balio du. Batzuetan, tarte batzuk aipatzen dira nahiz eta bide-zoru berdina izan, errei kopurua ezberdina bada.
- **Tartearen natura.** Tartearen natura aipatzen da: hiri-arteko, hiri-barneko, saihesbidea, etab.
- **Hasierako bide-zoruaren data.** Artxiboaren datu garrantzitsuenetariko bat da. Bide-zoru sekzioa Trazatu Berria proiektu baten bidez sartu baldin bada, 6.5.1 atalean komentatu den bezala, zerbitzuan jartzen den data aipatzen da. Irekitzearen dataz aparte, Trazatu Berria proiektu baten bidez bide-zoru egitura osoa ezagutzen dela adierazten du. Tarte berria martxan jarri zen data jakinarazten du. Data eskuragarria ez bada, bide-zoruaren geruza batzuk baino ez dira ezagutzen tarte horretan esan nahi du, mantentze- edo errehabilitazio-proiektu baten bidez.
- **Zelaigunea**. Eskuragarri badago, zelaigunearen sailkapena adierazten du Espainiako arauen arabera (Ministerio de Obras Públicas, 1976a; Ministerio de Obras Públicas y Urbanismo, 1986b; 1989a; Ministerio de Fomento, 2003b), edo Euskal Autonomia Erkidegoko arauen arabera (Departamento de Transportes y Obras Públicas, 2006; Departamento de Vivienda, Obras Públicas y Transportes, 2012). Hala ere, normalean, datu hau ez dago eskuragarri.
- **Nahaste bituminoso geruzen lodiera.** Bide-zoruaren nahaste bituminosozko geruza guztien lodiera adierazten du, cm-tan. Bide-zoru osoa ezaguna bada, Trazatu Berria proiektu baten bidez sartu bada (eta ondorengo mantentze- eta errehabilitazio-lanen bidez), nahaste bituminosoen lodiera osoa erakusten da. Tartearen hasiera bide-zorua ezagutzen ez bada, zutabe honetan ezagutzen diren geruza bituminosoen lodiera erakusten da, mantentze- eta errehabilitazio-proiektuen bidez sartutakoak. Hala ere, benetako lodiera handiagoa izan daiteke. Ezagutzen den benetako lodiera adierazten du.
- **Hormigoizko bide-zorua Lodiera** (HF). Bide-zoruaren hormigoizko bide-zoruaren geruzaren lodiera adierazten du, cm-tan.
- **Hartxintxar eta zementua Lodiera** (GC). Bide-zoruaren hartxintxar eta zementuko geruzaren lodiera adierazten du, cm-tan.

- **Zementu lurzorua**. Lodiera (SC). Bide-zoruaren zementu lurzoruaren lodiera adierazten du, cm-tan. Posiblea da Hartxintxar eta zementua (GC) eta zementu lurzorua (SC) sekzio berean izan, bide-zoruak dimentsionatzeko gidetan proposatzen den soluzioa batia.

- **Material pikortatua Lodiera** (MG). Bide-zoruaren material pikortatuaren lodiera adierazten du, cm-tan. Zagorra da gehien erabiltzen den materiala mota honetako geruzetan.

- **Zepa pikortsua Lodiera** (GE). Bide-zoruaren zepa pikortsuaren lodiera adierazten du, cm-tan. Bizkaian labe garaiak egon direnez, zepa eskuragarri zegoen eta errepideetan maiz erabili den materiala da.

- **Beste materialak Lodiera** (OM). Bide-zoruaren beste materialen lodiera adierazten du, cm-tan, aurreko materialak erabili ez badira.

- **Bide-zorua sailkapena.** Bide-zoruen sailkapena adierazi behar izango luke (malgua, erdi-malgua, erdi-zurrua eta zurruna) baina normalean ez da erabiltzen.

- **Errodadura geruza Materiala.** Errodadura geruzan erabilitako materiala adierazten du, 6.16 taulan erakusten direnen artean.

- **Errodadura geruza Materiala izendatzea.** Errodadura geruzan erabilitako materialaren izendatzea adierazten du, 6.17 taulan agertzen direnen artean.

- **Errodadura geruza Aurreko izendatzea.** Errodadura geruzan erabilitako materialaren aurreko izendatzea adierazten du, 6.17 taulan agertzen direnen artean.

- **Errodadura geruza Beste materiala.** Errodadura geruzan erabilitako materiala adierazten du, 6.16 taulan agertzen direnetako bat ez bada.

- **Errodadura geruza Data.** Oraingo errodadura geruza noiz gauzatu zen adierazten du. Data hasierako bide-zoruaren data berea bada, tartean inolako mantentze- eta errehabilitazio-lanik egin ez direla adierazten du. Datak ezberdinak direnean, mantentze- edo errehabilitazio-lanak egin direla esan nahi du. Hasierako bide-zoruaren data eskuragarri ez dagoenean, tartean burututako azken lana adierazten du, zeinek aurretik deskribatutako errodadura geruza gauzatu zuena.

Ikusten den gisa, Bide-zoru Egitura artxiboa informazio iturri baliagarria da errepidean aurki daitezkeen geruzak ezagutzeko. Bide-zoru sekzio osoa ezaguna denean azpimarratzen da eta datu eskuragarri guztiak erakusten dira. Bide-zoru osoa ezezaguna denean ere, informazio eskuragarria ere ematen du.

Hala eta guztiz ere, akats batzuk aurki daitezke.

- Ez du ematen hasierako bide-zoruaren proiektuaren kodea. Gainera, errodadura geruza baino ezagutzen ez denean, mantentze- eta errehabilitazio-proiektuaren kodea ez da ematen.

- Hasierako bide-zoruaren informazioa eskuragarri dagoenean, ez du erakusten egitura osoa. Nahaste bituminosoen lodiera osoa baino ez du aipatzen baina ez du jartzen zeintzuk diren bide-zorua osatzen duten beste geruzak. Informazio hau proiektuan aurkitu behar da.

- Artxiboak ez du ematen fresaketari buruzko informazioa. Proiektuetan aurkitu behar da datu hori.
- Nahiz eta hasiera bide-zoruari eta gaur egungo errodadurari buruzko informazioa eskuragarri egon, artxiboak ez du ematen momentu horien artean tartean gauzatu ahal diren mantentze- eta errehabilitazio-lanen informazioa. Batzuetan, nahaste bituminosoen lodieretan diferentzien bidez nabaritu ahal da, baina ez dago bitarteko proiektu horien informaziorik. Kasu hauetan informazioa bilatzeko errepidean gauzatutako proiektu guztien artean ikertu behar da.
- Errodadura geruzan, 6.17 taulan agertzen diren materialak baino ez dira agertzen. Gaur egungo errodadura geruza kare-esnea edo gainazaleko tratamendua bada, artxiboak ez du erakusten. Mota honetako materialek lodiera gehiagorik bide-zoruari ematen ez diotela kontsideratzen da. Hori dela eta, material hauek ez dira aipatzen errodadura geruza bezala bide-zoruaren egitura eraldatzen ez dutelako. Informazio hau Errodadura Geruza artxiboan egiaztatu behar da (6.5.2.2 azpi-atala).

6.5.2.2. Errodadura Geruza artxiboa

Errodadura Geruza artxiboa errepide osoa ikusten ahalbidetzen du, tarteetan sailkatuta errodadura geruzan dagoen materialaren arabera. Bide-zoruaren egitura osoa guztiz ezezaguna den kasuetan, informazio hau eskuragarri egon daiteke, 2000. urtetik aurrera gauzatuta mantentze- eta errehabilitazio-lanak datu-basean erregistratu direlako. Artxibo honek gauzatutako gainazaleko tratamenduei buruzko informazioa ematen du, errodadura geruzatik berezituz. Artxiboak 6.20 taulan erakusten denaren antzeko taula proportzionatzen du. Datu bakoitzak deskribatzen da.

- **Proiektuaren kodea**. Proiektuaren kodea adierazten da. Proiektuaren kode espezifikoa da, lanaren azpi-sailkapenarekin, bi zenbakirekin, Bide-zoruaren kodean azaldu den bezala.
- **Errepidearen ardatza**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Hasierako eta amaierako PK-ak**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **PK-en arteko luzera (m).** Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Galtzada.** Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Errei 1, 2, 3, 4, 5, 6**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Errodadura geruza Materiala**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Errodadura geruza Material mota**. Errodadura geruzan erabilitako materialaren natura adierazten du.
- **Errodadura geruza Material izendatzea**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Errodadura geruza Betuna**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Errodadura geruza Aurreko izendatzea**. Bide-zoru Egituran deskribatutakoaren antzekoa.
- **Errodadura geruza Data**. Bide-zoru Egituran deskribatutakoaren antzekoa.

- **Errodadura geruza Tratamendua**. Bide-zoruari aplikatutako gainazaleko tratamendua adierazten du, gauzatu baldin bada. Komentatu den bezala, Gestivíak era ezberdinetan interpretatzen ditu errodadura geruzak. Alde batetik 6.17 taulako materialek lodiera eraldatzen du. Bestetik, gainazaleko tratamenduak, kare-esneak edo geruza bakarreko tratamenduak, ez dute lodiera osoa ezta egitura aldatzen.

- **Errodadura geruza Tratamenduaren data**. Gainazaleko tratamendua burutu zen data adierazten du. Jakina, data horrek Errodadura geruza Dataren ondoren egon beha du.

Errodadura Geruza artxiboa beste datu-basearen funtsezko datua da. Errodadura geruzaren arabera, errepide bat zatitu ahal den tarte desberdinak aztertzen ahalbidetzen du. Errodadura geruzan material berea duen arte luze bat azpi-tarte batzuetan zatitzen du burututako gainazaleko tratamenduen arabera. Gainazaleko tratamendua berdina denean ere, berezituta dago tratamenduaren dataren arabera. Artxibo honek Bide-zoru Egitura artxiboa osatzen du, errodadura geruza zoom bat eginez. Errodadura geruzan bide-zoruaren ezaugarri garrantzitsuenetariko batzuk agertzen dira. Artxibo hau batez ere inportantea da bide-zoru egitura osoa ezezaguna denean, gutxienez errodadura geruzari buruzko informazioa ematen baitu. Gainera, artxibo hauetan oraingo errodadura geruza gauzatu duen proiektuaren kodea ikus daiteke, eta beraz, errazagoa da datu-basean proiektua aurkitzea errazagoa da.

Hala eta guztiz ere, akats batzuk azpimarratu behar dira:

- Nahiz eta oraingo errodadura geruzaren informazioa eskuragarri egon, ez da posible tartearen mantentze- eta errehabilitazio-lan guztien historia jakitea. Berriro, errepidean burututako proiektu guztiak kontsultatu behar dira lanen historia ezagutzeko. Oraingo informazioa baino ez dago eskuragarri.

- Taulan kodea erakutsi arren, errodadura geruzan materiala eta gainazaleko tratamenduak badaude, azkena baino ez du aipatzen, gainazaleko tratamendua, eta beraz, errodadura geruzari buruzko proiektua ez dago eskuragarri. Hala ere, bi datuak eskuragarri daudenez, Errodadura geruza Data eta Gainazaleko tratamendua Data, bilatzea ez da hain zaila

- Errodadura geruzari buruzko datu batzuk ez dira aipatzen, agregakinen Azeleratutako Leunketaren Koefizientea bezala.

6.20 taula. Errodadura geruzaren artxiboaren adibidea BI-2701 errepidean

Proiektuaren kodea	Errepidearen ardatza	Hasierako PK-a	Amaierako PK-a	PK-en arteko luzera (m)	Galtzada	Errei 1	Errei 2	Errei 3	Errei 4	Errei 5	Errei 6
PROJ-926	BI-2701 Nagusia	27+0000	27+0130	130	0	Bai	Bai				
PROJ-2098	BI-2701 Nagusia	27+0130	27+0200	70	0	Bai	Bai				
PROJ-2098	BI-2701 Nagusia	27+0200	27+0340	140	0	Bai	Bai	Bai	Bai		
PROJ-2098	BI-2701 Nagusia	27+0340	27+0420	80	0	Bai	Bai				
PROJ-926	BI-2701 Nagusia	27+0420	27+0840	420	0	Bai	Bai				
PROJ-116-92	BI-2701 Nagusia	27+0840	28+0050	270	0	Bai	Bai				
PROJ-116-1	BI-2701 Nagusia	28+0050	28+0160	110	0	Bai	Bai				
PROJ-927-952	BI-2701 Nagusia	28+0160	28+0260	100	0	Bai	Bai				
PROJ-1562-1256	BI-2701 Nagusia	28+0260	29+0280	540	0	Bai	Bai				

Hasierako PK-a	Amaierako PK-a	Errodadura geruza Material	Errodadura geruza Material mota	Errodadura geruza Material izendatzea	Errodadura geruza Betun	Errodadura geruza Aurreko izendatzea	Errodadura geruza Lodiera (cm)	Errodadura geruza Beste materialak	Errodadura geruza Data	Errodadura geruza Tratamendua	Errodadura geruza Tratamenduaren data
27+0000	27+0130	AC		AC 16 surf S		S12	5		01/06/1994	LB2	01/07/2012
27+0130	27+0200	AC		AC 16 surf S		S12	5		01/06/1994	Mikrofresaketa	08/05/2017
27+0200	27+0340	AC		AC 16 surf S		S12	5		01/06/1994	Mikrofresaketa	08/05/2017
27+0340	27+0420	AC		AC 16 surf S		S12	5		01/06/1994	Mikrofresaketa	08/05/2017
27+0420	27+0840	AC		AC 16 surf S		S12	5		01/06/1994	LB2	01/07/2012
27+0840	28+0050	AC		AC 16 surf S		S12	5		01/12/2004		
28+0050	28+0160									LB2	01/12/2004
28+0160	28+0260			AC 16 surf S		S12	5		01/06/2009		
28+0260	29+0280			BBTM 11 A		F10	3		01/06/2012		

6.6. Bide-zoruaren egoeraren datuak

Edozein Bide-zoruak Kudeatzeko Sisteman oinarrizko datu bezala, bide-zoruaren egoeraren datuak sartu behar dira hondatze-modeloak garatzeko (2.3 irudia). 3. kapituluan, ezaugarri nagusiak komentatu dira, ezaugarri horiek neurtzeko normalean erabiltzen diren tresnak deskribatu dira eta gehien erabiltzen diren adierazle edo indizeak zerrendatu eta azaldu ziren. Bizkaiko Foru Aldundiak datu hauek ere bildu ditu biltze-kanpainetan. Neurtutako datuak honako hauek dira: erregulartasuna, labainketarekiko erresistentzia eta egiturazko ahalmenaren balioak. Banan-banan deskribatzen dira, erregulartasun eta labainketarekiko erresistentziari arreta berezia eskainiz.

6.6.1. Bide-zoruaren erregulartasuna

Beste bide-administrazioak munduan zehar, Bizkaiko Foru Aldundiak erregulartasun datuak hartu ditu kudeatzen duen errepide-sarean International Roughness Index (IRI)-ren bitartez. IRI-a bide-zoruaren erregulartasuna karakterizatzeko munduan zehar gehien erabiltzen den indizea da, eta edozein beste ezaugarri kontuan hartuz ere bai. IRI datuak profilometro inertzial baten bitartez lortu ziren, hau da, profil zehatz bat lortu zen Class I-en barruan sartzen zen aparatu batekin (Sayers *et al*., 1986b). IRI beharrezko indize bezala inposatu da errepide berri baten kalitatea kontrolatzeko, edo zerbitzuan dagoen errepide batena, Sayers *et al*.-ek (1986b) garatu zuten ondoren. 1989an sartu zen Espainiako arauetan (Ministerio de Obras Públicas y Urbanismo 1989b), eraiki berri den bide-zoruaren ezaugarriak kontrolatzeko indize bezala ontzat emateko, eta ordura arte erabiltzen zen Viagrafoaren koefizientea (3.4.4.2.2) eta erregela zuzena (3.4.4.2.1) ordezkatzen hasteko. Ordutik aurrera, IRI-a erabili da errepide berriak eta konpondutako errepideak onartzeko indize bezala erabili da (Ministerio de Fomento, 1997; 2001; 2002; 2004a; 2004b; 2008, 2009; 2011b, 2015). Ondorioz, ohitura handia dago IRI-a erabiltzen erregulartasunaren indizea bezala.

IRI datu-biltze kanpainak ez dira urtero egin, urte zehatz batzuetan baino. IRI datuak hartu izan ziren BFAko errepide-sare osoan honako urte hauetan: 2000, 2002 (partzialki, 2000. urtean datuak hartu ez ziren tarte batzuetan baino ez), 2004, 2007, 2011 eta 2016. Datu biltzearen data ez dago erregistratuta, baina BFAko jendearen arabera, datuak udan hartu ziren, baina ez da posible data zehatza jakitea (eguna eta hilabetea), 2016. urtean salbu. Gainera, bide-zoru malguetan sasoiko aldakuntzak errodadura geruzaren bolumen aldaketengatik sortzen dira, eta 0,26 baino txikiagoak direla “lehor-izoztu” (*dry-freeze*) eremuetan eta 0,50 m/km baino baxuagoak “heze-izoztu” (*wet-freeze*) eremuetan (Perera and Kohn, 2002; Pérez-Acebo *et al*., 2018a).

Horrez gain, datuak 3, 4 eta 5 urteko denbora-tarteetan biltzen dira eta beraz, bide-zoruaren hondatzeak balizko sasoiko aldakuntza baino handiagoa izan behar du. Horren ondorioz, data zehatza eta da datu determinagarria, datuak beti udan hartu baitziren, hilabete hotzak ekidinez, eta data-biltze kanpainen arteko denbora-tartea (3 eta 5 urteren artean) luzeagatik.

IRI datuak errepidearen 100 m-tan behin aurkezten dira, tartearen hasierako eta amaierako PK zehatzak adieraziz. Errepide konbentzionaletan 100 m-ko tarte bakoitzean, datua eskuineko eta ezkerreko erreietan neurtu da. Galtzada banandutako errepideetan, galtzada bakoitzerako datuak hartzen dira, gorantz eta beherantz doazen galtzadak berezituz. Galtzada bakoitzean, IRI datuak eskuineko erreian (kanpokoa) eta ezkerreko erreian (kanpoko erreiaren ezkerraldean dagoen erreia) eskuragarri daude. Galtzada batean hiru errei edo gehiago egotekotan, ez dira datu gehiago hartu, eskuineko bi erreietakoak baino ez.

100 m-tan behin neurtutako datuak modu ezberdinean grabatu dira datu-basean. 6.21 taulan ikusten den bezala, urte bakoitzean, tartearen luzera ezberdina da (90 m-koa 2000n, 2004an eta 2011n, 99 m-koa 2007a eta 100 m-koa 2016an), 100 m-ko frekuentzia mantentzen den arren. Batzuetan, hasierako datua ez da 100 m-ko tarte batena, BFAk kudeatzen duen tartearen hasiera puntuan doitzeak egin behar direlako, 6.21 taulan ikusten den bezala. Horrek baldintzatzen du ondorengo tarteak berdinak ez izatea urte guztietan. Gainera, errepide batean sartutako tarte berriak, adibidez, saihesbide bat eraiki delako, ondorengo tarteen patroia ere eraldatzen du. Hala eta guztiz ere, hau ez da problema bat, tarte zehatz baten eboluzioa 2000tik 2016ra aztertzea posible da, hasierako eta amaierako puntuak metro batzuk aldatu arren.

6.21 taula. IRI datuen adibidea BI-631 errepidean. Errepidearen lehenengo 5 datuak data-biltze bakoitzean. Iturria: Gestivía

Datu-biltzearen urtea	Hasierako distantzia	Amaierako distantzia	Hasierako PK-a	Amaierako PK-a	IRI Eskuineko erreian	IRI Ezkerreko erreian
2000	0	120	31+0450	31+0570	2.45	2.97
2000	130	220	31+0580	31+0670	1.95	1.94
2000	230	320	31+0680	31+0770	5.88	4.7
2000	330	440	31+0780	32+0010	3.58	2.49
2000	450	540	32+0020	32+0110	2.64	2.17
2004	0	40	31+0450	31+0490	3.83	3.03
2004	50	140	31+0500	31+0590	3.16	2.71
2004	150	240	31+0600	31+0690	2.66	2.26
2004	250	340	31+0700	31+0790	4.68	3.96
2004	350	460	31+0800	32+0030	2.81	2.42
2007	0	99	31+0450	31+0549	2.92	2.86
2007	100	199	31+0550	31+0649	2.26	2.23
2007	200	299	31+0650	31+0749	5.72	5.35
2007	300	399	31+0750	31+0849	2.65	2.53
2007	400	519	31+0850	32+0089	2.38	2.49
2011	0	40	31+0450	31+0490	5.34	6.77
2011	60	150	31+0510	31+0600	1.77	1.7
2011	160	250	31+0610	31+0700	4.58	4.47
2011	260	350	31+0710	31+0800	7.26	6.38
2011	360	470	31+0810	32+0040	3.57	3.82
2016	0	100	31+0450	31+0550	2.71	2.88
2016	100	200	31+0550	31+0650	1.54	1.4
2016	200	300	31+0650	31+0750	1.04	1.36
2016	300	400	31+0750	31+0850	2.22	2.38
2016	400	500	31+0850	32+0070	2.3	2.32

6.6.2. Labainketarekiko erresistentzia eta egitura

Bizkaiko Foru Aldundiak labainketarekiko erresistentziako datuak ere neurtu ditu, marruskaduraren faktorearen bidez neurtuz, bide-segurtasuna oinarrizko faktorea da errepide-sarearen zerbitzu egokia emateko. Nahiz eta egiturazko ahalmen nahikoa eta erregulartasun egokia desiratu, Bizkaiko Foru Aldundiak bide-zoruen marruskadura kontrolatu du istripuetan daukan garrantziagatik (3.1.1 atala). 3. kapituluan azaldu den gisa, *International Friction Index* (IFI) indizea garatu zen arren, ez du lortu IRI-ak lortu duen nazioarteko estatusa eta beraz, ez da unibertsalki adoptatu. BFAk labainketarekiko erresistentziaren datuak hartu ditu *Sideway-force Coefficient Routine Investigation Machine* (SCRIM) aparatuaren bidez, Erresuma batuan garatua. 3. kapituluan komentatu den moduan, zeharkako marruskadura ematen duen aparatua da. *Side-force Coefficient* (SFC) Zeharkako Marruskadura Koefizientea ematen du. Hasieran, SCRIM irakurketa (SR)da ematen duen datua azpi-tarte bakoitzerako, 5, 10 edo 20 m-koa. SR-a da SFC-ren batez bestekoa azpi-tartearen luzera osoan, integral baten datua bezala adierazita, eta 100ez biderkatuta, abiadura zuzendu ondoren (3.17 ekuazioa).

Erresuma Batuan, SCRIM motako kamioiak erabiltzen hasi zirenean, SCRIM motako motorrak aparte utziz, zuzentze-faktore bat (0,78) ezarri zuten datu historikoak eta datu berriak konparatzeko eta SCRIM koefizientea (SC) sortu zuten. Espainian, SCRIM aparatua erabiltzen tradizio handia dago labainketarekiko erresistentzia lortzeko (Ministerio de Obras Públicas y Urbanismo, 1982; 1986a; 1991), baina ez da beharrezkoa zuzentze-faktorea erabili SCRIM motako kamioiak erabili direlako neurriak hartzeko. Horren ondorioz, esan daiteke bai Espainian bai Bizkaian neurtutako datu guztiak SCRIM koefizienteak (SC) bezala har daitezkeela.

Hasieran, Espainiako arauetan (PG-3 dokumentuan), errepide berri eta konpondutakoen kalitatea konprobatzeko SC datuak frakzio dezimala bezala adierazten ziren, hau da, 0tik 1era (Ministerio de Obras Públicas y Urbanismo, 1988a; 1989b; Ministerio de Fomento, 1997). Gaur egun, SC 0tik 100era adierazten da, hots, 100ez biderkatuta (Ministerio de Fomento, 2001; 2004a; 2008; 2011b; 2015). Gestivían, irizpide berdina mantentzeko, Bizkaiko SC datuak 0tik 100era adierazten dira ere bai.

IRI-en antzera, BFAk SCRIM koefizientea neurtu du urte zehatz batzuetan errepide-sare osoan, IRI neurtu zuen urte beretan: 2000, 2002 (partzialki, 2000n ez neurtutako tarte batzuetan baino ez), 2004, 2007, 2011 eta 2016. 2000, 2002, 2004 eta 2007ko datuen hartze-data ez dago erregistratuta. BFAko jendearen arabera, datuak udan hartu ziren, datu zehatza sartuta egon ez arren. 5.4.5.5. atalean luze azaldu den moduan, labainketarekiko erresistentziaren datuetan eragina handia dauka datuak biltzen den urte-sasoiak. Marruskaduraren datuak hartzen diren dataren garrantzia dela eta, BFAk bide-zoruak kudeatzeko sisteman sartu ditu datu-biltze kanpainen datak. Hala, 2011n, datu gehienak otsailean eta martxoan lortu zirela erregistratuta dago. Datuak erlazionatuta daude adierazitako datarekin, marruskadura balioak oso altuak baitira. 5.4.5.5 atalean komentatu den gisa, neguan, marruskadura balioak haien maximoetan daude. Horrez gain, 2011ko udan datu batzuk hartu zirela ere aipatzen da.

Beste aldetik, 2016an BFAk labainketarekiko erresistentziaren datuak hartu zituen udan, balioak haien minimoetan daudenean eta haien aldakuntza bere minimoan dagoenean. BFAk, Erresuma Batuko bide-administrazioak bezala, labainketarekiko erresistentzia minimo eskuragarri jakin nahi du. Neurtutako balio minimoa ezarritako atalase minimoa baino handiagoa bada, errepideak azpiegitura segurua dela esan daiteke.

SCRIM koefizientearen datuak 20 m-tan behin hartu dira, hasierako eta amaiera Puntu Kilometriko zehatzak adieraziz, IRI datuekin bezala. 20 m-ko tarte bakoitzean, errepide konbentzionaletan bi erreitako batean neurtu da. Ez da aipatzen zein erreitan neurtu da, baina normalena eskuineko erreian, PK-en araberakoa, neurtzen da. Galtzada banandutako errepideetan, datu bananduak ematen dira galtzada bakoitzerako, gorantz eta beherantz doazen galtzadak berezituz, berriro ere PK-en arabera. Galtzada bakoitzean, marruskadura balioak eskuineko erreian hartzen dira, kanpoan geratzen dena, ibilgailu astun gehienek errei horretatik zirkulatzen dutelako.

20 m-tan behin neurtutako datuak luzera ezberdinetan neurtu dira, nahiz eta datu guztiak 20 m-ko tarteei erreferentzia egin. 6.22 taulan erakusten den bezala, 2000n eta 2004an, 10 m-tan neurtu ziren 20 m-ko tarte bat irudikatzeko. 2011n eta 2016an, SFC balioak 19 m-ko luzerako tartetan neurtu ziren 20 m-ko SCRIM koefizientearen datua lortzeko. Salbuespena 2007an, 17 m eta 18 m-ko tarteetan neurtu zen, beraien artea distantziarik ez utziz.

Berriro ere, batzuetan lehenengo balioa ez da 20 m-ko tarte batena, Hala ere, kasu honetan, are errazagoa da tarte zehatz baten eboluzioa denboran zehar jarraitzea tarte bakoitzeko luzera laburragatik, 20 m baino ez dena. Berriro, errepidean sartutako tarte berriak ondorengo azpi-tarteen patroia eraldatzen du. Hala eta guztiz ere, hori ez da arazo bat tarte zehatz bateko marruskaduraren eboluzioa aztertzeko.

Horrez gain, egitura datuak bildu dira marruskadura datuekin batera. Erabilitako indizea Mean Profile Depth (MPD), Batez besteko Profil Sakontasuna, 3.3.9.3.2. atalean deskribatuta., laserrean oinarritutako aparatu batez neurtuta, eta mm-tan adierazita.Balioak SCRIM koefizientearen luzera berari erreferentzia egiten dio, 6.22 taulan ikusten den bezala.

6.22 taula. Labainketarekiko erresistentziaren datuen adibidea BI-633 errepidearen lehenengo metroetan. Iturria: Gestivía.

Datu-biltzearen urtea	**Hasierako distantzia**	**Amaierako distantzia**	**Hasierako PK-a**	**Amaierako PK-a**	**SCRIM Koefizientea**	**Egitura**
2000	0	20	31+0450	31+0470	38	0,80
2000	30	40	31+0480	31+0490	41	0,70
2000	50	60	31+0500	31+0510	39	0,80
2000	70	80	31+0520	31+0530	40	0,70
2000	90	100	31+0540	31+0550	40	0,90
2000	110	120	31+0560	31+0570	41	0,80
2004	0	20	31+0450	31+0470	40	1,02
2004	30	40	31+0480	31+0490	37	0,83
2004	50	60	31+0500	31+0510	39	0,95
2004	70	80	31+0520	31+0530	37	1,18
2004	90	100	31+0540	31+0550	38	1,07
2004	110	120	31+0560	31+0570	43	1,18
2007	0	18	31+0450	31+0468	66	1,28
2007	19	37	31+0469	31+0487	72	1,34
2007	38	56	31+0488	31+0506	79	0,91
2007	58	77	31+0508	31+0527	84	0,73
2007	78	97	31+0528	31+0547	81	1,02
2007	98	117	31+0548	31+0567	85	1,03
2011	0	19	31+0450	31+0469	58	1,42
2011	20	39	31+0470	31+0489	63	0,86
2011	40	59	31+0490	31+0509	60	0,80
2011	60	79	31+0510	31+0529	61	0,80
2011	80	99	31+0530	31+0549	65	0,81
2011	100	119	31+0550	31+0569	66	0,76
2016	0	19	31+0450	31+0469	44	0,63
2016	20	39	31+0470	31+0489	47	0,68
2016	40	59	31+0490	31+0509	53	0,77
2016	60	79	31+0510	31+0529	51	0,87
2016	80	99	31+0530	31+0549	48	0,86
2016	100	119	31+0550	31+0569	43	0,66

6.6.3. Egiturazko ahalmena

Bizkaiko Foru Aldundiak egiturazko ahalmenaren balioak hartu ditu kudeatutako errepide-sarean deflektografoa eta Dynatest-en bitartez.

Deflektografoarekin datuak 5 m ed 10 m-tan behin neurtu dira, errepidearen arabera honako urte hauetan: 2000, 2002, 2004, 2007, 2011, 2012 eta 2016, nahiz eta urte batzuetan errepide-sare osoan datuak ez hartu.

Deflexio maximoa eskuineko eta ezkerreko erreian neurtu zen errepide konbentzionaletan eta eskuinaldean dauden bi erreiak galtzada banandutako galtzada bakoitzean.

Dynatest Falling Weight Deflectometer (FWD) bezala karakterizatu ahal den aparatua da (3.5.3.6 atala). Datuak Dynatest-ekin hartu ziren luzera aldakorretan neurtze-puntuen artean, 20 m eta 50 m-ren arteko distantzietan. Datuak neurtu ziren urteak Deflektografoarekin neurtutakoen antzekoak ziren (2000, 2002, 2004, 2007, 2011, 2012 eta 2016), nahiz eta urte batzuetan errepide-sare osoan datuak ez hartu. Errepide konbentzionaletan, karga bakarra aplikatzen da, erreien artean ez berezituz. Galtzada banandutako errepideetan, kargak aplikatu ziren puntu bakarrean galtzada bakoitzean. Deflexio maximoaz gain, ondoan dauden geofonoetan ere erregistratu dira Gestivían

Horrez gain, errepideak zati homogeneotan zatitu dira deflexio karakteristikoa eta kalkulatutako deflexioaren arabera, Ministerio de Fomento (2003a)-ren irizpideak jarraituz kalkulatzen direnak.

6.6.4. Bide-zoruaren akatsak

Bizkaiko Foru Aldundiak bide-zoruaren akatsak neurtu ditu bere errepide-sarean, pitzadurak, batxeak eta gurpil-arrastoak barne.

Pitzadurak 2011 eta 2016an datuak hartu dira eta indize batzuk erabiliz. Alde batetik, 2011n, pitzaduren balio osoa erabili da, pitzadurak dituen azalera adieraziz, ehunetan, eta ezkerreko erreia eta eskuineko erreiaren batez besteko eginez lortzen da. Errei bakoitzeko pitzadurak neurtzeko krokodilo motako pitzadurak, luzetarako pitzadurak, zeharkako pitzadurak eta batxeak kontuan hartzen dira. Datu partzial guzti hauek ere kontsideratutako eremuaren portzentaje bezala adierazten dira. Beste aldetik, 206an, datuak aurkezteko Total *Cracking Index* (Pitzadura Osoaren Indizea) erabiltzen da, m/m adierazita, hau da, 0tik 1era. Indize hori lortzeko Longitudinal *Cracking Index* (Luzetarako Pitzadura Indizea), *Transverse Cracking Index* (Zeharkako Pitzadura Indizea) eta gurpil-arrastoarekin erlazionatutako indizea erabili dira. Balio hauek ere adierazten dute pitzadurak dituen eremua eta kontsideratutako eremua osoaren arteko erlazioa, m/m-tan. Gurpil-arrastoaren balioak 2004, 2007 eta 2016an neurtu dira 20 m-tan behin.

6.7. Ondorioak

Kapituluak Bizkaiko Foru Aldundiak erabiltzen duen Bide-zoruak Kudeatzeko Sistema deskribatu du. Arrazoi historiakoak azaldu dira errepide-sarea osoan (Espainiako Gobernuak kontzesioren bidez kudeatzen duen bide-sariko autobidea salbu) kudeatzeko eskumenak izateko. Eskumen horiek ematen dituzten legeak aipatu dira.

Orduan, bide-zoruak kudeatzeko sistemaren ezaugarriak komentatu dira, BKS batek izan behar duen informazioarekin konparatuz, 2. kapituluan azaltzen den bezala. Datu geometrikoak BKSen giden ideia nagusien araberakoak dira. Trafiko datuek ez daukate ESAL-ei (*Equivalent Single Axle Load*) buruz eta BKSk ibilgailu astunei buruzko informazioa baino ez du erabiltzen, ibilgailu astunak 3.500 kg baino gehiago pisatzen duena definitzen delarik. Ingurumen-datuei buruz, datu gutxi sartzen dira Gestivían, Bizkaiko probintziaren azalera txikia eta klima uniformeagatik.

Bide-zoruaren egitura eta proiektuen historiari dagokienez, ideia asko Espainiako Gobernuaren gidak deskribatzen direnak dira (Ministerio de Fomento, 2011a), baina tokiko egoeretara egokituta. Desberdintasun handienak bide-zoruaren informazioa nola sartzen den datza. BFAko BKSren kasuan, proiektuen informazioan datza. Informazio horrekin bi artxibo garrantzitsu prestatzen dira: Bide-zoru Egitura artxiboa eta Errodadura Geruza artxiboa, non bide-zoruaren sekzioaren eta errodadura geruzaren informazioa aurki daiteke, hurrenez hurren.

Bide-zoruaren egoerari dagokionez, BFAk erregulartasuna, labainketarekiko erresistentzia, egiturazko ahalmena eta bide-zoruen akatsen datuak bildu ditu. Erregulartasuna IRI-en (*International Roughness Index*) bidez adierazten da 100 m-tan behin. Labainketarekiko erresistentzia SCRIM aparatuaren bidez lortu da, SCRIM koefizientea (SC) lortuz 20 m-tan behin. IRI eta SC datuak errepide-sare osoan neurtu ziren 2000, 2004, 2007, 2011 eta 2016an.

7. kapitulua. Iragarritako aldagaiak eta modelo-mota hautaketa

7.1. Sarrera

Kapitulu honen helburua Bizkaiko probintziako errepideetarako bide-zoruen portaeraren modeloaren modelo-mota aukeratzea deskribatzea da, Bizkaiko Bide-zoruak Kudeatzeko Sistemaren arabera, Egoera Agenda izenekoa.

Lehenengo eta behin, 4.3 atalean azaldu den gisa, hondatze-modeloen bidez iragarriko diren indizeak (edo adierazleak) aukeratu behar dira, Hondatze-indize posibleen artetik, erabiliko direnak aukeratzen dira.

Bigarrenez, modelo-motaren aukeratzearen azalpen osoa ematen da. Bide-zoruen portaera-modeloen garapenean oinarrizko momentua da prozesu osoa baldintzatzen baitu. Beraz, modelo-mota posibleak baztertzeko arrazoiak aurkezten dira eta aukeratutako modelo-mota ondo arrazoitutako argumentuetan oinarritzen da.

Azkenik, aukeratutako modeloaren ezaugarri nagusiak azaltzen dira. Beharrezkoa da mota horretako modeloak garatzean agertzen diren arazoak eta agertutako arazo horiek gainditzeko soluzioak jakitea. Modelo bakoitzeko aukeratutako faktore azaltzaileak 8. eta 9. kapituluetan deskribatzen dira modeloaren garapenarekin batera.

7.2. Iragarritako aldagaiak aukeratzea

4. kapituluan azaldu zen bide-zorua modelizatzean modeloak zer aurreikusiko duen aukeratzea zen lehenengo pausoa. Aldagai posibleak bide-zoruaren egoeraren indize indibidualak, 3. kapituluan aurkeztutakoak, konposatutako indize bat indize indibidualak gehituz sortzen direnak edo akatsen larritasuna edo akats espezifiko baten zabalera.

6. kapituluan, Bizkaiko Foru Aldundiko Bide-zoruak Kudeatzeko Sistemarako bildutako datuak azaldu ziren. Haietako batzuk, gurpil arrastoak edo pitzadurak datu-biltze kanpaina batzuetan baino ez dira neurtu, eta beraz, ez da posiblea bizitza-zikloan zehar haien portaera jarraitzea. Hala ere, erregulartasunerako, labainketarekiko erresistentziarako eta egiturazko ahalmenaren ohiko indizeak bildu dira 2000, 2004, 2007, 2011 eta 2016ko datu-biltzetan. Horren ondorioz, azken hamarkadetan eraikitako bide-zoruak erabil daitezke hondatze-modeloetarako.

Indize eskuragarrien artetik, hurrengo hauek aukeratu dira iragartzeko:

- **International Roughness Index (IRI)** indizea erregulartasunerako. Bidearen erabiltzaileak nabaritzen duen erosotasuna ebaluatzen du. Bide-zoruaren akatsekin erlazio ona duela frogatu da eta beraz, bide-zoruaren portaera iragartzeko erabiltzen da (Meegoda and Gao, 2014; Park *et al*., 2007; Sandra and Sarkar, 2013). Bere egonkortasuna denboran zehar eta edozein baldintzetan erabiltzeko gaitasunagatik munduan zehar gehien erabiltzen den parametroa da.

- SCRIM koefizientea eta egitura (*texture*) balioa labainketarekiko erresistentziarako. Edozein errepidetan bide-segurtasuna oinarrizko ezaugarria da eta bide-administrazioek marruskadura nahikoa duten errepideak eraiki behar dituzte bide-istripuak murrizteko, batez ere errepide bustietan.

7.3. Bide-zoruaren portaera-modeloaren hautaketa

Atal honetan, dauden aurreikuspen modelo-motak azaltzen eta komentatzen dira. Lehengo eta behin, hondatze-modeloaren maila ezarri behar da: proiektu-maila edo sare-mailako modeloa. Orduan, modelo bakoitzaren abantailak eta desabantailak aurkezten dira. Batez ere, modelo gehienak erregulartasunari buruzkoak dira, modelo asko proposatu baitira (5.3 atala). Labainketarekiko erresistentziarako oso modelo gutxi aurkeztu dira, eta guztiak deterministakoak dira. Modeloen deskribapenaren ondoren, eta Bizkaiko Foru Aldundiko bide-zoruak kudeatzeko sistemaren datu eskuragarrien arabera, abantaila gehiago eta desabantaila gutxiago proportzionatzen dituen modeloa aukeratu da.

7.3.1. Proiektu maila edo sare-mailako modelatzea

2.4 atalean aurkeztu den bezala, modeloak bi maila nagusitan sailka daitezke bide-zoruak kudeatzeko sistemetan: proiektu-mailako eta sare-mailako modeloa.

- **Proiektu-mailako modelatzeak** nahi du errepide-tarte baten luzera mugatuko oraingo hondatzea eta bere aldakortasuna deskribatu. Oso ondo aurreikusitako etorkizuneko trafikoaren eta bide-zoruaren data zehatzen araberakoa da. Normalean bertan egindako neurketak behar ditu, hala nola, FWD-ren bidezko deflexioak edo zundaketak. Informazio horretan oinarrituta, posiblea da errehabilitazio-teknika aukeratzea beharrezko funtzionalitate-maila lortzeko.

- **Sare-mailako modelatzea** erabiltzen da konektatutako errepide-sare zabalaren batez besteko hondatze-maila aurreikusteko. Etorkizuneko trafiko aldakorrean oinarritzen da eta bide-zoruaren datuen araberakoa da. Datu horiek normalean eraikuntzaren momentuan erregistratutako datuetatik lortzen dira eta egoeraren neurketak trafiko-abiaduran gauzatzen dira, IRI-a, labainketarekiko erresistentzia eta deflexioak neurtzeko egiten den gisa. Informazio horretan oinarrituta, errehabilitazio- edo mantentze-lanen aukeren artean hautatu ahal da. Lanak hautatzen dira beharrezko errepide-sarearen egoeraren eta funtsa eskuragarrien arabera.

Ondorioz, proiektu-mailan portaera aurreikusten da errepide-tarte zehatz baterako. Hau ez da tesi honen helburua. Errepide-sare osorako hondatze-modeloa garatzea da bere xedea. Bizkaiko errepideei buruzko informazioa dago, baina ez da proiektu-mailako modeloak garatzeko behar den zehatza. Gainera, proiektu-mailako modelo bat errepide-tarte baterako baino ez litzateke erabiliko. Hala eta guztiz ere, lortu nahi da Bizkaiko Foru Aldundiak kudeatzen dituen errepide guztien etorkizuneko egoeraren aurreikuspena, funtsak hobeto esleitzen lagunduko duena.

Horren ondorioz. Bizkaiko errepide guztien batez besteko egoera-parametroak kalkulatuko dira eragina duten faktoreen arabera. Beraz, sare-mailako modeloak hautatzen dira.

7.3.2. Modelo subjektiboak

Hondatze-modelatzean modelo subjektiboak garatu ahal dira. Adituen ezagutza eta espezializazioa erabiltzen dira ereduak sortzeko, eta modeloak 5.3 atalean ikusitako edozein motatakoak izan ahal dira. 5.3.6 atalean komentatu den moduan, modelo subjektiboak bereziki interesgarriak eta erabilgarriak dira bide-administrazioak datu historikoak ez dituenean (Osorio-Lird *et al*., 2017).

Bizkaian bide-zoruak kudeatzeko sistema martxan jarri baino lehen, BFAko errepide-adituek aurreikusten zuten subjektiboki Bizkaiko errepide-sarearen bide-zoruaren eboluzioa. Teknika honek emaitza onak eman ditzake, adituek benetako ezagutza baldin badaukate, BFAn gertatzen zen bezala. Errepideen eraikuntza kudeatzen eta kontrolatzen 20 urte baino gehiago eman duten ingeniariek ezagutza bikaina daukate Bizkaiko errepide-sarearen bide-zoruen portaerari buruz.

Hala eta guzti ere, tesi honetako helburuetako bat tresna lagungarria proportzionatzea da, eta ez partziala, bide-zoruaren hondatzearen eboluzioa aurreikusteko edozein bide-administrazioko edozein ingeniarirentzat. Horrez gain, 6. kapituluan aurkeztu den gisa, Gestivían informazio asko eskuragarri dago, datu-biltze kanpaina batzuetako datuekin eta azken 3 hamarkadetako errepide-sare osoaren egitura, trafiko eta eraikuntza-, errehabilitazio- eta mantentze-lanen historiako datuekin.

Hori dela eta, modelo subjektiboak baztertzen dira Bizkaiko errepide-sarearen hondatze-modelo bezala. Portaera-modelo zehatzagoak garatu nahi dira, aukeratutako indizeetan benetan eragina duten faktoreak adieraziz. Faktore hauek lagungarriak izan daitezke etorkizuneko egoera aurreikusteko faktore hauek ezagunak edo aurreikusten badira.

7.3.3. Sare neural artifizialak (SNA)

Konputazio modelo hauek adibideetatik ikas dezakete sarrerak eta irteeren arteko harremana simulatzeko. Prozesuak ez du aldagaien arteko erlazio zehatza behar eta hainbat aldagaik emaitzan eragina daukatenean erabil daiteke (Osorio-Lird *et al*., 2017; Simpson *et al*., 1994), zenbakizko metodologiak eta estatistika metodo tradizionalen bitartez ebazten zailak diren arazo mota batzuei soluzio ematen laguntzen. Prestakuntza-tarte baten ostean, datu berriak sar daitezke ebaluatzeko. Hala ere, modelatzea konplexua da, datu esperimental ugari behar ditu. Mugatze nagusia sare neuralak "*kutxa beltza*" gisa jarduten dutela da eta ez da posible soluzio bat lortzeko bidea erraz jakitea (Flintsch and Chen, 2004).

BFAko BKSren datu kopurua nahikoa izan litzateke, batez ere erregulartasunaren aurreikuspenerako, Sare Neural Artifizial batean erabiltzeko. Datu horietako atal bat erabil daiteke prestakuntza-tarterako, eta horren ostean, beste datu guztiak modeloaren efikazia ezartzeko balio izango lukete.

Hala eta guztiz ere, Sare Neural Artifizialek "*kutxa beltza*" gisa aritzea bere erabilpenerako muga bat izan daitekeela ondorioztatu da, eta hori dela eta, Sare Neural Artifizialen bidezko modeloa baztertu da. Oso modelo zehatza garatu ahal da SNA-en bitartez, errepideen egoeraren etorkizuneko egoera aurreikusteko BFAko ingeniariei laguntza ematen, eta korrelazio altuko iragarpenak ere lortu ahal dira IRI-erako (Abdelaziz *et al*., 2018). Hala eta guztiz ere, mota honetako modeloek ez dituzte erakusten sartutako faktoreen eta lortutako emaitzen arteko harremanak. Ezin da jakin faktore bakoitzaren garrantzi erlatiboa eta lortutako ezagutza ezin izango litzaioke emango beste bide-administrazioari, nahiz eta zirkunstantzia berdinetan egon. Tesiaren helburua da transferitu ahal den modelo bat garatzea, egoera berdinetan edozein lekutan erabil daitekeena.

7.3.4. Probabilitatezko modeloak

Bi probabilitatezko modelo nagusiak, modelo bayestarrak eta modelo markoviarrak, indibidualki komentatzen dira.

7.3.4.1. Modelo bayestarrak

Modelo bayestarrek adituen panelen ebaluazioan objektibotasuna eskaintzen dute eta datu-base txikietarako emaitza fidagarriak eskaintzen dituzte. Konbinatzen dituzte zenbait gertakizunen probabilitateen aurrezko ezagutza eta ikusitako datuak (probabilitatea) gertaeraren probabilitatezko banaketaren adierazpen egokitua lortzeko (ondorengoa bezala sailkatzen dena).

Modelo bayestarren abantaila da ziurgabetasuna sartzearen gaitasuna da. Gainera, adituen iritziak sartzen ditu datu historikoak osatzeko datu historikoak eskuragarri ez daudenean (Amador-Jiménez and Mrawira, 2011a; 2011b). Modeloak analisirako hautatuko aurreko probabilitatearen banaketan datza.

Beste alde batetik, mota honetako modeloen desabantaila nagusia da zenbait ondorengo inferentzien ebaluazioa erabiltzen duela inferentzia bayestarra. Kate markoviarrak eta Monte Carloren metodoak analisi hauek errazten dute (Lunn *et al*., 2000).

Berriro ere, BFAko bide-zoruak kudeatzeko sistemak ez du behar datu gutxi erabiltzen duen portaera-modelorik. UTE Agenda de Estado-ren ingeniariek gauzatutako datu-bilketari esker, informazio asko dago eskuragarri. Horren ondorioz, ez da beharrezkoa adituen iritziak sartzea datu historikoak osatzeko datu kantitate handi bat eskuragarri dagoelako. Gainera, adituen iritziek sar dezaketen partzialtasuna ekidin nahi da. Hori dela eta, modelo bayestarra ukatzen da bide-zoruen hondatze-modelatzeko.

7.3.4.2. Markov-en kateetako modeloak

4. eta 5. kapituluetan azaldu den bezala, Markov-en kateetako modeloek bide-zoruen hondatzearen probabilitatea kontuan hartzen dute, trantsizio-probabilitateen matrizeen (TPM) bitartez bide-zoruen etorkizuneko egoera iragartzeko gaurko egoeran oinarrituta. Monte Carloren simulazioekin konbinatuta, modeloak bide-zoruaren egoeraren trantsizio estokatikoa islatzen du denboran zehar.

Literaturak erakusten du datu-base historiko handirik gabe posible dela mota honetako modeloak garatzea (Pérez-Acebo *et al*., 2017b; 2018a) eta beste ikerketan aplikatu dira arrakastaz (Black *et al*., 2005; Chamorro and Tighe, 2011; Hassan *et al*., 2015; Silva *et al*., 2000; Yang *et al*., 2006). Ortiz-Garía *et al*., (2006)-en arabera, Markov-en kateen prozesuaren mugak bide-zoruen hondatzea iragartzeko honako hauek dira: denboran diskretua da, egoera kopuru finitua izan behar du eta memoriarik gabeko propietatea bete behar du. Propietate honek, (Markov-en propietatea), adierazten du bide-zoruaren etorkizuneko egoera orainaldiko egoeraren araberakoa baino ez dela, eta ez dela iraganean egondako egoeren araberakoa. Muga hauek jar daitezke Markov-en kateak erabiltzeko bide-zoruen hondatzea iragartzeko honako puntu hauek kontsideratuz (Ortiz-García *et al.*, 2006):

- Bide-zoruaren hondatzea etengabea da denboran zehar baina bide-sarearen egoera puntu zehatzetan analizatzen da denboran zehar.
- Egoera kopurua finitua da baina kontsideratutako indizearen tarte kopuru finitu bat definitu ahal da.
- Bide-zoruen hondatzeak Markov-en propietatea betetzen duela suposatzen da (Kerali and Snaith, 1992) eta TPM definitzeko kontsideratutako eszenarioen definizioa oinarrizko joera da propietate hori suposatzearen ziurgabetasuna eratzeko.

Mota honetako modelatzean, gutxienez bi momentuetako datu-biltzeak behar dira TPM-an kontsideratutako egoera-tarte guztietarako. Modelo honetako muga bat TPM-an oinarrituta dagoela da. Beraz, beharrezkoa da TPM bat garatzea bide-zoruaren portaeran eragina izan ahal duten faktoreen konbinazio bakoitzerako, hala nola, materiala, egitura, trafikoa, klima, etab. Ikertzaileak normalean TPM serie labur bat garatu dute konbinazio posible guztietarako, familietan bilduz, adibidez, bide-zoru motaren arabera (Osorio-Lird *et al.*, 2017; Pérez-Acebo, 2017b; 2018a), errepide-mailaren arabera, trafiko bolumenak bertan sartzen direlarik eta horren ondorioz bide-zoruaren egitura (Hassan *et al.*, 2015; Osorio-Lird *et al.*, 2017; Pérez-Acebo *et al.*, 2017b, 2018a; Soncim *et al.*, 2017) ,edo klimaren arabera (Chamorro, 2012; Chamorro and Tighe, 2015; Soncim *et al.*, 2017). Hori dela eta, hainbat talde proposatzen badira, kalkulatu beharreko TPM-ren kantitatea oso handia da. Adibidez, Soncim *et al.*, (2017)-k konbinatu zituen trafikoaren konbinazioen araberako 11 TPMak ezarritako 4 klimarekin (azpi-hezea, hezea, idorra edo lehorra), eta 44 balio lortu zituzten. Hala ere, kasu honetan, adituen ezagutza erabili zen matrizeak garatzeko, datu historikoen falta dela eta.

Metodologia hau Bizkaian erabiltzekotan, 8 trafiko kategoria kontuan hartu behar dira, Espainiako arauetan adierazten denaren arabera (Ministerio de Fomento, 2003b). Beste zatiketa bat bide-zoruaren egiturarena izango litzateke, bide-zoru malguak eta bide-zoru erdi-zurrunak berezitzeko (bide-zoru zurrunak, hau da, Portland zementuzko hormigoizko bide-zoruak ez daude Bizkaian). Bide-zoru erdi-zurrunen artean, azpi-oinarrian hainbat material erabiltze dira, hala nola, hartxintxar eta zementua, lurzoru eta zementua, bien konbinazioa, eta zepa pikortsua. Azkenik, errodadura geruzaren materiala ere eragina duen beste faktore bat ere izan daiteke, eta hainbat aukera aurkitzen dira: Hormigoi bituminosoak, nahaste drainatzaileak, nahaste etenak eta kare-esneak). Horren ondorioz, kalkulatu beharreko matrizeen kopurua oso handia izango zen, eta TPM ez homogeneoak kontsideratzen badira, are handiagoa. Mota batzuk biltzen badira TPM-ren kantitatea murrizten da. Honek sinplifikazioa izango litzateke. Kasu honetan, Gestivíako informazio eskuragarria ez litzateke aprobetxatuko. Sinplifikazio hauek normalean erabiltzen dira oso datu gutxi daudenean eta errepideen sailkapenez gain ez dago beste sailkapenik (Pérez-Acebo *et al.*, 2017b; 2018a). Gainera, sinplifikazio horiek ez dute modelo zehatzagorik sortzen, batzuetan fidagarriak izan ezin direnak.

Hori dela eta, Markov-en kateen bidezko modeloak baztertu dira. Bide-zoruen hondatzearen berezko aldakortasuna galtzen da modelo hauek aplikatzen ez badira. Hala eta guztiz ere, modelo zehatzak garatzea nahiago izan da, aldatu ahal direnak, gutxi izan arren, faktoreen aldakuntzak kontuan hartu ahal izateko.

7.3.5. Modelo deterministakoak

Modelo deterministakoen artean, estatistikako erregresiozko analisiak dira gehien erabiltzen diren modeloak hondatze-modeloak garatzeko (Amador-Jiménez and Mrawira, 2011b). Erregresio linealez gain, erregresio modeloek erlazio ez linealak gara ditzakete eta sinpleak eta errazak dira aplikatzeko (Hassan *et al.*, 2015). Haien efikazia probatu da datu historiko eta esperimental askorekin (Arambula *et al.*, 2011; Dong *et al.*, 2015; Meegoda and Gao, 2014).

Modelo hauei buruz normalean aipatzen den desabantailak dira datu esperimentalen mugatik at ezin direla estrapolatu eta menpeko aldagaiaren balio bakarra sortzen dutela (Amador-Jimenez and Mrawira, 2011a). Beraz, kalibrazioa normalean gomendatzen da modelo deterministakoa alde batetik bestera eramaten denean.

Erregresio-analisia erabiliz lortutako portaera-modeloen zehaztasuna informazio historiakoaren eskuragarritasunaren araberakoa da, bide-zoruaren adina eta egoera denboran zehar barne (TAC, 2013). Modelo deterministakoek tradizio handikoak dira bide-zoruen portaera modelizatzeko, batez ere erregulartasunaren eboluziorako. Garrantzitsuenetariko eta gehien erabiltzen direnak mota honetako modeloak dira, hala nola, World Bank-ek (Munduko Bankua) proposatutakoak (garapen-bidean dagoen herrialdeetan erabili behar direnak funtsak lortzeko errepideak eraikitzeko) (Paterson, 1987; Kerali *et al.*, 2004) edo AASHTO-k proposatutakoak: modelo mekanizistak (AASHTO, 1993) edo, gaur egungo azken modelo mekanizista-enpirikoak (AASHTO, 2008; 2010; 2015). 5. kapituluan adierazi denez, bide-administrazio askok hondatze-modelo deterministakoak garatu dituzte (European Commission, 1997; European Communities, 1999; Busch *et al.*, 2010), batez ere erregulartasuna eta bide-zoruen akatsen progresioari buruzkoa. Gainera, ikerketa indibidualak eta bide-administrazio lokalek ere modelo deterministako asko garatu dituzte. Esan beharra dago Long-Term Pavement Performance (LTPP) datu-base mota honetako modelizazioaren erabilera bultzatzen arei da, beste motak ere erabiltzen diren arren.

BFAko BKSn dagoen informazio eskuragarriaren kantitate handia dela eta, **modelo deterministakoak hautatu dira** etorkizuneko erregulartasuna eta labainketarekiko erresistentziaren egoera iragartzeko Bizkaiko errepide-sarean. Gestivían gordetako datu historikoek sekzio ezberdinen eboluzioa aztertzen ahalbidetzen du. Gainera, Hasierako Bide-zorua artxiboak sartu direnez, non bide-zoruaren egitura osoa eta adina ezagunak diren, bide-zoruen hainbat geruzaren konbinazioaren progresioa aztertzen errazten dut, bai geruza bituminosoena bai azpi-oinarriko geruzena, adina eta trafikoaren arabera. Horrez gain, ondorengo errehabilitazio- eta mantentze-lanak ere sartuta egoteak konpondutako errepideen modeloak sortzea posiblea egiten du. Hala, mantentze- eta errehabilitazio-lan bakoitza aztertu ahal da ikuspuntu ekonomiko eta tekniko batetik. Laburbilduz, bide-zoruaren egiturari, errodadura geruzari, trafiko bolumenari, eraikuntzaren eta errehabilitazioaren adinari buruzko datu kantitate handiak modelo deterministakoa hautatzera bultzatu du, faktore batzuk aldatzen direnean sortzen diren bariazioak aprobetxatzeko asmoz.

7.4. Modelo mekanizista, enpirikoa edo mekanizista-enpirikoa

Hurrengo pausua bide-zoruaren portaera modelizatzeko modelo deterministakoa hautatu denean, modelo mekanizista, enpirikoa edo mekanizista-enpirikoaren artean hautatzea da erregulartasunaren progresioa kontuan hartuz.

4.4.5 atalean komentatu den bezala, modelo mekanizistak garatu dira erantzun primarioak (portaera) aztertuz, hala nola, karga, tentsioa edo deflexioa (Haas *et al.*, 1994). Bide-zoruetan portaeraren oinarrizko teorietatik lortutako kargak eta tentsioetan oinarritzen dira. Orokorrean, mota honetako modeloak garatzen dira proiektu-mailako ebaluazioetarako, laborategiko entsegu eta bide-zoruko entsegu azeleratu askoren ondoren (Roberts *et al.*, 2003; Molenaar, 2003). Modelo mekanizistak erabiliak izan dira iraganean bide-zoruen diseinurako, batez ere Iparramerikan (AASHTO, 1993) eta Sobietar Batasunean eta bere eraginpeko lurraldeetan (Pérez-Acebo, 2017b; 2018a). Hala ere, modelo mekanizistak bertan kalibratzea gomendatzen da, datu errealekin ez baitituzte egoki kontuan hartzen materialaren aldakortasuna, eguraldia, trafikoko kargak eta haien arteko konbinaketak (Uddin, 2006). Hori dela eta, portaera neurketa egokiak identifikatu eta bildu behar dira hondatze-modeloak garatzeko. Adibide ona Long-Term Pavement Performance (LTPP) datu-basea da, non 2.500 proba-sekzioak dauden Iparramerikan zeuden eta eraiki berri diren errepideetan, eta datu asko proportzionatzen die ikertzaileei. Hondatzearen oinarri mekanizistak eta bertako neurketak konbinatzen dituzten modeloak mekanizista-enpirikoak (ME) dira eta Iparramerikako bide-zoruak diseinatzeko gida berriaren oinarria dira, Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008).

Aldiz, modelo enpirikoak erabiltzen dira errepide-mailako analisietarako. Modelo hauek erlazionatzen dute menpeko aldagai bat, ikusitako edo neurtutako egiturazko parametroa edo parametro funtzionala, eta aldagai independente bat edo batzuek, azaltzaileak direnak. Eremu espezifiko batean ikusitako hondatze-joeraren analisi estatistikoan oinarrituta daude. Aldagaiak hoberen doitzeko erregresio analisiak normalean erabiltzen direnez, modelo hauei ere erregresio modeloak deitzen zaizkie (Haas *et al*., 1994).

7.3.1 atalean azaldu den gisa, tesi honen helburua Bizkaiko probintzian aurki daitezkeen bide-zoru sekzio guztietarako erabil daitezkeen hondatze-modeloak garatzea da, hau da, sare-mailako modeloa garatu nahi da. Gainera, modelo mekanizistek datu asko behar dute eta errepide bertan lortzea zaila diren parametroak behar ditu. Horrez gain, tesi honetan ez dira gauzatu laborategiko entsegurik, mota honetako modeloak garatzea ezinezkoa izanik.

Gainera, modelo mekanizista-enpirikoek, batez ere MEPDG-en (AASHTO, 2008) agertzen direnek, bide-zoruaren erantzuna kalkulatzen dute (kargak, tentsioak eta deflexioak) eta erantzun horiek erabiltzen dute denboran zehar kalteak gehitzea kalkulatzeko. Prozedurak erlazionatzen ditu kate metatuak eta bertan neurtutako bide-zoruaren kalteak. 5.42 eta 5.45 ekuazioetan aurkeztu den bezala, IRI-erako MEPDG-en proposatutako modeloetan honako parametro hauek erabiltzen dute: fatigako pitzaduraren azalera, zeharkako pitzaduren luzera, batez besteko gurpil-arrastoa eta lekuko faktore bat, zeinek eguraldiko parametroez gain zelaigunearen materiala 0,02 eta 0,075 mm-ko bahetatik pasatzen diren ehunekoak. Pitzaduraren datuak lortzeko datu hauek behar dira: tentsio elastikoa egiturazko erantzun modelo baten arabera, nahaste bituminosoen modulu dinamikoa konpresiopean, benetako betun edukiera, hormigoi bituminosoaren bao portzentajea, etab. Gurpil-arrastoak kalkulatzeko karga bertikal plastikoa egoera espezifiko batzuetan eta beste parametroak laborategietan lortzen direnak behar dira. Hori dela eta, ME modeloek behar dute laborategiko entseguak eta informazio zehatza proiektu bakoitzetik, eskuragarriak ez direnak BFAko BKSn. Hori dela eta, modelo mekanizista-enpirikoak ezin dira garatu IRI iragartzeko.

Horren ondorioz, modelo enpirikoak hautatzen dira Bizkaiko errepide-sarerako modelo deterministako egokiak bezala. Modelo enpirikoak gehien erabiltzen direla adierazten da haien sinpletasuna eta azaltzaile guztiak sartzeko gaitasuna direlako; eta literatura handia daukate, batez ere, IRI-a aurreikusteko (Haas *et al.*, 1994; Giummarra *et al.*, 2007; Mubaraki, 2010; Alaswadko *et al.*, 2017). Erregresio analisiak, metodologia linealak eta ez linealak barne, aukeratzen dira erregulartasunaren eboluziorako hondatze-modeloa sortzeko. Labainketarekiko erresistentziarako, proposatutako modelo guztiak enpirikoak dira, errepide bertan neurtutako datuak aztertuz (Szatkowski and Hosking, 1972; Roe and Hartshorne, 1998; European Commission, 1997; Transit New Zealand, 2002a; NZTA, 2013a), edo laborategi entseguak erabiliz (Rezaei *et al.*, 2009; Rezaei and Masad, 2013; Khasawneh, 2017). Beraz, antzeko modelo enpirikoak garatzea hautatzen da Gestivían gordetako bertako datuen arabera.

Laburbilduz, literaturako bide-zoruen portaera-modeloak aztertu eta gero, eta Bizkaiko bide-zoruak kudeatzeko sisteman (Egoera Agenda) dagoen informazioaren arabera, erabaki da modelo deterministakoa eta enpirikoa garatzea International Roughness Index (IRI) eta Zeharkako Marruskadura Koefizientea (SCRIM Koefizientearen bidez) iragartzeko Bizkaiko errepide-sarean. Erabaki honetarako arrazoi nagusiak jarraian laburtzen dira:

- Sare-motako modeloak aukeratu ziren Bizkaiko errepide-sare osorako modeloak garatu nahi zirelako. Gaur egun dauden bide-zoru sekzio guztietan aplikatzeko gai izatea nahi da.
- Modelo partzial bat ez zen aukeratu nahi, batez ere adituen ezagutzaren bidez sartzen dena modelo subjektiboetan.
- BKSn gordetako datu historikoen kantitate handiak, bide-zoruen sekzioaren bizitza-zikloa aztertzen ahalbidetzen duena, datu historiko gutxi daudenean erabiltzen diren beste modeloak ez erabiltzera bultzatzen du, probabilitatezko modeloak bezala. Bide-zoruaren portaeraren natura probabilitatezkoa baztertzen du, baina beharrezkoa ez dela ondorioztatu da.
- Bizkaian edo beste lekuren batean erabil daitezkeen modeloak garatzea da helburua. Sare Neural Artifizialak baztertu ziren "*kutxa beltza*" bezala aritzen direlako, eta ez dute ematen informaziorik parametro bakoitzeko eraginaz azken emaitzan. Nahiz eta determinazio-koefiziente (R^2) hobeagoak lortu ahal izan arren, aldagai azaldua eta aldagai azaltzaileen arteko erlazio zehatza jakitea, ulertzea eta erakustean nahiago izan da.
- Laborategiko entsegurik ez burutzeak modelo mekanizistak eta modelo mekanizista-enpirikoak baztertzera bultzatu du. Hala ere, modelo hauek ematen duten informazioa eragina daukaten faktoreei buruz kontuan hartzen da modelo enpirikoak garatzean.

7.5. Erregresio-analisiaren metodologia

Bizkaiko bide-zoruen portaerarako hautatutako modeloak erregresio-analisiaren bidezko deterministakoak eta enpirikoak direnez, metodologia aurkeztea lagungarria dela estimatu da, agertu ahal diren problema nagusiak eta normalean aplikatu ahal diren soluzioak problema horiek ebazteko ere azalduz.

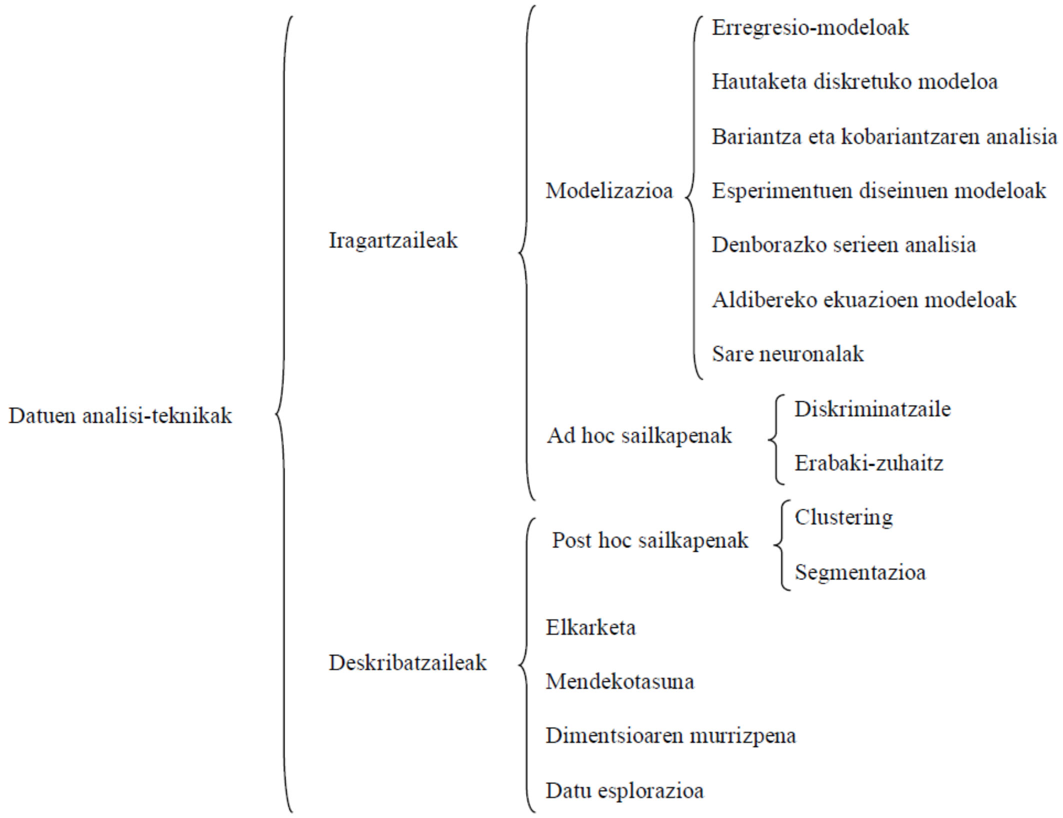
7.5.1. Datuen analisi-teknikei buruzko ikuspegi orokorra

Iragartzeko teknika estatistikoak datuak aztertzeko teknika estatistiko orokorren azpi-multzo gisa kontsidera daitezke.Bertan *ad hoc* sailkatzen eta modelizazioan orientatutako aurreikusteko teknikak eta post hoc sailkapenean orientatutako teknika deskribatzaileak ere sartzen dira, beste tekniken artean.

Aurreikusteko teknikak, ekonometrian, medikuntzan, biologian, epidemiologian eta ingeniaritzan erabil daitezkeenak, datuentzako modeloa adierazten dute aurreko ezagutza teorikoaren arabera. Datuentzako modelo teorikoa identifikatu denean, modelo estimatzen da eta, orduan, egiaztatzen da baliozko bezala onartu baino lehen. Momentu horretan posible da modelo erabiltzea iragartzeko (Pérez López, 2014)

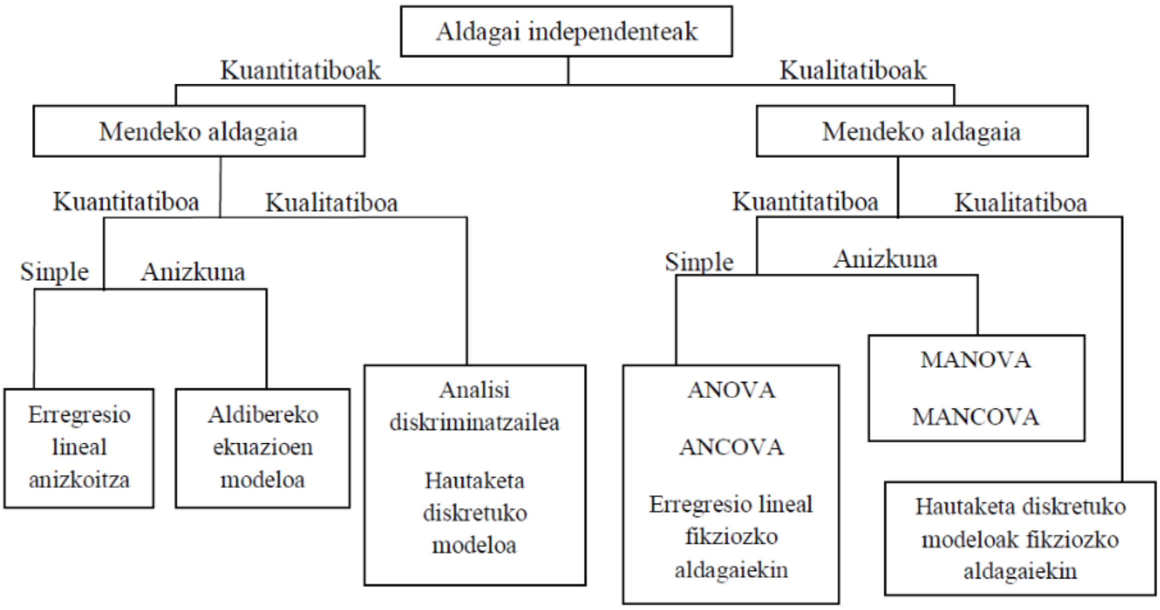
Iragartzeko tekniken artean mota guztietako erregresioak, denborazko serieak, bariantza eta kobariantzaren analisiak, modelo diskriminatzaileak, erabaki-zuhaitzak eta sare neuralak sartu ahal dira. Azken hiru teknikak ere profil portaerak edo klaseak lortzeko sailkatze-teknikak bezala sailkatu ahal dira, datu berriak sailkatzen duen modeloa proportzionatuz (Pérez-López, 2014).

Beste aldetik, teknika deskribatzaileetan, ez dago aldez aurretiko rolik aldagaietarako. Mendeko aldagai eta aldagai independenteren existentzia ez da onartzen ezta datuentzako modelorik. Modeloak sortzen dira patroiak aurkitzean. Talde honetan, honelako teknika hauek aurkitu ahal dira: *clustering* eta segmentazio teknikak, elkarketa eta mendekotasun teknikak, datu esplorazio-teknikak eta dimentsioa murrizpen-teknikak (faktoriala, osagai nagusiak, korrespondentziak, etab.) 7.1 irudiak erakusten du datuen analisi-tekniken sailkapen orokorra.



7.1. irudia. Datuen analisi-tekniken sailkapena. Pérez López (2014)-tik egokitua.

Azaldu den gisa, aldagai azalduen eta haiei dagozkien aldagai azaltzaileen arteko mendekotasuna baldin badago, modelo baten bidez azaldu ahal dena, iragartzeko teknika edo azaltzeko metodoa da (Pérez-López, 2014). Analisi-teknika hauek sailkatu ahal dira mendeko aldagai eta aldagai independenteen natura kualitatibo edo kualitatiboaren arabera, 7.2 irudian erakusten den gisa.



7.2 irudia. Iragartzeko tekniken sailkapena. Pérez López (2014)-tik egokitua.

Erregresio anizkoitzaren analisia teknika estatistikoa da mendeko aldagai kuantitatibo baten eta aldagai independente kuantitatibo batzuen arteko erlazioa analizatzeko (Hair *et al.*, 1999; Pérez López, 2014). Erregresio anizkoitzaren analisiaren helburua aldagai independenteak, haien balioak ezagunak direnak, erabiltzea da mendeko aldagaia (azaldua) iragartzeko. Erregresio anizkoitzaren adierazpen funtzionala honako hau da:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.1]$$

Non, printzipioz, bai mendeko aldagaia, *y*, eta aldagai independenteak, x_i kuantitatiboak dira. Gainera, erregresio anizkoitzak kualitatiboak diren aldagai independentearekin lan egiten ahalbidetzen du, fikziozko aldagaiak erabiliz gero (erregresio modeloak fikziozko modeloekin), aldagai kuantitatibo bihurtu eta gero (Pérez López, 2004).

Analisi kanonikoa (korrelazio kanonikoa) erregresio anizkoitzaren orokortzea da zeren eta mendeko aldagai kuantitatibo anizkoitzen eta aldagai independente kuantitatibo batzuen arteko erlazioa analizatzen ahalbidetzen baitu. Korrelazio kanonikoaren adierazpen funtzionala honako hau da:

$$G(y_1, y_2, \cdots, y_n) = F(x_1, x_2, \cdots, x_n) \qquad [7.2]$$

Teknika hauek ere mendeko aldagai edo aldagai independenteak (edo biak) kualitatiboak direnean.

Aldibereko ekuazioen modeloak teknika estatistikoak dira mendeko aldagai kuantitatibo anizkoitzen eta aldagai independente kuantitatibo batzuen arteko erlazioa analizatzeko. Adierazpen funtzionala honako hau da:

$$\vec{G}(y_1, y_2, \cdots, y_n) = \vec{F}(x_1, x_2, \cdots, x_n) \quad [7.3]$$

Modelo hau erregresio anizkoitzaren modeloaren orokortze bezala kontsideratu ahal da mendeko aldagai batzuen kasurako.

Analisi diskriminatzailea teknika estatistikoa da mendeko aldagai kualitatibo baten (azaldua) eta aldagai independente kuantitatibo (azaltzaile) batzuen arteko erlazioa analizatzeko. Aldagai independenteren balio ezagunak erabili nahi ditu zein mendeko aldagaiaren kategoriari dagokion iragartzeko. Analisi diskriminatzaileren adierazpen funtzionala 7.4 ekuazioa da.

$$y = F(x_1, x_2, \cdots, x_n) \quad [7.4]$$

Non mendeko aldagaia, *y*, kualitatiboa da eta aldagai independenteak kuantitatiboak dira. Erregresio anizkoitzaren kasu partikularra da.

Hautaketa diskretuko modeloak modelo diskriminatzaileen antzeko natura daukate baina kasu honetan iragartzen dena kategoria batean sartzearen probabilitatea da aldagai independentearen balio ezagunekin. Beraz, hautaketa diskretuko modeloak zuzen iragartzen du gertaera bat gertatzearen probabilitatea, aldagai independenteak definitutakoa. Probabilitatearen balioak 0 eta 1en artean dagoenez, iragarpenak 0 eta 1en arteko heinean egon behar dira. Betebehar hau betetzen duen modelo orokorra erregresio anizkoitzaren kasu partikularra da, modelo linealaren probabilitatea deitzen dena eta dauka honako adierazpen funtzional hau:

$$P_i = F(x_i, \beta) + u_i \quad [7.5]$$

Ausazko aldagai baten banaketaren funtzioa *F* bada, *P*-ren balioa 0 eta 1en artean dagoela ikusten da. *F* logit funtzioa izatekotan, modeloa Logit modelo bat da, eta bere adierazpena honako hau da:

$$P_i = F(x_i, \beta) + u_i = \frac{e^{x_i \beta}}{1 + e^{x_i \beta}} + u_i \quad [7.6]$$

Bariantzaren analisia (ANOVA) teknika estatistikoa da mendeko aldagai kuantitatibo (azaldua, erantzuna) baten eta aldagai independente kualitatibo (azaltzaile) batzuen arteko erlazioa analizatzeko. Helburua da lagin batzuk batez besteko bereko populazio beretik datozen determinatzea. Aldagai independenteen balio kualitatiboek determinatuko dute talde-serie bat mendeko aldagaian. Beraz, ANOVA modeloek neurtzen dute aldagai independenteen balioek mendeko aldagaian determinatutako taldeen batez bestekoen arteko diferentziaren esangura estatistikoa. Bariantzaren analisiaren adierazpen funtzionala honako hau da:

$$y = F(x_1, x_2, \cdots, x_n) \quad [7.7]$$

Non mendeko aldagaia, *y*, kuantitatiboa da eta aldagai independenteak ez dira kuantitatiboak.

Kobariantzaren analisia (ANCOVA) teknika estatistikoa da mendeko aldagai kuantitatibo baten eta aldagai independente batzuen arteko erlazioa analizatzeko, non aldagai independente batzuk kualitatiboak diren eta besteak kuantitatiboak diren. Bere adierazpen funtzionala honako hau da:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.8]$$

Non mendeko aldagaia, *y*, kuantitatiboa da eta aldagai independente batzuek kualitatiboak dira eta besteak kuantitatiboak dira.

Bariantzaren analisi anizkoitza (MANOVA) teknika estatistikoa da mendeko aldagai kuantitatibo batzuen eta aldagai independente kualitatibo batzuen arteko erlazioa analizatzeko. Helburua da egiaztatzea aldagai independenteen balio kualitatiboek determinatuko duten aldagai independenteek mendeko aldagaietan determinatutako talde serie baten batez besteko faktoreen berdintasuna. Beraz, MANOVA modeloek neurtzen dute aldagai independenteen balioek mendeko aldagaian determinatutako taldeen batez bestekoen bektoreen arteko diferentziaren esangura estatistikoa. Bariantzaren analisiaren adierazpen funtzionala honako hau da:

$$G(y_1, y_2, \cdots, y_n) = F(x_1, x_2, \cdots, x_n) \qquad [7.9]$$

Non mendeko aldagaiak kuantitatiboak dira eta aldagai independenteak kualitatiboak dira.

Kobariantzaren analisi anizkoitza (MANCOVA) teknika estatistikoa da mendeko aldagai kuantitatiboen eta aldagai independenteen arteko erlazioa analizatzeko, non aldagai independente batzuk kualitatiboak diren eta besteak kuantitatiboak diren. MANCOVA-ren adierazpen funtzionala honako hau da:

$$G(y_1, y_2, \cdots, y_n) = F(x_1, x_2, \cdots, x_n) \qquad [7.10]$$

Non mendeko aldagai kuantitatiboak dira eta aldagai independente batzuk kualitatiboak dira eta besteak kuantitatiboak dira.

Erabaki-zuhaitzak aldagai kualitatibo baten eta aldagai independente kualitatibo batzuen arteko erlazioa analisia gauzatzen du. Bere adierazpena honako hau da:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.11]$$

Logit erregresioa eta analisi diskriminatzailearen kasuetan bezala, helburua da zein mendeko aldagai kualitatiboaren kategoriatan sailkatzen dira behaketak aldagai independente kualitatiboen balioen arabera. Normalean, erabaki-zuhaitzetan aldagai kuantitatiboak ere erabiltzen dira haien balioak taldeetan taldekatuz (normalean gehienez bost taldetan).

Erregresio anizkoitzak aldagai independente kualitatiboekin lan egiten ahalbidetzen du fikziozko aldagaiak erabiltzen badira aldagai kuantitatibo bihurtzen badira. Aldagai kualitatibo bakoitzari zenbaki bat ematen zaio.

Erregresio anizkoitzaren modeloa fikziozko aldagaiekin erregresio anizkoitzaren analisiaren antzekoa da eta bere ezberdintasuna da aldagai independenteak kualitatiboak ere izan daitezkeela. Beraz, mendeko aldagai kuantitatibo baten eta aldagai independente batzuen arteko erlazioa analizatzeko teknika estatistikoa da, non aldagai independenteak kualitatiboak eta kuantitatiboak izan daitezkeen. Erregresioaren helburua da aldagai independentearen balioak erabiltzea, ezagunak direnak, aldagai azaldu bakuna iragartzeko. Bere adierazpen funtzionala honako hau da:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.12]$$

7.1 taulak laburtzen ditu iragartzeko teknikak mendeko aldagaien eta aldagai independenteen naturaren arabera (kuantitatiboak edo kualitatiboak).

7.1 taula. Iragartzeko tekniken mendeko aldagaien eta aldagai independenteen natura (Pérez López, 2014)

Teknika	**Mendeko aldagaia**	**Aldagai independenteak**
ANOVA and MANOVA	Kuantitatiboa	Kualitatiboa
ANCOVA and MANCOVA	Kuantitatiboa	Kuantitatiboa eta kualitatiboa
Erregresio anizkoitza	Kuantitatiboa	Kuantitatiboa
Erregresio anizkoitza fikziozko aldagaiekin	Kuantitatiboa	Kuantitatiboa eta kualitatiboa
Korrelazio kanonikoa	Kualitatiboa eta kuantitatiboa	Kuantitatiboa eta kualitatiboa
Hautaketa diskretua	Kualitatiboa	Kuantitatiboa
Hautaketa diskretua fikziozko aldagaiekin	Kualitatiboa	Kuantitatiboa eta kualitatiboa

Ikusten den moduan, Bizkaiko errepide-sarean IRI eta SCRIM koefizientea iragartzeko modelo aproposenak erregresio anizkoitz linealen modeloak (edo erregresio linealaren modeloak aldagai baten araberakoa bada) eta ANCOVA modeloak. IRI eta SCRIM koefizienteak aldagai kuantitatiboak dira. Aldagai independente posibleak kuantitatiboak izango dira (adina, geruzen lodiera, trafiko bolumenak, beste modeloetan ikusi den bezala) edo aldagai kuantitatibo eta kualitatiboen konbinazioa. Aldagai kualitatiboen artean erabil daitezke bide-zoruaren mota (malgua edo erdi-zurruna) eta geruza bakoitzean erabilitako materialak. Aukeratutako aldagai independenteei buruzko azalpen gehiago aurkezten dira 8. eta 9. kapituluan. Hurrengo atalean, erregresio lineal anizkoitzari buruzko deskribapen zehatzagoa aurkezten da.

7.5.2. Erregresio lineal anizkoitza

Lehen komentatu den bezala, erregresio lineal anizkoitzak analizatu nahi du aldagai (mendeko aldagaia, azaldua) baten portaera, Y, aldagai azaltzaileren (independente) serie baten balioek emandako informazioa erabiltzen. Aldagai independenteak X_2,..., X_k dira. Modelo linealak honako adierazpen hau dauka(Sierra Bravo, 1994; Montgomery *et al.*, 2012; Pérez López, 2014):

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots + \beta_K X_K + u \qquad [7.13]$$

Erregresio koefizienteek (parametroak) β_1, β_2,..., β_k aldagai azaltzaileen efektuaren magnitudea adierazten dute, hau da, X_1, X_2, ..., X_K aldagai independenteek Y mendeko aldagaian daukan efektua. β_0 koefizientea modeloaren termino konstantea da eta u osagaia modeloaren errorea da. Mendeko aldagai eta aldagai independente bakoitzerako T behaketak eskuragarri badaude, modeloa honela adieraz daiteke:

$$Y_t = \beta_0 + \beta_1 X_{1t} + \beta_2 X_{2t} + \ldots + \beta_k X_{kt} + u_t \quad t = 1,2,3,\ldots,T \qquad [7.14]$$

Termino independentearen presentzia (ez beharrezkoa) modeloan, X_0 lehenengo aldagai bezala interpretatu ahal da, zeinen balioa beti 1 den.

Arazo nagusia da, Y aldagaiaren eta X_1, X_2,..., X_k aldagaien serieen arteko erlazioa 7.13 ekuazioan bezala dela suposatuz, eta T behaketa serie bat dagoela mendeko aldagai eta aldagai independente bakoitzerako, nola kalkulatu ahal diren β_1, β_2,..., β_k parametroen zenbakizko balioak informazio eskuragarrian oinarrituta. Balio horiek parametroen estimatzaileak dira. Modelo lineal honako hipotesi hauek suposatuz formulatzen da (Pérez López, 2014):

- X_1, X_2, ..., X_K aldagaiak deterministakoak dira, hau da ez dira ausazko aldagaiak lagin batetik balio konstante baitauka.
- Errorearen osagaia, u terminoa, ausazko aldagaia da eta bere batez bestekoa nulua da eta kobariantzaren matrizea konstantea eta diagonala da (matrize eskalarra). Beste era batera esanda, edozein t-rako u_t aldagaiaren batez bestekoa zero da eta bere bariantza σ^2 ez da t-ren araberakoa, eta gainera, $Cov(u_i,u_j)= 0$, edozein i eta edozein j-rako haien artean ezberdinak badira. Orduan, u_t -re bariantza konstantea izatean edozein t-rako (ez dela t-ren araberakoa) homoszedatizitatearen hipotesia da. Beste aldetik, $Cov(u_i,u_j)= 0$ edozein i eta j-rako, i eta j ezberdinak izatekotan, ez auto-korrelazioaren hipotesia da.
- Ausazko aldagaia da Y aldagaia, u aldagaiaren araberakoa baita.
- Errorearen espezifikaziorik ez dagoela suposatzen da, hau da, Y aldagaia azaltzeko esanguratsuak diren X aldagai guztiak modelo linealean sartuta dago suposatzen da.
- Linealki independenteak dira X_1, X_2, ..., X_K aldagaiak, hau da, haien artean ez dago erlazio lineal zehatzik. Hipotesi hau independentziaren hipotesia da eta betetzen ez denean, modeloak multikolinealtasuna duela esaten da.
- Batzuetan, erroreen normaltasuna ere kontuan hartzen den, u_t aldagaia normalki bananduta edozein t-rako daudela inposatzen duena.

7.13 ekuazioaren adierazpenera modela doitu nahi dela suposatuz eta mendeko aldagaiarentzat eta aldagai independenteentzat T behaketen serie bat dagoela, 7.14 ekuazioan bezala modeloa adierazten ahalbidetzen duena; karratu txikien metodo (*ordinary least squares*, *OLS*, ingelesez) erabil daiteke erregresio-koefizienteak kalkulatzeko. Metodo honek kontsideratzen du datuak hobeto doitzen duen kurba u errorearen bariantza minimizatzen duena dela (Montgomery *et al*., 2012; Pérez López, 2016). Karratu txikien funtzioak, S, honelako adierazpena dauka:

$$S(\beta_0, \beta_1, \ldots, \beta_k) = \sum_{t=1}^{T} u_t^2 = \sum_{t=1}^{T} \left(y_t - \left(\beta_0 + \beta_1 X_{1t} + \beta_2 X_{2t} + \ldots + \beta_k X_{kt}\right)\right)^2 \qquad [7.15]$$

S funtzioa β_1, β_2,..., β_k. koefizienteekiko minimizatu behar da, eta beraz, β_1, β_2,..., β_k koefizienteen karratu txikien estimatzaileek bete behar dute:

$$\frac{\partial S}{\partial \beta_0} = 2\sum_{t=1}^{T}\left(y_t - \left(\beta_0 + \beta_1 X_{1t} + \beta_2 X_{2t} + \ldots + \beta_k X_{kt}\right)\right)\left(-1\right) = 0 \qquad [7.16]$$

Eta $\beta_1,\ldots,\beta_k$ erregresio-koefizientetarako:

$$\frac{\partial S}{\partial \beta_j} = 2\sum_{t=1}^{T}\left(y_t - \left(\beta_0 + \beta_1 X_{1t} + \beta_2 X_{2t} + \ldots + \beta_k X_{kt}\right)\right)\left(-x_{jt}\right) = 0, \quad j = 1,2,\ldots,k \qquad [7.17]$$

7.16 eta 7.17 ekuazioak sinplifikatuz, karratu txikien ekuazio normalak lortzen dira:

$$\sum_{t=1}^{T} y_t = T\beta_0 + \beta_1 \sum_{t=1}^{T} x_{1t} + \ldots + \beta_k \sum_{t=1}^{T} x_{kt}$$

$$\sum_{t=1}^{T} y_t x_{1t} = \beta_0 \sum_{t=1}^{T} x_{1t} + \beta_1 \sum_{t=1}^{T} x_{1t}^2 + \ldots + \beta_k \sum_{t=1}^{T} x_{1t} x_{kt}$$

$$\vdots$$

$$\sum_{t=1}^{T} y_t x_{kt} = \beta_0 \sum_{t=1}^{T} x_{kt} + \beta_1 \sum_{t=1}^{T} x_{kt} x_{1t} + \ldots + \beta_k \sum_{t=1}^{T} x_{xt}^2 \qquad [7.18]$$

Ekuazioa hauek [7.18 ekuazioak] sistema bat osatzen dute, karratu txikien ekuazio normalak izenekoa, eta β_1, $\beta_2,\ldots,\beta_k$-rako ebatzi ahal dira ekuazio linealak ebazteko edozein metodo aproposarekin. Azpimarratu behar da p = k + 1 ekuazio daudela, bat erregresio-koefiziente bakoitzerako. Honela modeloa estimatzen da. Erregresio anizkoitzeko modeloekin lan egiteko matrize notazioa erabiltzea egokiagoa da. Hala, modeloa, datuak eta emaitzak era konpaktuago batean erakusten dira. Modeloa 7.19 ekuazioaren bidez azal daiteke (Groβ, 2003; Bingham and Fry, 2010; Olive, 2017):

$$y = X\beta + u \qquad [7.19]$$

Non

$$y = \begin{bmatrix} y_1 \\ y_2 \\ \vdots \\ y_t \end{bmatrix};\; X = \begin{bmatrix} 1 & x_{11} & x_{12} & \cdots & x_{1k} \\ 1 & x_{21} & x_{22} & \cdots & x_{2k} \\ & \vdots & \vdots & & \vdots \\ 1 & x_{t1} & x_{t2} & \cdots & x_{tk} \end{bmatrix};\; \beta = \begin{bmatrix} \beta_0 \\ \beta_1 \\ \vdots \\ \beta_k \end{bmatrix};\; u = \begin{bmatrix} u_1 \\ u_2 \\ \vdots \\ u_t \end{bmatrix}$$

Orokorrean, y da T x 1 behaketen bektore bat, X da T x k aldagai independenteren mailen matrize bat, β da k x 1 erregresio-koefizienteen bektore bat da eta u da T x 1 ausazko erroreen bektore bat da.

Estimatutako modela $\hat{y} = X\hat{\beta}$ bezala adierazten bada, erroreen bektorea $\hat{u} = y - \hat{y} = y - X\hat{\beta}$ bezala adieraz daiteke. Erroreen karratuen batuketa S bezala adierazten bada:

$$S = \hat{u}'\hat{u} = (\hat{u}_1, \hat{u}_2, \ldots, \hat{u}_T)\begin{bmatrix} \hat{u}_1 \\ \hat{u}_2 \\ \vdots \\ \hat{u}_T \end{bmatrix} = \sum_{t=1}^{T} \hat{u}_1{}^2 \qquad [7.20]$$

7.20 ekuazioa honela ere adieraz daiteke:

$$S = (y - X\hat{\beta})'.(y - X\hat{\beta}) = y'y - \hat{\beta}'X'y - y'X\hat{\beta} + \hat{\beta}'X'X\hat{\beta} = y'y - 2\hat{\beta}'X'y + \hat{\beta}'X'X\hat{\beta} \qquad [7.21]$$

$\beta'X'y$ 1 x 1 matrizea denez, edo eskalarra, eta bere matrize iraulia $(\beta'X'y)' = y'X\beta$ eskalar berdina da. Karratu txikienen irizpidea aplikatzea S eskalarra minimizatzearen baliokidea da. Karratu txikien estimatzaileek bete behar dute:

$$\frac{\partial S}{\partial \hat{\beta}} = -2X'y + 2X'X\hat{\beta} = 0 \qquad [7.22]$$

Eta hau honako adierazpen honetara sinplifikatzen da:

$$X'X\hat{\beta} = X'y \qquad [7.23]$$

7.23 ekuazioak karratu txikien ekuazio normalak dira. Ekuazio hauek 7.18 ekuazioan eskalar aurkezpenaren antzeko matrizea dira. Ekuazio normalak ebazten, 7.23 ekuazioaren alde biak $X'X$ matrizearen alderantzizkoaz biderkatu behar da, $(X'X)^{-1}$ alderantzizko matrizea existitzen dela suposatuta. Matrize hori, $(X'X)^{-1}$, beti existituko da erregresio koefizienteak linealki independenteak badira, hau da, X matrizearen zutabe bat ere ez da besteen zutabeen konbinazio lineaka. Beste era batera esanda, matrizearen heina k da. Betebeharra betetzen bada, sistemaren bi aldeak $(X'X)^{-1}$-z aurre-biderkatu ahal dira (Montgomery *et al.*, 2012; Rao *et al.*, 2017):

$$(X'X)^{-1}(X'X)\hat{\beta} = (X'X)^{-1}X'y \Rightarrow \hat{\beta} = (X'X)^{-1}X'y \qquad [7.24]$$

7.5.2.1. Karratu txikien estimatzaileen propietateak

Karratu minimoen estimatzaileen, $\hat{\beta}$, propietate estatistikoak erraz froga daitezke. Modeloa zuzena dela suposatuz:

$$E(\hat{\beta}) = E\left[(X'X)^{-1}X'y\right] = E\left[(X'X)^{-1}X'(X\beta + u)\right] = E\left[(X'X)^{-1}X'X\beta + (X'X)^{-1}X'u\right] = \beta \qquad [7.25]$$

$E(u) = 0$ eta $(X'X)^{-1}X'X = I$ direlako. Beraz, $\hat{\beta}$ **estimatzaile inpartziala** da modelo zuzena bada.

$\hat{\beta}$ -ren bariantzaren propietatea **kobariantza-matrizea**ren bitartez azaltzen da:

$$Cov\left(\hat{\beta}\right)=E\left\{\left[\hat{\beta}-E\left(\hat{\beta}\right)\right]\left[\hat{\beta}-E\left(\hat{\beta}\right)\right]'\right\} \quad [7.26]$$

Matrize simetrikoa da, p x p heinekoa, non diagonalaren j elementua $\hat{\beta}_j$-ren bariantza da eta diagonalean ez dauden beste ij elementuak $\hat{\beta}_i$ eta $\hat{\beta}_j$-ren arteko kobariantza da. $\hat{\beta}$-ren kobariantza-matrizea lortzen da $\hat{\beta}$-ri bariantzaren operatzailea aplikatuz:

$$Cov\left(\hat{\beta}\right)=Var\left(\hat{\beta}\right)=Var\left[\left(X'X\right)^{-1}X'y\right] \quad [7.27]$$

Eta $\left(X'X\right)^{-1}X'$ konstanteen matrize bat da eta y-ren bariantza $\sigma^2 I$ da, beraz:

$$Var\left(\hat{\beta}\right)=Var\left[\left(X'X\right)^{-1}X'y\right]=\left(X'X\right)^{-1}X'Var\left(y\right)\left[\left(X'X\right)^{-1}X'\right]'=$$

$$=\sigma^2\left(X'X\right)^{-1}X'X\left(X'X\right)^{-1}=\sigma^2\left(X'X\right)^{-1} \quad [7.28]$$

Hori dela eta, honela deitzen badugu $A=\left(X'X\right)^{-1}$, $\hat{\beta}$-ren bariantza $\sigma^2 A_{jj}$ da eta $\hat{\beta}_i$ eta $\hat{\beta}_j$-ren arteko kobariantza $\sigma^2 A_{ij}$. Gainera, karratu txikien $\hat{\beta}$ estimatzailea β-ren estimatzaile lineal inpartzial onena da frogatu ahal da (Gauss-Markov-en teorema).

Onartzen bada u_i erroreen banaketa normal dela, orduan, $\hat{\beta}$ ere β-ren egiantz handienen estimatzailea da. Egiantz handienen estimatzailea β-ren bariantza minimoko estimatzaile ez partziala da (Montgomery *et al*., 2012).

Beste aldetik, σ^2-ren estimatzailea garatu ahal da erroreen karratuen batuketa, SS_{res}, erabilita (Montgomery *et al*., 2012):

$$SS_{res}=\sum_{t=1}^{T}\left(y_t-\hat{y}_t\right)^2=\sum_{t=1}^{T}\hat{u}_t^2=\hat{u}'\hat{u} \quad [7.29]$$

Eta $\hat{u}=y-X\hat{\beta}$ ordezkatuz, lortzen da:

$$SS_{res}=\left(y-X\hat{\beta}\right)'\left(y-X\hat{\beta}\right)=y'y-\hat{\beta}'X'y-y'X\hat{\beta}+\hat{\beta}'X'X\hat{\beta}=y'y-2\hat{\beta}'X'y+\hat{\beta}'X'X\hat{\beta} \quad [7.30]$$

Eta $X'X\hat{\beta}=X'y$ denez, 7.30 ekuazioa beste hau bihurtzen da:

$$SS_{res}=y'y-\hat{\beta}'X'y \quad [7.31]$$

Frogatu ahal da erroreen karratuen batuketa $T-(k+1)=T-k-1$ askatasun-gradu dituela berarekin elkartuta, erregresio modeloan $p=k+1$ parametroak estimatzen direlako. **Erroreen batez besteko karratua** da:

$$MS_{res} = \frac{SS_{res}}{T-(k+1)} \quad [7.32]$$

Frogatu ahal da MS_{res}-ren itxarondako balioa σ^2 dela, eta beraz, σ^2-ren estimatzaile inpartziala 7.33 ekuazioaren bidez ematen da eta modeloaren araberakoa da (Montgomery *et al.*, 2012).

$$\hat{\sigma}^2 = MS_{res} \quad [7.33]$$

7.5.2.2. Bariantzaren analisia

Honako kontzeptu hauek sartzen dira (Pérez López. 2014; 2016):

- **Karratuen batuketa totala**, SS_T, behaketen karratuen batuketa zuzenduta da, eta behaketa guztien aldakortasuna mendeko aldagaiaren batez bestekoarekiko, $\bar{y}$, neurtzen du (7.34 ekuazioa)

$$SS_T = \sum_{t=1}^{T}\left(y_t - \bar{y}\right)^2 = y'y - T\bar{y}^2 \quad [7.34]$$

- **Modeloaren edo erregresioaren karratuen batuketa**, SS_R, eta erregresio modeloak azaltzen duen y_i behaketen aldakortasunaren kantitatea neurtzen du. Iragarritako $\hat{y}_i$-ren balioen aldakortasuna mendeko aldagaiaren batez bestekoarekiko (7.35 ekuazioa)

$$SS_R = \sum_{t=1}^{T}\left(\hat{y}_t - \bar{y}\right)^2 = \hat{y}'y - T\bar{y}^2 \quad [7.35]$$

- **Erroreen karratuen batuketa**, 7.29 ekuazioan azaldu dena. Modeloaren errorearen maila adierazten du y_t.-ren aldakortasuna azaltzen saiatzen denean.

7.29 eta 7.30 ekuazioetatik jakina da:

$$SS_{res} = \sum_{t=1}^{T}\left(y_t - \hat{y}_t\right)^2 = \hat{u}'\hat{u} = \left(y - X\hat{\beta}\right)'\left(y - X\hat{\beta}\right) = y'y - \hat{\beta}'X'y = y'y - \hat{y}'y \quad [7.\ 36]$$

Beraz, $y'y = \hat{y}'y + \hat{u}'\hat{u}$ adierazpena idatzi daiteke. Ekuazio honen bi aldeetatik $T\bar{y}^2$ kentzen bazaie, lortzen da:

$$\left(y'y - T\bar{y}^2\right) = \left(\hat{y}'y - T\bar{y}^2\right) + \hat{u}'\hat{u} \Rightarrow SS_T = SS_R + SS_{res} \quad [7.37]$$

Ondorioz, karratuen batuketa total, SS_T, bi osagai ditu, erregresioaren karratuen batuketa, SS_R, eta erroreen karratuen batuketa, SS_{res}. Hiru osagaiei Karratuen Batuketa deritze (Hair *et al.*, 1999; Montgomery *et al.*, 2012; Darlington and Hayes, 2017).

Karratuen batuketa bakoitzari bere askatasun-graduz zatitzen bada, **Batez Besteko Karratua** da. Erroreen banaketa normala denaren hipotesia onartzen bada, SS_R χ^2 banaketa jarraitzen du k askatasun-gradurekin, modeloan daude aldagai independentearen kantitatea. SS_{res}–k χ^2 banaketa jarraitzen du $T - k - 1$ askatasun-gradurekin. SS_T –k χ^2 banaketa jarraitzen du $T - 1$ askatasun-gradurekin.

Hori dela eta, modeloaren Batez Besteko karratua 7.38 ekuazioaren bidez adierazten da eta erroren Batez Besteko Karratua 7.39 ekuazioaren bidez adierazten da.

$$MS_R = \frac{SS_R}{k} \qquad [7.38]$$

$$MS_{res} = \frac{SS_{res}}{T - k - 1} \qquad [7.39]$$

Determinazio-koefiziente, R^2, erregresio modeloaren doitze globalaren neurri deskribatzailea da, eta honela definitzen da:

$$R^2 = \frac{SS_R}{SS_T} = 1 - \frac{SS_{res}}{SS_T} \qquad [7.40]$$

Aldagai independente edo azaltzaileen efektua kontsideratu gabe SS_T y-ren aldakortasunaren neurria denez eta SS_{res} y aldagaian geratzen den aldakortasunaren neurria denez aldagai independenteak kontsideratu ondoren, R^2 kontsideratzen da modeloak azaltzen duen aldakortasunaren proportzioa. Beti $0 \leq SS_{res} \leq SS_T$, betetzen denez, beteko da $0 \leq R^2 \leq 1$. R^2-ren balioak 1etik hurbil egoteak adierazi nahi du y aldagaian aurkitzen den aldakortasun gehiena erregresio-modeloak azaltzen duela. Orokorrean, R^2 ez da inoiz txikiagoa beste aldagai bat modeloan sartzen bada, aldagai horren kontribuzioaren balioa kontsideratu gabe. Hori dela eta, ez da erraza jakitea R^2 handia egiteak noiz adierazten du zerbait garrantzitsu. Erregresio-modeloak garatzen dituzten ikerlariek nahiago dute doitutako determinazio-koefizientea, R^2_{adj}, erabiltzea nahiago dute, honako adierazpen honekin definitzen den estatistikoa:

$$R^2_{adj} = 1 - \frac{SS_{res}/(T - k - 1)}{SS_T/(T - 1)} = 1 - \left(1 - R^2\right)\frac{T - 1}{T - k - 1} \qquad [7.41]$$

Ekuazio horretan, $SS_{res}/(T - k - 1)$ erroreen batez besteko karratua denez eta $SS_T/(T - 1)$ konstantea denez modeloan dauden aldagai gorabehera, R_{adj}^2 handiagoa egingo da, aldagai gehiago gehitzeak erroreen batez besteko karratua txikitzen duenean baino ez. Doitutako R^2 erabiltzea lagungarria da modeloan aldagai gehiegi sartzearen kontra, hau da, beharrezkoak ez diren aldagaiak ez sartzeko (Pérez López, 2014). Gainera, ikusten da $T \to \infty$ doanez, hau da, behaketa asko dagoenean, $(T - 1)/(T - k - 1) \to 1$ eta ez da k-ren araberakoa, modeloaren aldagai independentearen arabera. Horrez gain, $T \to \infty \Rightarrow \overline{R}^2 \to R^2$ (Pérez López, 2016; Darlington and Hayes, 2017).

SS_R eta SS_{res}-ren banaketetatik ondorioztatu ahal da F estatistikoak Snedecor-en F banaketa jarraitzen du

$$F = \frac{\frac{SS_R}{k}}{\frac{SS_{res}}{T - k - 1}} = \frac{MS_R}{MS_{res}} \to F(k, T - k - 1) \qquad [7.42]$$

Estatistiko hau erabiltze da erregresioaren esangura-testean, zeinek determinatzen duen y erantzunaren eta edozein X_1, X_2, ..., X_K. aldagai independenteren artean erlazio bat dagoen ala ez. Orokorrean, prozedura hau modeloaren egokitasunaren test globala bezala erabiltzen da. Dagozkion hipotesiak honako hauek dira:

$$H_0 : \beta_0 = \beta_1 = \cdots = \beta_k = 0 \qquad [7.43]$$

$$H_1 : \beta_j \neq 0 \quad \text{gutxienez } j\text{-ren baterako.} \qquad [7.44]$$

Hipotesi nuluaren errefusatzeak inplikatzen du X_1, X_2, ..., X_K aldagai independenteren bat modeloaren esanguran laguntzen duela. *F*-ren balioa handia bada, orduan probable da gutxienez aldagai independenteren bat nulua ez izatea, $\beta_j \neq 0$. Beraz, $H_0 : \beta_0 = \beta_1 = \cdots = \beta_k = 0$ hipotesia kontrastatzeko, *F* estatistikoaren testa gauzatu behar da eta errefusatu honako hau betetzen bada:

$$F > F_{\alpha,k,T-k-1} \qquad [7.45]$$

Orokorrean, test prozedura *Analysis-of-variance* (Bariantzaren analisia)-ren taulan laburbiltzen da, 7.2 taulan bezala (Montgomery *et al*., 2012; Darlington and Hayes, 2017; Binghan and Fry, 2010).

7.2 taula. Erregresio lineal anizkoitzaren esangurarako bariantzaren analisia

Aldakuntzaren iturria	Karratuen batuketa	Askatasun-graduak	Batez besteko Karratuak	*F*
Erregresioa (modeloa)	SS_R	k	$MS_R = SS_R/k$	MS_R/MS_{res}
Erroreak	SS_{res}	$T-k-1$	$MS_{res} = SS_{res}/(T-k-1)$	
Totala	SS_T	$T-1$		

7.42 ekuazioko estatistikoa 7.46 ekuazioan dagoen bezala ere adieraz daiteke, eta Fisher-Snedecor-en F banaketa jarraitzen du.

$$F = \frac{\left(\hat{\beta}-\beta\right)' X'X\left(\hat{\beta}-\beta\right)}{k\hat{\sigma}^2} \rightarrow F(k, T-k-1) \qquad [7.46]$$

Estatistikoa hipotesi nulua, $\left(\hat{\beta}_1, \hat{\beta}_2, \cdots, \hat{\beta}_1\right) = (0,0,\cdots,0)$, egiaztatzen ahalbidetzen du. Fisher-Snedecor-en *F*-ren kontrastearen *p*-balioa baxua bada, modeloaren estimatutako parametroen esangura globala onartzen da. Estatistiko honek konfiantza-tarteak aurkitzen ahalbidetzen du %α konfiantza-mailaz modeloaren β_i parametroen multzoarentzat.

Gutxienez aldagai azaltzaileen bat garrantzitsua dela determinatu denean, galdera logikoa da zein den. Erregresio-modeloari aldagai bat gehitzeak beti erregresioaren karratuaren batuketa handitzea eta erroreen karratuen batuketa gutxitzea dakartza. Erabaki behar da erregresioaren karratuen batuketaren handitzea nahikoa den modeloan aldagai gehigarria sartuz gero. Aldagai bat gehitzeak doitutako $\hat{y}$ -ren balioaren aldakuntza ere handitzen du, beraz, kontuz ibili behar da benetan erantzuna azaltze duten aldagaiak sartzeko. Horrez gain, garrantzirik gabeko aldagai bat gehitzekotan, erroreen batez besteko karratua handituko da, eta modeloaren erabilgarritasuna murriztuko du.

7.28 ekuazioan ikusi den bezala, $\hat{\beta}_i$-ren bariantza $\sigma^2 A_{ii}$ da, non A_{ii} $[X'X]^{-1}$ matrizearen (*ii*) elementua den. Era berean, $\hat{\beta}_i$ eta $\hat{\beta}_j$-ren arteko bariantza $\sigma^2 A_{ij}$ da, non A_{ij} $[X'X]^{-1}$ matrizearen (*ij)* elementua den. Emaitza horietatik ondorioztatu ahal da edozein β_i koefizienteren estimatzailearen, $\hat{\beta}_i$, esperantza matematikoa β_i, da eta bere bariantza $\sigma^2 A_{ii}$, non A_{ii} $[X'X]^{-1}$ matrizearen (*ii)* elementua den. Beraz, erroreen banaketa normalaren hipotesipean, 7.47 ekuazioaren estatistikoak banaketa normala jarraitzen du.

$$N_i = \frac{\hat{\beta}_i - \beta_i}{\sigma\sqrt{A_{ii}}} \rightarrow N(0,1) \qquad [7.47]$$

Lehen azaldu den bezala σ^2 –ren estimatzailea karratu txikien metodotik (eta egiantz handienetik, komentatu ez dena) $\hat{u}'\hat{u}/T$ da, baina ez da inpartziala. Errorearen bariantzaren estimatzaile inpartziala honako hau da:

$$\hat{\sigma}^2 = \frac{\hat{u}'\hat{u}}{T-k-1} \qquad [7.48]$$

Beste aldetik, frogatu ahal da $G = u'u/\sigma^2$ estatistikoak χ^2 banaketa jarraitzen duela, $T–k–1$ askatasun-graduekin, zeinek σ eta bere karratuarentzat konfiantza-tarteak eta hipotesi-kontrasteak kalkulatzen ahalbidetzen duen. N_i eta G estatistikoen banaketek honako hau ondorioztatzera eramaten du: $N_i/[G/(T-k-1)]^{1/2}$ estatistikoak Student-en t banaketa jarraitzen du, $T - k - 1$ askatasun-gradurekin. Beraz, honako estatistiko honek

$$T_i = \frac{\hat{\beta}_i - \beta}{\hat{\sigma}\sqrt{A_{ii}}} \qquad [7.49]$$

Student-en t banaketa jarraitzen du, $T - k - 1$ askatasun-gradurekin. Estatistikoak konfiantza-tarteak eta hipotesi-kontrasteak modeloaren β_i parametroentzat aurkitzen ahalbidetzen du..

Edozein β_i, erregresio-koefiziente indibidualerako esanguraren hipotesi-kontrasteak dira:

$$H_0 : \beta_i = 0, \qquad H_1 : \beta_i \neq 0 \qquad [7.50]$$

Ho: $\beta_i = 0$ errefusatzen ez bada, orduan X_i aldagai independentea modelotik kendu ahal dela adierazi nahi du. Test estatistikoa hipotesi honetarako honako hau da (Montgomery *et al.*, 2012; Pérez López, 2014; 2016):

$$T_i = \frac{\hat{\beta}_i}{\sqrt{\hat{\sigma}^2 A_{ii}}} = \frac{\hat{\beta}_i}{\hat{\sigma}\sqrt{A_{ii}}} = \frac{\hat{\beta}_i}{se(\hat{\beta}_i)} \qquad [7.51]$$

Non A_{ii}, $\hat{\beta}_i$-ri dagokion $[X'X]^{-1}$ matrizearen elementu diagonal, hau da, $[X'X]^{-1}$ matrizearen (*ii)* elementua. Test estatistikoaren izendatzaileari normalean **estimatutako errore estandarra** esaten zaio, edo sinpleago, $\hat{\beta}_i$ koefizientearen **errore estandarra**, eta honela adierazten da:

$$se(\hat{\beta}_i) = \sqrt{\hat{\sigma}^2 A_{ii}} = \hat{\sigma}\sqrt{A_{ii}} \qquad [7.52]$$

Ho: $\beta_i = 0$ hipotesi nulua errefusatzen da, honako hau betetzen bada

$$|T_i| > t_{\alpha/2, T-k-1} \qquad [7.53]$$

Azpimarratu behar da test partziala edo marjinala dela $\hat{\beta}_i$ erregresio-koefizienteak modeloan dauden beste $X_j (j \neq i)$ aldagai independenteren araberakoa da. Beraz, test hau X_i-ren **kontribuzioa**ren testa da, **modeloan dauden beste aldagai independenteak kontuan hartuta**.

7.5.2.3. Konfiantza-tarteak erregresio anizkoitzetan

Erregresio-koefiziente indibidualaren konfiantza-tarteak eta esperantzaren konfiantza-tarteak aldagai independenteen maila zehatz batzuk hartuta, garrantzi handia daukate erregresio anizkoitzean (Montgomery *et al.*, 2012).

7.5.2.3.1. Erregresio-koefizienteen konfiantza-tarteak

Erregresio koefizientetarako, β_i, konfiantza-tarteak eraikitzeko, beharrezkoa da suposatzea erroreek banaketa normala daukatela, independenteak dira (ez dira t-ren araberakoak) eta hain batez besteko nulua da eta haien bariantza σ^2 da (7.5.4 atalean erakusten da nola egiaztatu). Horren ondorioz, y_i behaketek banaketa normal eta independentea daukate eta haien batez bestekoa $\beta_0 + \sum_{i=1}^{k} \beta_i X_{ji}$ da eta bariantza σ^2 da. Karratu txikien estimatzailea, $\hat{\beta}_i$, behaketen konbinazio lineala denez, $\hat{\beta}$-ek banaketa normala dauka eta batez bestekoa β bektorea da eta kobariantza-matrizea $\sigma^2[X'X]^{-1}$ da. Honek inplikatzen du edozein $\hat{\beta}_i$ erregresio-koefizientearen banaketa marjinala normala da eta batez bestekoa β_i dela eta bariantza $\sigma^2 A_{ii}$, dela, non A_{ii} $[X'X]^{-1}$ matrizearen diagonaleko *i* elementua den. Hori dela eta, mota honetako estatistiko bakoitza

$$\frac{\hat{\beta}_i - \beta_i}{\sqrt{\hat{\sigma}^2 A_{ii}}}, \quad i = 0,1,\ldots,k \qquad [7.54]$$

Student-en *t* distribuzioa jarraituz banatzen da, $T - k - 1$ askatasun-gradurekin, non $\hat{\sigma}^2$ errorearen bariantzaren estimazioa da, 7.33 ekuaziotik lortuta. 7.54 ekuazioan emandako emaitzan oinarrituta, $\boldsymbol{\beta_i}$, i = 0,1,…,*k* **erregresio-koefizientearen konfiantza-tartea 100(1-α) konfiantza mailaz honela definitu ahal da:**

$$\hat{\beta}_i - t_{\alpha/2, T-k-1}\sqrt{\hat{\sigma}^2 A_{ii}} \leq \beta_i \leq \hat{\beta}_i + t_{\alpha/2, T-k-1}\sqrt{\hat{\sigma}^2 A_{ii}} \qquad [7.55]$$

7.5.2.3.2. Esperantzaren konfiantza-tarteak

Posiblea da erantzunarentzat puntu zehatz batean konfiantza-tartea eraikitzea, hala nola, X_{01}, X_{02},...,X_{0k}. X_0 bektorea honela definitzen da:

$$X_0 = \begin{bmatrix} 1 \\ X_{01} \\ X_{02} \\ \vdots \\ X_{ok} \end{bmatrix} \qquad [7.56]$$

Doitutako balioak puntu honetan honako hauek dira:

$$\hat{y}_0 = X_0' \hat{\beta} \qquad [7.57]$$

Hau da E(y/X_0)-ren estimatzaile inpartziala, zeren eta $E(\hat{y}_0) = X_0'\beta = E(y|X_0)$ den eta $\hat{y}_0$-ren bariantza da:

$$Var(\hat{y}_0) = \sigma^2 X_0' [X'X]^{-1} X_0 \qquad [7.58]$$

Horren ondorioz, $X_{01}, X_{02},...,X_{0k}$ puntuan batez besteko esperantzaren konfiantza-tartea 100(1 – α) konfiantza-mailaz honako hau da:

$$\hat{y}_0 - t_{\alpha/2,T-k-1}\sqrt{\hat{\sigma}^2 X_0' [X'X]^{-1} X_0} \leq E(y|X_0)_i \leq \hat{y}_0 + t_{\alpha/2,T-k-1}\sqrt{\hat{\sigma}^2 X_0' [X'X]^{-1} X_0}$$

7.5.3. Hipotesiak erregresio lineal anizkoitzeko modeloan

Edozein erregresio lineal anizkoitzeko modeloan bete behar diren oinarrizko hipotesiak 6 taldeetan batu ahal dira modeloaren osagaien arabera. Talde bakoitzeko hipotesiak hurrengo azpi-ataletan komentatzen dira. Lehenengo taldeak modeloaren indize globalei erreferentzia egiten die. Bigarren taldeak modeloaren estimatutako parametroen esangura indibiduala eta baterako esanguraren testez osatzen da. Hirugarren taldea u ausazko aldagaiari buruzko da. Laugarren taldeak aldagai independenteei erreferentzia egiten die. Bosgarren taldean itxura funtzionalean oinarritzen da. Azken taldea (β_0, $\beta_1, \beta_2,..., \beta_k$) parametroen bektoreei buruzko hipotesiez osatzen da (Hair *et al*., 1999; Pérez López, 2016).

7.5.3.1. Modeloaren indizeak. Determinazio-koefizientea eta informazioaren kantitateari buruzko estatistikoa

Erregresio modelo batean kontuan hartu beharreko lehenengo adierazlea determinazio-koefizientea da. **Determinazio-koefizientea** (R^2), 7.40 ekuazioan definitua, modeloaren doitze globalaren neurri deskribatzailea da. Modeloan sartutako aldagai independenteek azaltzen dute mendeko aldagaiaren aldakuntzaren proportzioa kuantifikatzen du, eta 0 eta 1en artean dago. R^2 handiago batek modelo hobeagoa esan nahi du, baina ideia hau ezin da kontuan hartu aldagai azaltzaile gehiago sartzeko modeloan. Arazo hau ebatzi ahal da **doitutako determinazio-koefizientea** erabiliz, R^2_{adj} , 7.41 ekuazioan definitua. Azaldu den gisa, behaketa askorekin, doitutako determinazio-koefizientea ez da k-ren araberakoa, modeloaren aldagaien kopurua. Hori dela eta, erregresioaren kalitatearen neurri ona kontsideratu ahal da. Zenbat eta R^2_{adj}

handiagoa, modeloa hainbat eta hobeagoa.

Erabiltzen diren beste adierazle talde bat informazioaren kantitatearen estatistikoak dira (edo iragartzeko ahalmena).

Erregresio anizkoitzeko modeloa doitzean, hainbat modu daude (Hair *et al.*, 1999; Pérez López, 2014; 2016). Aldagai guztiak batera pausu bakarrean sartuz egin daiteke, metodo ohikoa dena. **Pausoz pauso hautaketa** (*step by step selection*) analisiak aldagai berriak sartzeko eta daudenak baztertzeko irizpideak espezifikatzen ahalbidetzen du, betebehar batzuk betetzen badira pausu bakoitzean. Sartuta ez daudenen artetik aldagai berria ekuazioan sartzen da, F-ren probabilitate txikiena duena, probabilitate hori nahikoa bada. Modeloan sartutako aldagaiak baztertzen dira haien F-ren probabilitatea nahiko handia bada. Prozedura amaitzen da aldagai hautagai gehiagorik. ez dagoenean sartzeko eta aterako. **Aurrera pausoz pauso** (*forward stepwise*) erregresio-analisia gauzatu ahal da, non aldagai independenteak sartzen dira modeloan doitze ideala lortu ondoren. Sartutako lehenengo aldagaia korrelazio handiena daukana da, negatiboa edo positiboa, mendeko aldagaiarekiko, baldin eta sartzeko irizpidea betetzen den. Lehenengo aldagaia sartu denean, hurrengo hautagaia izango da ekuazioan ez dagoena eta bere korrelazio partziala handiena dena. Prozedurak amaitzen da aldagai gehiagok sartzeko irizpidea betetzen ez duenean. **Atzera pausoz pauso** (*backward stepwise*) analisia modeloan aldagai posible guztiak sartzen ditu eta baztertzen ditu beharrezkoak modelo optimo bat doitzeko, arazorik gabe. Korrelazio partzial txikienak dituen aldagaia baztertzeko kontuan hartuko den lehenengoa izango da eta baztertuko da baztertzeko irizpidea betetzen badu. Lehenengo aldagaia baztertu ondoren, prozedura errepikatzen da ekuazioaren aldagaiekin. Prozedura amaitzen da baztertzeko irizpidea betetzen ez duen ekuazioaren aldagairik ez dagoenean.

AKAIKE’s AIC eta Schwarz’s SC estatistikoak iragartzeko ahalmen onena duen doitutako modeloa hautatzen ahalbidetzen du, hurrengo estatistikoetan balio txikienak lortzen dituena (Pérez López, 2016):

$$AIC = -\frac{2l}{T} + \frac{2(k+1)}{T} \tag{7.59}$$

$$SC = -\frac{2l}{T} + \frac{(k+1)\log(T)}{T} \tag{7.60}$$

$$l = -\frac{T}{2}\left(1 + \log(2\pi) + \log\frac{e'e}{T}\right) \tag{7.61}$$

Non k modeloaren aldagai independentearen kopurua den (konstantea kontuan hartu gabe), T behaketa eskuragarrien kopurua da eta e modeloaren errorea den.

7.5.3.2. Modeloan estimatutako parametroen esangurari buruzko hipotesiak

Modeloaren estimatutako parametroak esanguratsuak izan behar dute, bai indibidualki, $\hat{\beta}_i \neq 0$ edozein i = 1,2,…,k-rako (esangura indibiduala) eta globalki $\left(\hat{\beta}_1, \hat{\beta}_2, \ldots, \hat{\beta}_k\right) \neq (0,0,\ldots,0)$ (esangura globala).

Erregresio-koefizienteen esangura indibidualerako, T_i estatistikoa (7.51 ekuazioa) erabiltzen da, Student-en t

banaketa jarraitzen duena, $T - k - 1$ askatasun-graduekin. Hipotesi nulua egiaztatzen (7.50 ekuazioa) eta konfiantza-tarteak lortzen (7.55 ekuazioa) ahalbidetzen du.

Modeloaren esangura globalera, F estatistikoa (7.46 ekuazioa) erabiltzen da, Fischer-Snedecor-en $F(k,T - k - 1)$ banaketa jarraitzen duena. Hipotesi nulua egiaztatzen ahalbidetzen du.

7.5.3.3. Ausazko erroreari buruzko hipotesiak

Erregresio anizkoitzeko modeloa formulatu zen u ausazko erroreari lotutako hipotesi klasikoak jarraituz (Pérez López, 2016).

- Errore terminoa (u aldagaia) ausazko aldagaia da, bere esperantza nulua eta kobariantza-matrizea konstantea eta diagonala (matrize eskalarra) izanik. Beste era esanda, edozein t behaketarako, u_t aldagaiaren batez bestekoa 0 da eta bariantza σ^2 ez da t-ren araberakoa. Horrez gain, $Cov(u_i, u_j) = 0$ da edozein i eta j behaketetarako, ezberdinak baldin badira haien artean, $\mathrm{Var}(u) = \sigma^2 I_k$. Orduan, ut-ren bariantza edozein t-rako konstantea izatea (ez da t-ren araberakoa), **homoszedastizitatearen hipotesia** da (**ez heteroszedastizitatea**) eta honela adieraz daiteke: $V(u|X_1,X_2,\ldots,X_k) = \sigma^2$ eta $V(y|X_1,X_2,\ldots,X_k) = \sigma^2$. Beste aldetik, $Cov(u_i, u_j) = 0$ izatea edozein i eta j-rako, ezberdinak badira haien artean, **ez auto-korrelazioaren hipotesia** da.
- Errore terminoa, u, neur ez daitekeen ausazko aldagaia, zeinek inplikatzen duen y aldagaia ausazkoa dela u-ren araberakoa baita (7.13 ekuazioa).
- Erroreen normaltasuna era kontuan hartzen da, zeinek inplikatzen duen u_t aldagaiak normalak direla edozein t-rako. Beste era batera esanda, modeloaren ausazko erroreen bektoreak banaketa normala dauka, bere batez besteko zeroa izanik, $E(u) = 0$, eta kobariantza-matrizea eskalarra da $E(uu') = \sigma^2 I$. Idatzi daiteke honako adierazpen hau: $u \rightarrow N(0, \sigma^2 I)$.
- Erroreak ez daukate datu atipikorik.

7.5.3.4. Aldagai independenteei (azaltzaileak) buruzko hipotesiak

Aldagai independenteei buruz honako hipotesi hauek kontsideratzen dira (Pérez López, 2016):

- Aldagaiak, $X_1, X_2,\ldots,X_k$, ez daude esanguratsuki erlazionatuta, hau da, ez dago beraien arteko erlazio linealik. Hipotesi honi independentziaren hipotesia deitzen zaio eta betetzen ez denean, modeloak multikolinealtasuna duela esaten da. Aldagai independentearen matrizea k heina duela ere esan daiteke.
- Aldagaiak, deterministakoak dira (ez dira aldagai estokatikoak) zeren eta haien balioa konstantea da lagin batetik eta ez daude erlazionatuta errorearekin, u, hau da, $E(u|X_1,X_2,\ldots,X_k) = 0$ (exogenitatearen hipotesia). Exogenitatearen hipotesia betetzen ez denean, endogenitatearen arazoa dago.

- Aldagaiak ez daukate behaketa edo neurtze-errorerik.
- Datu-basean ez dago ez dago behaketa eraginkorrik.

7.5.3.5. Parametroen bektoreari buruzko hipotesiak

Onartzen da parametroen bektorea konstantea da (Pérez López, 2016). Hipotesiak estimazioen egonkortasuna deboran ziurtatzen du. Hipotesi hau aztertzea kointegrazioaren teoriara eramaten du.

7.5.3.6. Forma funtzionalari buruzko hipotesiak

Bi hipotesi onartzen dira (Pérez López, 2016):

- Y eta X_1, X_2,…,X_k-ren arteko erlazioa efektiboki lineala da (linealtasunaren hipotesia).
- Ez dagoela azaltze-errorerik onartzen da, hau da, Y aldagaia azaltzeko esanguratsuak diren aldagai guztiak modelo linealaren definizioan sartuta daude. Beste era esanda, ez dago aldera batera utzitako aldagairik ezta aldagai erredundanterik.

7.5.4. Erroreen analisia

Erregresio modeloa eraiki denean, hipotesi batzuk egiaztatu behar dira, lehen azaldutakoak. Beraien artean, linealtasuna, normaltasuna, ez auto-korrelazioa eta independentzia hipotesiak egiaztatu behar dira (Pérez López, 2014). Hurrengo azpi-ataletan erakusten da nola egiaztatu ahal diren arazo hauek, eta egotekotan, ohiko soluzioak aipatzen dira.

7.5.4.1. Auto-korrelazioaren arazoa

7.19 ekuazioaren itxurako modelo lineal batean, hipotesi serie bat onartzen dira eta hein artean, u aldagaia (errorea) ausazko aldagaia da eta bere itxaropena nulua da, eta kobariantza-matrizea konstantea eta diagonala da. Beste era esanda, t behaketa guztietarako, u_t aldagaiaren batez bestekoa nulua da eta bariantza σ^2 da eta ez t-ren araberakoa. Gainera, Cov(u_i, u_j) = 0, edozein i eta j-rako, haien artean ezberdinak baldin badira. Azken hau ez auto-korrelazioaren hipotesia esaten zaio (Pérez López, 2014).

Modelo baten auto-korrelazioa analizatzeko, erroreen grafikoarekin hasten da, batez ere erroreen grafikoa (ahal bada estudentizatuak) behaketarekiko (edo denbora), non ausazko patroia edo egitura ikusi behar den. Grafikoen analisiaz aparte, kontraste batzuen bitartez egiaztatzea normala da: Durbin-Watson, Wallos, h-Durbis, Creusch-Godfrey and Cochrane-Orcutt.

Haien artean, Durbin-Watson estatistikoa da gehien erabiltzen dena eta honela definitu ahal da:

$$DW = \frac{\sum_{t=2}^{T}\left(\hat{u}_t - \hat{u}_{t-1}\right)^2}{\sum_{t=1}^{T}\hat{u}_t^2} \cong 2\cdot\left(1-\rho\right) \Rightarrow \begin{cases} DW \cong 2 & if\ \rho = 0 \\ DW \cong 0 & if\ \rho = 1 \\ DW \cong 4 & if\ \rho = -1 \end{cases} \qquad [7.62]$$

Non ρ elkarren segidako behaketen erroreen arteko perturbazioaren koefizientea da. Orokorrean ausazko perturbazioa autorregresio modeloa jarraitu ahal du. Batez besteko mugikorrarena edo beste motakoa, baina normalean 1. mailako autorregresio modeloa da gehien erabiltzen dena, eta honela definitu ahal da:

$$u_t = \rho \cdot u_{t-1} + e_t \qquad [7.63]$$

DW 0 bada, auto-korrelazio positibo perfektua dago; *DW* 2-ren inguruan badago, ez dago auto-korrelaziorik eta *DW* 4-tik hurbil badago, auto-korrelazio negatibo perfektua dago. *DW* 1,5 eta 2,5-ren artean badago esan daiteke ez dagoela auto-korrelaziorik. Hala ere, badaude taula batzuk ez auto-korrelazioaren tartea onartzeko, *k* aldagaien kopuruaren arabera.

Auto-korrelazioaren presentzia modeloan, serietan ohikoa dena, Cochrane-Orcutt-en metodoarekin ebatzi ahal da edo "*dummy*" aldagai egokia sartuz modeloan. Karratu Txikien Orokortuen Metodoa ere erabiltzen da. Hainbeste erabiltzen ez diren beste teknikak Durbin-en estimazioaren metodoa eta Prais-Winstein-en prozesura.

7.5.4.2. Heteroszedastizitatearen arazoa

Azaldu den bezala, u_t aldagaia behaketa guztietarako konstantea (eta ez *t*-ren araberakoa) izateari esaten zaio homoszedastizitatea. Hipotesi hau betetzen ez denean, eta u_t-ren bariantza konstantea ez denean, heteroszedastizitatea dago. Hipotesi hau betetzearen inportantzia Karratu Txikien Metodotik lortutako estimatzaileak ez daukate bariantza minimo inpartzialak izan arrean. Horrez gain, modeloaren aldagai bakoitzerako, bariantzaren errore bat kalkulatuko da.

Heteroszedastizitatea analizatzeko erroreen grafikoarekin hasten da normalean. Oso lagungarria da estandarizatutako erroreak iragarritako estandarizatutako balioekikozko grafikoa, lehenengo koadrantearen lehenengo diagonala doitu behar duena. Horrez gain, erroreak (ahal bada estudentizatuta) mendeko aldagaiarekiko eta aldagai independente bakoitzarekikozko grafikoak aztertu, ausazko patroiak erakutsi behar dituztenak. Errore eta aldagai independente bakoitzaren grafikoak zein aldagai den heteroszedastizitatearen erruduna identifikatzen ahalbidetzen du=. Horrez gain, heteroszedastizitatearen beste kontraste formalak gauzatu ahal dira: Goldfeld-Quant, Glesjer, Breush-Pagan, White, GARCH, ARCH and Ramsey's RESET (Pérez López, 2014).

Orokorrean, heteroszedastizitatearen arazoa ebazten da Karratu Txikien Orokorren bidez estimatuz, edo Karratu Txikien Metodo erabiliz 2 fasetan.

7.5.4.3. Multikolinealtasunaren arazoa

7.5.3.4 atalean komentatu da X_1, X_2,…,X_k aldagaiak linealki independenteak direla, hau da, ez da berain arteko erlazio zehatzik. Hipotesi hau independentziaren hipotesia da eta modeloak betetzen ez duenean, modeloak dauka multikolinealtasuna.

Multikolinealtasunaren sintoma batzuk honako hauek dira:

- Aldagai independenteen korrelazio-matrizean balio altuak moduluan

- Aldagai independenteen esangura baxua eta aldi berean, determinazio-koefiziente altua (R^2).
- Modeloaren esangura altua ($R^2 = 0$ hipotesia errefusatzen da).
- Eragina handia estimatzaileetan datu-baseetako behaketaren bat kentzen bada.
- Bariantzaren Inflazioaren Faktorea (VIF), 7.64 ekuazioan definitzen den bezala, altua bada (> 10).

$$VIF = 1 \big/ \left(1 - R_j^2\right) \qquad [7.64]$$

Non R_j^2 j aldagai independentearen R^2 den, beste aldagaien arabera.

Gehien erabiltzen diren soluzioak multikolinealtasunarako honako hauek dira (Pérez López, 2014):

- Behaketa gehiago hartu edo aldagai batzuk bihurtu (adibidez, ratioak edo diferentziak erabiliz)
- Kendu aldagai batzuk justifikazio estatistiko edo ekonomiko batekin
- Ordezkatu aldagai azaltzaile batzuk haien osagai nagusi esanguratsuen ordez (puntuazioak) eta kalkulatu erregresioa osagaiekin (erlazionatuta ez daudenak eta ez dago multikolinealtasunik).

7.5.4.4. Erroreen normaltasuna

Erregresio anizkoitzaren modeloaren hipotesirik garrantzitsuenetariko bat erroreen banaketa normala da. Nahiz eta hipotesi hau ez da beharrezkoa karratu txikien metodoaren bidez modeloaren parametroen estimatzaileak lortzeko beharrezkoa ez izan, modeloan inferentzia gauzatzeko beharrezko da.

Erroreen normaltasuna egiaztatzeko banaketa normalaren edozein kontraste-test erabil daiteke, *Khi* karratuko kontrastea edo Kolmogorov-Smirnov estatistikoa. Hala ere, datu askotarako (30 behaketa baino behiago), Shapiro-Wilks-en estatistikoa gomendatzen da, kontserbadoreagoa dela esaten dena.

Shapiro-Wilks-en estatistikoak erregresioaren erroreak probabilitatezko paper normal batean linea bat doitzen duten neurtzen du. Normaltasuna errefusatzen da doitzea txikia denean, estatistikoaren balio baxuak lortzen direnean. Estatistikoak honako adierazpen hau dauka:

$$w = \frac{1}{ns^2}\left[\sum_{j=1}^{h} a_{j,n}\left(x_{(n-j+1)} - x_{(j)}\right)\right]^2 = \frac{A^2}{ns^2} \qquad [7.65]$$

Non $ns^2 = \sum \left(x_i - \overline{x}\right)^2$,$h = n/2$ da, n bikoitia bada eta h = $(n-1)/2$, n bakoitia bada. $a_{j,n}$ koefizienteak taula batean daude eta $x_{(j)}$ laginaren balio ordenatuak dira j posizioan dagoena. Estatistiko, w, honen balioak taula batean daude, eta normaltasuna errefusatzen da, datuetarako kalkulatutako balioa dagokion balio kritikoa baino baxuagoa bada. Hala ere, p-balioaren irizpidea erabili ahal da, erroreen banaketa normalaren hipotesiaren nulua α mailarekin errefusatuz p-balioa α baino txikiagoa denean, eta onartuz beste kasuan.

Orokorrean, erroreen normaltasunaren falta balio atipikoen presentziagatik sortzen da banaketa ez simetrikoa edo beste bat sortuz. Erroreen arazo hauek agertzen dira aldagai azaltzaile inportanteak modeloan agertzen ez direnean edo linealtasuna ez dago haien espezifikazioetan. Arazo hauek ebazten badira, normaltasunaren arazoak desagertzen dira. Erroreak normalak ez direnean eta moda bat baino gehiago daukatenean, datuak lagin ezberdinetatik datoz. Arazo hori konpon daiteke fikziozko aldagaiak sartuz. Beste kasuetan, normaltasunaren faltaren soluzioa aldagaien transformazio egokia da (Pérez López, 2014).

7.5.4.5. Palanka-efektuaren analisia

Palanka-efektuaren analisiaren helburua modeloaren estimazioa eragina handia daukaten behaketak eta modeloa doitzen duten behaketa heterogeneo edo atipikoak ezagutzea da

Mahalanobis-en distantziaren bitartez ikus daiteke. Mahalanobis-en distantzia $\vec{x}_i = (x_{i1}, x_{i2}, \ldots, x_{ik})$ $i = 1,2,\ldots,n$ behaketen distantzia $\bar{x} = (\bar{x}_1, \bar{x}_2, \ldots, \bar{x}_k)$ aldagai azaltzaileen hodeiaren batez besteko puntura, non $\bar{x}_j$, $j = 1,2,\ldots,k$ j aldagaiaren datuen batez besteko balioa da. Distantzia hau honako adierazpen honen bidez definitzen da:

$$d_M^2(\vec{x}_i;\bar{x}) = (\vec{x}_i - \bar{x})S^{-1}(\vec{x}_i - \bar{x})^t \qquad [7.66]$$

Non S ($x_1,x_2,\ldots,x_k$). aldagaien bektorearen bariantza-kobariantza matrizea den.

Mahalanobis-en distantzia bi bektoreen arteko distantzia euklidearra orokortzen duen distantzia estatistikoa da, non aldagaien dispertsioa eta haien dependentzia kontuan hartzen diren. Mahalanobis-en distantziaren balio handiak puntua hodeiaren zentrotik urrun dagoela esan nahi du, eta ondorioz, eragina duen puntu posiblea da (Pérez López, 2014).

7.5.5. ANOVA, ANCOVA modeloak eta Modelo Lineal Orokortuak

7.5.5.1. Bariantza sinplearen analisia ANOVA

7.2 irudian erakusten zen aldagai kualitatiboak aldagai kuantitatibo bat iragartzeko erabiltzen zenean, Bariantzaren analisia (ANOVA) erabiltze zela. ANOVA modeloak neurtzen du aldagai independente kualitatiboen kategoriek (mailek) mendeko aldagaian sortutako mailen batez bestekoen arteko diferentzien esangura. 7.7 ekuazioa erakusten du modeloaren adierazpena.

Faktore bakarreko bariantzaren analisia (ANOVA) erabiltzen da mendeko aldagai kuantitatibo baten eta aldagai independente kualitatibo baten arteko erlazioa analizatzen denean, aldagai independentearen mailetan aztertuta (Pérez López, 2014).

Ausazko efektuak dituen modeloaren kasuan, kontsideratzen da, esperimentuan erabiltzen diren faktorearen mailen populazio totaletik G mailak ausaz aukeratzen dira. Kasu honetan, ausazko efektuko ANOVA modeloak 7.67 ekuazioaren forma dauka (Pérez López, 2014).

$$Y_{ij} = \mu_i + \varepsilon_{ij} \qquad [7.67]$$

Eta ekuazio baliokidea, $\mu_i = \mu + \beta_i$ dela kontsideratzen bada:

$$Y_{ij} = \mu_i + \beta_i + \varepsilon_{ij} \qquad [7.68]$$

Non μ konstante bat den; β_i i = 1,..., G -rako ausazko aldagai independenteak diren N(0, σ^2_β) banaketa dutenak; ε_{ij} i = 1,..., G -rako eta j = 1,..., n_i –rako ausazko aldagaiak diren N(0, σ^2_β) banaketarekin. Modelo honetan egiaztatzen da E[Y_{ij}] = μ dela eta Y_{ij}-ren bariantza, σ_Y^2 adierazita, $V[Y_{ij}] = \sigma_Y^2 = \sigma_\beta^2 + \sigma^2$ da, non σ_β^2 eta σ^2 *bariantzaren osagaiak* bezala ezagutzen direnak. Modeloa estimatuta dagoenean, egiaztatu behar dira oinarrizko hipotesiak ez daudela kontraesanetan behatutako datuekin, hainbat testen bitartez.

Interesantea da egiaztatzea batez bestekoaren berdintasunaren hipotesia onartu daitekeen, hau da, behatutako mailen batez bestekoak, esperimentua errepikatu eta gero, berdinak diren (u_1 = u_2 = ...= u_G = u). Testak hipotesi hau egiazkoa dela esaten badu, behaketa bat talde batean egotea ez dauka interesik, eta behaketa guztiak populazio beretik datozela kontsideratu ahal da. Ikuspuntu alternatibo bat, ondorio berera eramaten duena, mailen batez bestekoen arteko diferentziak txikiak diren aztertzea da. Mailen batez bestekoen arteko diferentzietarako (u_i – u_j) konfiantza-tarteak egin ahal dira beraien arteko diferentziak aztertzeko helburuarekin. Tartean 0 sartze bada, mailen batez bestekoen berdintasunaren hipotesia onartu ahal da (Pérez López, 2014).

Orokorren, faktore baten mailak aztertzean, faktorearen mailak mendeko aldagaiarekiko konfiantza-maila batez beraien artean ezberdinak diren jakiteaz gain (interesantea dena), diferentziak esanguratsuak diren jakin ondoren zein mailaren artean dago diferentzia. Hau gauzatzeko Bonferroni, Tukey's HSD; Tamhane's T2 edo Dunnett's T3 estatistikoen bitartez burutzean da (Pérez López, 2014).

Honaino, faktoreen mailak infinituak direla suposatu da, hau da, faktoreak ausazkoak dira. Kasu honetan, *ausazko efektuko modeloa* aztertu da. Hala ere, bariantzaren analisiaren modeloa efektu finkoko modeloa da, non emaitzak faktorearen maila batzuetarako bakarrik baliozkoak diren, kasu honetan aztertutakoak (faktore konstanteak), beste mailetan gertatzen dena ezberdina izan daitekeelarik. Bariantzaren analisiaren taula bera da ausazko efektuetarako eta efektu finkoetarako. Dagoen diferentzia bakarra da ausazko efektuekin, bariantzaren osagaiak kontsideratu behar direla (σ_β^2 presentziarekin) (Pérez López, 2014).

Kontsideratuko da bi faktore badaude, eta faktoreen bi mailen taldeak aztertuko den populazio handiko lagin batekoak direla. Kasu honetan, bi faktoreko ANOVA orokorra da ausazko efektuekin, eta 7.69 ekuazioaren bidez adieraz daiteke.

$$Y_{ij} = \mu_i + \beta_i + \delta_j + (\beta\delta)_{ij} + \varepsilon_{ijl} \qquad \beta\delta = \text{faktoreen arteko interakzioak} \qquad [7.69]$$

Non i = 1,..., h; j = 1,..., k; l = 1,..., t-rako, μ konstantea dela egiaztatzen den; β_i ausazko aldagai independenteak diren N(0, σ^2_β) banaketa normala banatuta; δ_j ausazko aldagai independenteak diren N(0, σ^2_β) banaketa jarraituz; $(\beta\delta)_{ij}$ ausazko aldagai independenteak diren N(0, σ^2_β) banaketa jarraituz; ε_{ijl} ausazko aldagaiak diren N(0, σ^2) banaketa jarraituz; eta β_i, δ_j, $(\beta\delta)_{ij}$, eta ε_{ijl} ausazko aldagaiak diren haietako edozein bi aldagairen artean independenteak izanik.

Bi faktoreko ANOVA modeloa nahastutako efektukoa, A faktoreak maila finkoak baditu eta B faktoreak ausazko mailak baditu, β_i efektuak konstanteak dira, δ_j efektuak ausazkoak dira eta konbinazioaren efektuak

$(\beta\delta)_{ij}$ ausazkoak dira δ_j ere ausazkoak direlako ere. Tratamendu bakoitzerako laginaren tamaina berdina suposatuz, nahastutako efektuko bi faktoreko modeloa 7.70 ekuazioarekin adieraz daiteke (Pérez López, 2014).

$$Y_{ijl} = \mu + \beta_i + \delta_j + (\beta\delta)_{ij} + \varepsilon_{ijl} \quad [7.70]$$

Non μ konstantea den; β_i konstanteak diren $\Sigma\beta i = 0$ egiaztatuz; δ_j $j = 1,..., k$-rako ausazko aldagaiak diren $N(0,\sigma^2_\beta)$ banaketa jarraituz; $(\beta\delta)_{ij}$ $i = 1, ..., h$–rako ausazko aldagaiak diren $N(0, \sigma^2_{\beta\delta}(h\text{-}1/h))$ banaketa jarraituz $\Sigma(\beta\delta)ij = 0$ murrizketarekin edozein $j = 1,..., k$-rako; ε_{ijl} $i = 1,..., h$, $j = 1,..., k$ eta $l = 1,..., t$–rako ausazko aldagaiak diren $N(0, \sigma^2)$ banaketa jarraituz; eta β_i, δ_j, $(\beta\delta)_{ij}$, eta ε_{ijl} aldagai independenteak diren haietako edozein bi aldagai hartuta.

Azken aukera efektu finkoko bi faktoreko ANOVA modeloa da eta bere ekuazioa orokorra bi faktore dituena, *A* eta *B*, efektu finkokoak, 7.71 ekuazioa da.

$$Y_{ijk} = u + A_i + B_j + AB_{ij} + E_{ijk} \quad i = 1,\ldots,t,\ j = 1,\ldots r,\ k = 1,\ldots n_{ij}\ \ u = cte \quad [7.71]$$

A_i eta B_j irudikatzen dituzte *A* eta *B* faktoreen efektuak (efektu nagusiak), eta konstanteak dira honako murrizpena honekin:

$$\sum_{i=1}^{t} A_i = \sum_{j=1}^{r} B_j = 0 \quad [7.72]$$

AB_{ij} osagaiak irudikatzen du *A* eta *B* faktoreen arteko interakzioaren efektua, eta konstantea da 7.73 ekuazioaren murrizpenarekin:

$$\sum_{i=1}^{t} (AB)_{ij} = \sum_{j=1}^{r} (AB)_{ij} = 0 \quad [7.73]$$

E_{ijk} terminoak irudikatzen du errore esperimentala, zero batez besteko ausazko normal banaketako aldagaiari degokiona eta bere bariantza σ^2 konstantea delarik edozein *k*-rako (E_{ijk} aldagaiek independenteak izan behar dute).

A, *B*, eta *C* faktoreen hiru faktoreko modelorako bere adierazpena 7.74 ekuazioa da.

$$Y_{ijkl} = u_i + \alpha_i + \beta_j + \gamma_k + (\alpha\beta)_{ij} + (\alpha\gamma)_{ik} + (\alpha\beta\gamma)_{ijk} + \delta_{ijkl}$$

$$i = 1,\ldots,t,\ j = 1,\ldots r,\ k = 1,\ldots s,\ l = 1,\ldots n_{ijk} \quad [7.74]$$

Bertan, α_i, β_j eta γ_k osagaiek *A*, *B* eta *C* faktoreen efektuak (efektu nagusiak) irudikatzen dituzte; $(\alpha\beta)_{ij}$ osagaiak *A* eta *B* faktoreen arteko interakzioaren efektua irudikatzen du; $(\alpha\gamma)_{ik}$ osagaia *A* eta *C* faktoreen arteko interakzioa irudikatzen du; $\beta\gamma)_{jk}$ osagaiak *B* eta *C* faktoreen arteko interakzioaren efektua irudikatzen du eta $(\alpha\beta\gamma)_{ijk}$ osagaiak *A*, *B* eta *C* faktoreen arteko interakzioa hirukoitza irudikatzen du. Beste aldetik, δ_{ijkl} osagaiak errore esperimentala irudikatzen du, zero batez besteko ausazko aldagai normal bati dagokiona eta bere bariantza konstantea izanik edozein *l*-rako. Aldagaiek independenteak izan behar dute. Modeloak ere osagai konstanteekin ere kontsideratu ahal dira. Hiru faktoreko modelo faktoriak batean, hiru faktoreak finkoak izan ahal dira, hiruak ausazkoak, bat ausazkoak eta bi finkoak eta azkenik, bi ausazkoak eta bat finkoa (Pérez López, 2014).

7.5.5.2. Kobariantza sinplearen analisia ANCOVA

7.5 atalean azaldu den gisa, kobariantza sinplearen analisia teknika estatistikoa da mendeko aldagai kuantitatibo baten eta aldagai independente kuantitatibo batzuen eta aldagai independente kualitatibo batzuen arteko erlazioa aztertzeko. Kobariantza sinplearen analisiaren adierazpena 7.8 ekuazioan azaltzen zen (Pérez López, 2014).

ANCOVA modelorik sinpleena faktore bat (aldagai independente kualitatiboa) eta koaldagai bat (aldagai independente kuantitatiboa) eta 7.75 ekuazioan adierazten da.

$$Y_{ij} = u + A_i + \beta X_{ij} + E_{ij} \quad i = 1,\ldots,t,\ j = 1,\ldots n_i \qquad [7.75]$$

Non A faktore finkoa den eta X_{ij} koaldagaia den. X_{ij} ez da ausazko aldagaia. Errorea, E_{ij}, ausazko aldagaia da, banaketa normala, homoszedastizitatea, independentzia eta esperantza nuluzko hipotesiekin. Errorearen aldagaiek, E_{ij}, N(0, σ^2) banaketa normal dute eta independenteak dira. *A* faktore finkoa denez, bere mailek bete behar dute 7.76 ekuazioaren baldintza:

$$\sum_{i=1}^{t} n_i A_i = 0 \qquad [7.76]$$

$n_i = n$ bada edozein $i = 1,\ldots, t$,-rako, modeloa orekatuta dago.

Bi faktore eta koaldagai bat dituen modelorako, modeloaren ekuazioa honako hau da:

$$Y_{ij} = u + A_i + B_j + \beta X_{ij} + E_{ij} \quad i = 1,\ldots,t,\ j = 1,\ldots n_i \qquad [7.77]$$

Non A_i eta B_j faktore finkoak diren eta X_{ij} koaldagaia den, ausazko aldagaia ez dena. Errorea, E_{ij}, ausazko aldagaia da, banaketa normala, homoszedastizitatea, independentzia eta esperantza nuluzko hipotesiekin. Errorearen aldagaiek, E_{ij}, N(0, σ^2) banaketa normal dute eta independenteak dira. *A* eta *B* faktoreak finkoak direnez, haien mailek baldintza hau bete behar dute:

$$\sum_{i=1}^{t} A_i = \sum_{j=1}^{n} B_i = 0 \qquad [7.78]$$

$n_i = n$ bada edozein $i = 1,\ldots, t$,-rako, modeloa orekatuta dago.

Bi faktore eta bi koaldagai dituen modelorako, modeloaren ekuazioa honako hau da (Pérez López, 2014).

$$Y_{ij} = u + A_i + B_j + \gamma X_{ij} + \delta W_{ij} + E_{ij} \quad i = 1,\ldots,t,\ j = 1,\ldots n_i \qquad [7.79]$$

Non A_i eta B_j faktore finkoak dira eta X_{ij} eta W_{ij} koaldagaia diren, ausazko aldagaiak ez direnak. , Errorea, E_{ij}, ausazko aldagaia da, banaketa normala, homoszedastizitatea, independentzia eta esperantza nuluzko hipotesiekin. Errorearen aldagaiek, E_{ij}, N(0, σ^2) banaketa normal dute eta independenteak dira. A eta B faktoreak finkoak direnez, haien mailek baldintza hau bete behar dute:

$$\sum_{i=1}^{t} A_i = \sum_{j=1}^{n} B_i = 0 \qquad [7.80]$$

$n_i = n$ bada edozein $i = 1,\ldots, t$,-rako, modeloa orekatuta dago.

7.5.5.3. Erregresio Lineal Orokor Anizkoitzeko (GLM) modeloak

Erregresio Lineal Orokor Anizkoitzeko (GLM) modeloak erregresio linealeko modelo orokorrena da, erregresio lineal anizkoitzeko modeloak aldagai kuantitatiboekin eta erregresio anizkoitzeko modeloak aldagai kuantitatibo eta kualitatiboekin batera barne hartzen dituena, eta beraz, bariantzaren analisia (ANOVA) eta kobariantzaren analisia (ANCOVA) hartzen ditu barne (Pérez López, 2014) Modelo hau software estatistiko gehienen bidez garatu ahal da.

7.6. Ondorioak

Kapituluak erakutsi zeuen azaldutako edo iragarritako aldagaiak aukeratzeko arrazoiak portaera modeloetarako eta modelo-mota haiek iragartzeko. International Roughness Index (IRI) eta SCRIM koefizientea (*SC*) aukeratu ziren iragarritako aldagaiak bezala erregulartasuna eta labainketarekiko erresistentzia iragartzeko, hurrenez hurren. Beste bide-zoruko parametroek ez daukate hainbeste datu historikorik eta beraz, baztertu ziren.

Modelo posible guztiak aztertu ostean, modelo determinista eta enpirikoa, erregresio analisien bidezkoa, aukeratu zen. Aukera onena kontsideratu zen datu eskuragarria eta bide-administrazioaren helburua kontuan hartuta. Faktore asko aldagai azaltzaile bezala sartzen ahalbidetzen du eta baztertzen ditu aurreikuspenean esanguratsuak ez direnak. Gainera, beste bide-administrazioek ere erabil dezakete. IRI eta SCRIM koefizientearen modeloetan sartutako aldagai azaltzaileak 8. eta 9. kapituluetan deskribatzen dira, garatutako modeloekin batera.

Horrez gain, erregresio analisiaren metodologia deskribatu zen datuen analisien tekniken aukera bat bezala. Espezifikoki, erregresio lineal anizkoitzeko teknika sakontasunez deskribatzen da, Karratu Minimoen teknikaren metodologian zentratuta, bere propietateak eta onartutako hipotesiak identifikatuz. Modeloa garatu ondoren, onartutako hipotesi horiek egiaztatu behar dira. Hipotesi horiek azaltzen dira, nola egiazta daitezkeen eta ez betetzekotan gauzatu ahal diren soluzioak ere.

III. ATALA. EMAITZAK: PROPOSATUTAKO MODELOAK

8. kapitulua. IRI portaera-modeloak Bizkaiko errepideetarako

8.1. Sarrera

Kapitulu honek Bizkaiko errepide-sarerako garatutako IRI-a iragartzeko portaera-modeloak azaltzen ditu. Lehenengo eta behin, IRI-en progresioa eragiten duten faktoreak komentatzen dira, beste ikertzaileek, erakundeek eta bide-administrazioek garatutako beste IRI modeloak kontsideratuz. Orduan, komentatutako aldagaiak eta Bizkaiko Foru Aldundiko bide-zoruak kudeatzeko sistemako datu eskuragarriak kontuan hartuz, erregulartasunaren eboluzioa azterketan sartu ahal diren bide-zoru sekzioak adierazten dira.

Orduan, errepideak analizatzen dira kalkuluan sartu ahal diren bide-zoru tartean lortzeko eta modelo ezberdinak proposatzen dira bide-zoruaren egituraren arabera. Modelo bakoitzerako, sartzen diren aldagai azaltzaileak komentatuko dira. Hurrengo pausoan, datu eskuragarriak aztertu eta gero, erregresio-analisiaren bidez, hondatze-modeloak sortzen dira. Sartutako aldagai azaltzaile guztiak ez dira erabiltzen azken modeloan, mendeko aldagaian eraginik ez dutelako. Modeloetan aldagai azaltzaileen sarrera, edo ez, analisi estatistiko bat gauzatuz egiten da, 7.5 atalean aurkeztutako metodologiaren arabera.

Azkenik, modelo bakoitzaren zehaztasuna komentatzen da, batez ere determinazio-koefizientearen arabera, R^2, eta beste parametro estatistikoen arabera, hala nola, F testa eta Student-en t testa aldagai independente bakoitzerako

8.2. Erregulartasunaren eboluzioan eragina duten faktoreak

Atal honetan, modelo deterministakoetan erabilitako faktoreak, bai enpirikoetan bai mekanizista-enpirikoetan, 5.3.1 eta 5.3.2 ataletan komentatutakoak eta beste batzuk ere, berrikusten eta iruzkintzen dira. Faktoreak berrikusi ondoren, Bizkaiko errepide-sarerako hondatze-modeloetan sartzeko aukeratutako faktoreak proposatzen dira hurrengo ataletan.

Orokorrean, erregulartasunari buruzko modeloek erabiltzen dituzte honako faktore hauek: bide-zoruko akatsak, lekuaren baldintzak, egoera klimatikoak, egiturazko parametroak, adina eta trafiko bolumenak.

Lehenago azaldu den gisa, Munduko Bankuak (*World Bank*) erregulartasunen progresiorako proposatutako modeloak, HDM-III eta HDM-4, asko erabiltzen dira haien irmotasun eta kalibrazio-koefizienteen bidez hainbat klimatarako egokitzearen posibilitateagatik. Nahiz eta deterministakoak eta enpirikoak bezala sailkatu, haiek garatzean, Paterson (1987)-ek HDM-III modeloan egitaratutako ikuspuntu enpirikoa jarraitu zuen. Informazio mekanizista eta enpirikotik bide-zoruaren propietate bakoitzean eragina zuten oinarrizko aldagaiak identifikatu ziren eta teknika estatistika batzuen bitartez haien eragina eta inpaktua konbinatu ziren. Beraz, ikuspuntu mekanizistaren oinarri esperimentalak eta teorikoak eta egoera errealetako bertako portaera konbinatzen zituen modeloak garatu ziren. **Batez ere egituratutako modelo enpirikoak** bezala definitzen dira.

Erregulartasunen progresiorako Paterson (1987)-ek proposatutako modeloan bost parametro sartzen ziren (5.8 ekuazioa): egiturazko deformazioa, gurpil-arrastoak, pitzadurak, batzeak eta ingurumen-faktoreak. Gainera, beste balioak behar ziren, hala nola, azken errehabilitazio- edo mantentze-lanetik igarotako denbora, urteetan; igarotako kargak, urteko ardatz estandar baliokideen kopuruaren (*YE4*) bidez; eta erregulartasuna aztertze-urtearen hasieran (IRI_a). Faktore nagusia *SNCK* da, zeinek geruza bituminosoetan dauden pitzaduraren bidezko bide-zoruaren erresistentziaren murrizpena kontuan hartzen du. Horrez gain, kalibrazio-faktoreak erabiltzen dira ingurumen-ezaugarriak sartzeko.

Munduko Bankuko hurrengo erregulartasunaren eboluzioaren modeloa HDM-4-an sartu zen (Odoki and Kerali, 2000; Morosiuk *et al.*, 2004). HDM-III modeloan oinarritzen da eta aipatutako bost faktoreak erabiltzen ditu. Batxearen faktorea da aldatu duen bakarra. Faktoreek elkarri eta erregulartasunaren eboluzioari eragiten diote 5.3 irudian erakusten den bezala. Modeloa beste gehikuntzako modeloa da, bost parametroen kontribuzioa gehitzen duena, 5.13 ekuazioan erakusten den bezala. Aldagaiak 5.3.1.1.5 atalean deskribatzen dira. Bide-zoruko erresistentziarako, HDM-4 modeloak erabiltzen du doitutako egiturazko zenbakia (*SNP*) HDM-III-ren eraldatutako egiturazko zenbakiaren ordez (*SN*). Nahiz eta 5.13 ekuazioan zehazki ez adierazi, bide-zoruaren adina (azken mantentze- edo errehabilitazio-lanetik), eta trafiko kargak (urteko ardatz estandar baliokideen kopurua) erabiltzen dira ΔRI_s-en kalkuluan, 5.16 ekuazioan erakusten den bezala. Horrez gain, nahiz eta gehikuntzako modeloa izan, urtearen hasierako IRI-ak IRI-en handitzean eta ingurumen-osagaian eragina dauka (5.27 ekuazioa).

COST Action 324-ren barruan (European Commission, 1997), 5.3.1.2 atalean gehiago deskribatzen dena, Europar Batasunean dauden hondatze-modeloen bilduma gauzatu zen. Erregulartasunaren eboluziorako, 5 modelo identifikatu ziren parte hartzen zuten Europako bide-administrazioen artean. Modelo hauek labur komentatzen dira:

- Danimarkan, Bump Integrator aparatuaren bidezko datuak erabiltzen, proposatutako modeloa (5.30 ekuazioa), bide-zoruaren adina baino ez du erabiltzen aldagai independente bezala eta materialaren ezaugarriak koefizienteen bidez sartzen dira.

- Finlandian, modeloak proposatzen dira hurrengo urterako erregulartasuna kalkulatzeko hormigoi bituminosozko eta hotzean egindako geruza berrietarako eta urtean neurtutako erregulartasuna (IRI) baino ez dute erabiltzen (5.31 eta 5.32 ekuazioak).

- Hungarian proposatutako modeloek errodadura geruzaren adina edo trafiko bolumenaren (ibilgailu arinetan azalduta) araberakoak dira, modelo esponentziala jarraituz (5.33 eta 5.34 ekuazioak.

- Suedian erabilitako modeloek etorkizuneko IRI-a kalkulatzen dute bide-zoruaren adinaren (azken neurketatik igarotako urtean eta adina totala), ingurumen-faktorearen (izozte-indizea), egiturazko erresistentziaren (deflexioak eta geruza bituminosoen lodiera) eta ezaugarri geometrikoen (errepidearen zabalera) arabera (5.35 ekuazioa).

COST Action 324-ren ondorioz, PARIS (Performance Analysis of Road InfraStructure) programa gauzatu zen Europako inferentzia espazioan bide-zoruko portaera-modelo sendoak garatzeko bide-zoru malguak eta erdi-zurrunetarako (European Communities, 1999). IRI-en eboluzioa aztertzean, denborarekiko progresio linealak doitzen zituen sekzio gehienak, eta beraz, erlazio lineala ezarri zen IRI-en urteko aldakuntza modelizatzeko, 5.36 ekuazioan ikusten den bezala. Test-sekzioetan IRI aldakuntza urteko oso txikia izan zela, sekzioen %90ak 0,1 m/km baino txikiagoko balio bat zeukan urteko IRI aldakuntzarako. IRI-en aldakuntza baztergarria izan ahal zen modeloaren malda ez zen estatistikoki zerotik ezberdina. IRI-en aldakuntza motelak ere Paterson (1987)-ek aipatu zituen, trafiko altuko errepideetan 0,1 m/km urteko eta trafiko gutxiko errepideetan 0,2 m/km urteko aldakuntzak adieraziz.

NordFoU proiektua herrialde nordikoek gartu zituzten, beste helburuez gain, bide-zoruko portaera-modeloak garatzeko helburuarekin (Busch *et al*., 2010). Existitzen ziren hondatze-modeloak bildu ziren. Herrialde bakoitzeko modeloaren faktore nagusiak jarraian laburtzen dira:

- Danimarkan, mantentze-lanik gabeko errepideetako IRI progresiorako bigarren graduko modeloa erabiltzen zen, bide-zoruaren adinaren araberakoa baino ez zena, COST Action 324 programan bildu zen bezala (European Commission, 1997). Errehabilitazioaren osteko erregulartasunaren hobekuntzarako modeloa ere erabiltzen zen, oraingo bide-zoruaren erregulartasunaren araberakoa.

- Norvegian, iragarritako hazkuntza tasa maximoa, minimoa eta batez bestekoa azaltzen duen taula erabiltzen da, Eguneko Batez Besteko Intentsitatearen (AADT) arabera (5.12 taula)

NordFoU proiektuaren bigarren fase batean, HDM-4 modeloa aukeratu zen bide-zoruen portaera modelizatzeko herrialde nordikoetan. HDM-4ko kalibrazio-faktoreak kalibratu ziren baldintza nordikoetara kalibrazio-faktore nagusiak kalibratuz. 5.13 taulak ematen ditu kalibrazio-faktoreak Suedia, Norvegia eta Danimarkarako trafiko bolumen arabera.

Mississippi Department of Transportation (MDOT)-eko datu-basearekin, George (2000) bi IRI portaera-modeloak garatu zituen. Lehenengoa eraiki berriko errepideentzat zen (8.1 – 8.4 ekuazioak) eta bigarrenak beste geruza berri bat daukatenentzat (8.5 ekuazioa).

$$IRI = \left[2{,}4169 + Age^{0,2533} \cdot \left(1 + CESAL^{0,2575}\right)\right] \cdot MSN^{-0,7753} \quad [8.1]$$

$$MSN = SN + SN_{SG} \quad [8.2]$$

$$SN = a_1 \cdot D_1 + a_2 \cdot D_2 \cdot m_2 + a_3 \cdot D_3 \cdot m_3 \quad [8.3]$$

$$SN_{SG} = 3{,}51 \cdot \log_{10} CBR - 0{,}85 \cdot (\log_{10} CBR) - 1{,}43 \quad [8.4]$$

$$IRI = \left[3{,}5746 + Age^{0,1701} \cdot \left(1 + CESAL^{0,6972}\right)\right] \cdot MSN^{-0,3438} \cdot TOPTHK^{-0,1313} \cdot RES^{-0,1056} \quad [8.5]$$

Non:

IRI: erregulartasuna da (m/km)

Age: bide-zoruaren adina da eraiki zenetik (urteak)

CESAL: bide-zoruan aplikatutako 18 kip-eko Ardatz Sinpleko Karga Baliokide ren (*Equivalent Single Axle Load, ESAL*) kopuru metatua (trafiko astun gehiago duen erreia) (milioitan)

MSN: eraldatutako egiturazko zenbakia da (*modified structural number*)

SN: egiturazko zenbakia da (*structural number*)

a_i: bide-zoruaren *i* geruzaren koefizientea da

m_i: drainatze koefiziente da.

D_i: bide-zoruko i geruzaren lodiera

CBR: California Bearing Ratio

TOPTHK: zabaldutako azken errodadura geruzaren lodiera (mm)

RES: errodadura geruza berriaren mota

Dubaiko errepideetarako Al-Suleiman and Shiyab (2003)-ek garatu zituzten bi IRI erregresio modeloak, bat errei motelerako (8.6 ekuazioa) eta bestea errei azkarrerako (8.7 ekuazioa) erlazio esponentzial baten bidez eta bide-zoruaren adina aldagai azaltzaile bakarra izanik:

$$IRI_s = 0{,}796 \cdot e^{0{,}0539 \cdot Age} \qquad [8.6]$$

$$IRI_f = 0{,}824 \cdot e^{0{,}0359 \cdot Age} \qquad [8.7]$$

Non

IRI_f : International Roughness Index (m/km) errei motelean

IRI_s: International Roughness Index (m/km) errei azkarrean

Age: bide-zoruaren adina eraiki zenetik edo azken geruza zabaldu zenetik (urteak)

Choi *et al*. (2004)-ek proposatu zituzten erregresio-analisiko eta Sare Neural Artifizialeko modeloak erregulartasunaren eboluziorako LTPP datu-basean oinarrituta. Erregresio modeloa 8.8 ekuazioan erakusten da:

$$IRI = 4{,}08 - 0{,}616 \cdot SN + 4{,}51 \cdot AC + 7{,}79 \cdot P_{200} \cdot AC - 3{,}78 \cdot P_{200} + 0{,}709 \cdot CESAL - 0{,}489 \cdot Thick \qquad [8.8]$$

Non

IRI: International Roughness Index (in./mila)

SN: egiturazko zenbakia (*structural number*)

P_{200}: 200 zenbakiko bahetik batzen den portzentajea

Thick: errodadura geruzaren lodiera

AC: betunaren edukiera

CESAL: *ESAL* metatuak

Brasilgo ipar-ekialdeko errepideetarako Alburquerque and Núñez (2011)-ek bi IRI portaera-modeloak proposatu zituzten. Modelo hauek IRI-a iragartzen dute Ardatz Sinpleko Karga Baliokidearen (*Equivalent Single Axle Load, ESAL*), SN-ren (egiturazko zenbakia, *Structural Number*) eta prezipitazioen arabera.

Owolabi *et al*. (2012)-ek proposatu zituzten hondatze-modeloak Nigeriako bide-zoru malguko errepideetarako pausoz pauso hautaketa funtzioaren erregresio-analisia burutuz eta 8.9 ekuazioa lortu zuten:

$$IRI = 1{,}441 + 0{,}073 \cdot C_{long} - 0{,}336 \cdot P_a + 0{,}029 \cdot R \qquad [8.9]$$

Non

C_{long}: luzetarako pitzaduren larritasuna (luzetarako pitzaduraren luzeraren araberakoa)

P_a: konponketen kopurua

R: gurpil-arrastoaren larritasuna (gurpil-arrastoaren sakontasunaren araberakoa)

Louisianako estatuaren bide-zoru malguetarako hormigoi bituminosozko geruza berriak zabaldu diren errepideen IRI-a modelizatzeko, Khattak *et al.* (2014)-ek erregresio-modelo bat proposatu zuen (8.10 eta 8.11 ekuazioak):

$$\ln(IRI) = -0{,}902 - 0{,}2798 \cdot \left(\frac{1}{F_n}\right) + 0{,}12078 \cdot \left(\frac{\ln(CESAL)}{T_o}\right) + 2{,}66 \cdot 10^{-4} \cdot TI + 9{,}19 \cdot 10^{-8} \cdot CPI \cdot t + \Delta \quad [8.10]$$

$$\Delta = 4{,}388 + 0{,}723 \cdot \ln(SD_0) + 0{,}513 \cdot \ln(IRI_{pp}) \quad [8.11]$$

Non:

F_n: sailkapen funtzionala

CESAL ESAL metatuak

T_o: errehabilitazioaren lodiera totala

TI: tenperatura indizea (°C·days)

t: tratamenduaren adina (urteak)

CPI: Cumulative Precipitation Index-a metatua (cm-days)

SD_o: hasierako IRI-en desbideratze estandarra tratamenduaren ostean urte bakoitzean tratamenduaren bizitza-epean (0,99 m/km) azterlan honetan.

IRI_{pp} iragarritzako IRI aurreko urteetarako

Texas Department of Transportation-eko datuekin, Dalla Rossa *et al*. (2017)-ek garatu zuen modelo bat (5.37 ekuazioan erakutsita), hurrengo azaltzaileekin: hasierako IRI (eraiki edo konpondu ondoren) eta adina, biak zuzen sartuta; klima, zelaigunea, tratamendu-mota, bide-zoru mota, trafikoko karga eta sistema funtzionala (hiri barnekoa edo hiri artekoa) kalibrazio-koefizienteen bidez sartuta.

Alaswadko *et al.* (2017)-ek garatu zituzten erregulartasuna iragartzeko modeloak gainazaleko tratamenduak dituzten errepideetarako Victoria (Australia) hierarkizazioko maila anitzeko modeloak, zeinek sekzio berearen denbora-serieen arteko korrelazioa kontuan hartzen duena eta ez behatutako efektuen eragina harrapatzen duena (5.39 ekuazioa). Erabilitako aldagai azaltzaileak dira honako hauek: bide-zoruaren adina, trafikoko karga, bide-zoruaren hasierako erresistentzia eta zelaigunearen espantsio potentziala.

Abdelaziz *et al*. (2018)-ek bi IRI portaera-modeloak konparatu zituen, bat erregresio-analisiaren bidez garatuta, eta bestea Sare Neural Artifizialen bidez, Erregresio-modeloari dagokionez, proposatutako ekuazioa honako hau da:

$$IRI = IRI_0 + 0{,}014479 \cdot Age + 0{,}00382 \cdot FC_{all} + 0{,}00053 \cdot TC_{all} + 0{,}08941 \cdot SDRUT \quad [8.12]$$

Non

IRI_0: hasierako IRI-a (m/km)

Age: eraikuntza edo geruza berria zabaldu denetik adina, bai errepide berrietan bai aurretik konpondutakoetan (urteak)

FC_{all}: larritasun mota guztietako fatigako pitzadurak (gurpilaren bidean dagoen portzentajea, %)

TC_{all}: larritasun mota guztietako zeharkako pitzaduren luzera (m/km)

SDRUT: gurpil-arrastoaren desbideratze estandarra (mm)

Modelo mekanizista-enpirikoei dagokienez, *Mechanistic-Empirical Pavement Design Guide* (MEPDG) (AASHTO, 2008; 2010; 2015) gidan sartutakoek ikerketa handiko arreta erakartzen ari dira. Bide-zoru malguetako hormigoi bituminoso berrietan eta hormigoi bituminosozko geruzak zabalduz konpondu diren bide-zoru malguetarako IRI-a iragartzeko honako azaltzaile hauek behar dira (5.40 ekuazioa): eraiki osteko hasierako IRI-a, fatigako pitzaduren azalera, zeharkako pitzaduren luzera, gurpil-arrastoaren batez besteko sakonera eta lekuko faktore bat (5.41 ekuazioa), elementu hauen araberakoa dena: bide-zoruaren adina, zelaigunearen plastikotasun indizearen portzentajea, urteko batez besteko izozte-indizea, batez besteko prezipitazioa eta 0,02 eta 0,075 mm-ko bahetik pasatzen den zelaigunearen portzentajeak. Gurpil-arrastoaren sakonera kalkulatzeko, deformazio plastikoa metatuaren ratio behar da, laborategiko entsegu batean neurtzen dena. Pitzaduren datuak lortzeko materialaren datu zehatzak behar dira: hormigoi bituminosoaren modulu dinamikoa konpresiopean neurtuta, betunaren benetako kopurura bolumenean, nahastean dagoen aire baoen portzentajea, etab.

Ikusi den bezala, IRI portaera modelizatzeko proposatutako modelo deterministakoetan hainbat aldagai azaltzaile sartu dira. Aldagai hauek talde batzuetan batu ahal dira: adina, hasierako IRI-a, akatsak, klima-faktoreak, zelaigune edo lurzoruaren parametroak, trafikoa, egiturazko parametroak eta beste faktoreak. 8.1 taulak aurkezten du zein faktore-talde erabiltzen duen atal honetan deskribatutako modelo bakoitzak.

8.1 taula. Literaturan aurkitutako modeloetan kontuan hartzen diren aldagai independenteen laburpena. Iturria: egilea.

Modeloa	Adina	Hasierako IRI-a	Akatsak	Klima	Zelaigunearen parametroak	Trafikoa	Egiturazko parametroak	Beste parametroak	Doitzearen kalitatea
HDM-III (Paterson, 1987)	Bai	Bai	Gurpil-arrastoa, batxeak, pitzadurak	Bai	-	Bai	SNCK		
HDM-IV (Odoki and Kerali, 2000, Morosiuk et al., 2004)	Bai	Bai	Gurpil-arrastoa, batxeak, pitzadurak	Bai	-	Bai	SNP		
Danimarka (European Commission, 1997; Saba, 2006)	Bai						Materialaren ezaugarriak		
Finlandia (European Commision, 1997)		Bai							
Hungaria (European Commision, 1997)	Bai					Bai			
Suedia (European Commission, 1997)	Bai			Bai	-		D900, th1	Errepidearen zabalera	
PARIS proiektua (European Communities, 1999)	Bai								
Danimarka (Saba, 2006)		Bai							
Norvegia (Sabas, 2006)						Bai			
NordFoU proiektua (Skar et al., 2014)	Bai	Bai	Gurpil-arrastoa, batxeak, pitzadurak	Bai	-	Bai	SNP		
MDOT (George, 2000) new pavement	Bai					Bai	SN		N = 690, R^2 = 0,35
MDOT (George, 2000) overlaid pavement	Bai					Bai	SN, lodiera		N = 4109, R^2 = 0,48
Dubai (Al-Suleiman and Shiyab, 2003)	Bai								N = 440, R^2 = 0,61-0,80
Choi et al. (2004)					P200	Bai	SN, AC, lodiera		R^2 = 0,714
Brasil (Alburqueque and Núñez, 2011)	-	-	-	Bai		Bai	SN		N = 18-27, R^2 = 0,94 - 0,87
Nigeria (Owolabi et al. 2012)			Gurpil-arrastoa, batxeak, luzetarako pitzadurak						R^2 = 0,78
Louisiana (Khattak et al. 2014)	Bai	Bai	-	Prezipitazioa, tenperatura	-	Bai	Lodiera		N = 623, R^2 = 0,87
Texas (Dallar Rossa et al. 2017)	Bai	Bai		Bai	Bai	Bai	Tratamendu mota, bide-zoru mota	Errepide-sare mota	
Australia (Alaswadko et al. 2017)	Bai				Bai	Bai	SNC0		R^2 = 0,60
MEPDG (AASHTO, 2008; 2015)	Bai	Bai	Gurpil arrastoa, fatigako eta luzetarako pitzadurak	Prezipitazioa, izozte-indizea	Bai		Materialaren ezaugarriak		N = 2439, R^2 = 0,57

8.3. IRI-a iragartzeko aukeratutako bide-zoruko sekzioak eta errepideak eta analisi-metodologia

8.3.1. IRI iragartzeko aukeratutako bide-zoruko sekzioak eta errepideak

Aurreko ataletan ikusi den gisa, faktore-talde batzuk IRI-a iragartzeko modeloetan gehien erabiltzen direnak dira. 8.1 taularen arabera, gehien erabiltzen diren faktoreak honako hauek dira: **bide-zoruaren adina** (eraiki zenetik edo azken mantentze- edo errehabilitazio-lanak gauzatu zenetik igarotako urteak), **trafiko bolumen**ekin erlazionatuta daudenak eta **egituraren parametro**ekin erlazionatuta daudenak, zeintzuk bide-zoruaren sekzioan dauden materialen eta haien ezaugarrien informazioa proportzionatzen dituztenak.

6. kapituluan azaldu zen gisa, errepide bakoitzeko bide-zoruaren sekzioaren informazioa BKSn proiektuen bidez sartzen da. Proiektu horiek errepidean gauzatutako lanei buruzko informazioa daukate eta lan horiek **trazatu berria** edo **mantentze- edo errehabilitazio-lana** bezala sailkatu ahal dira.

Proiektuaren Artxiboan deskribatutako bide-zoruaren sekzioa trazatu berria bezala sailkatuta badago, hasierako bide-zoruaren egitura ezaguna da, geruza guztien deskribapenarekin (materialak eta lodiera) eta zerbitzuan jarri zen data zehatza.

Beste aldetik, bide-zoruaren sekzioa mantentze- edo errehabilitazio-lan bezala sailkatuta badago, aurreko bide-zoruaren egitura kontsideratu behar da. Batzuetan, bide-zoruaren sekzioa aurretik hasierako bide-zorua sailkatu bazen, dagoen egitura ezagutzen da eta tartearen mantentze- eta errehabilitazio-historia aztertu eta analizatu ahal da. Hala eta guztiz ere, tarte hori hasierako bide-zorua bezala sailkatu ez batzen, azken geruzak baino ez dira ezagutzen, errodadura geruza(k), behean geratzen direnak ezezagunak izanik. Batzuetan, laginak ateratzen dira bide-zoru osoa ezagutzeko helburuarekin. Hala ere, nahiz eta ondorio batzuk atera daitezkeen bertako behaketetatik, hartutako laginek errepidean puntu bat (zeharkako sekzioa) baino ez dute adierazten, eta bide-zoru egitura hori noraino zabaltzen den ezin da jakin.

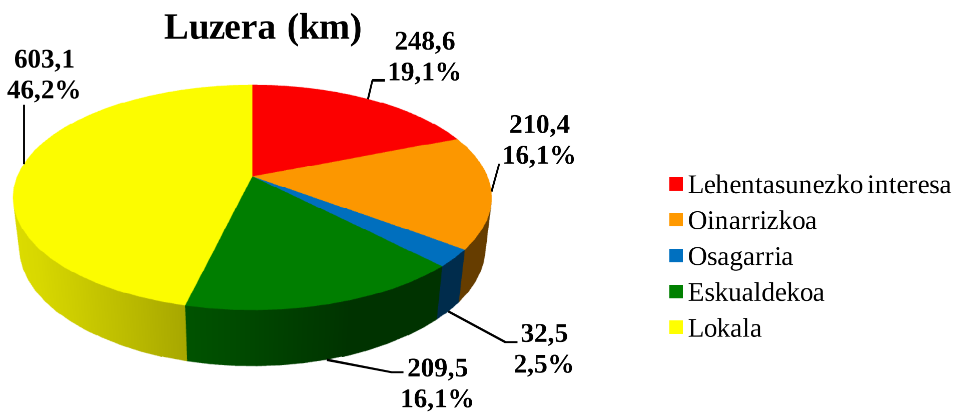
Gestivían (BFAko BKS) errepide zehatz baten Bide-zoru Egitura artxiboa (6.5.2.1 atalean gehiago deskribatuta) lortzen denean, errepidea hainbat tartetan zatituta dago ezagutzen den bide-zoruaren egituraren arabera (eta galtzada eta erreien kopuruaren arabera), 6.19 taulan aurkezten den bezala. Mota honetako artxiboetan, hasierako bide-zoruaren data daukaten tarteak bide-zoruaren egitura osoa ezagutzen diren tartean dira. Hau taularen beste zutabeetan agertzen diren datuekin ere ondorioztatu ahal da.

Hori dela eta, bide-zoruaren egituran dauden materialen garrantzia dela eta, haien lodiera eta materialak (modelo deterministakoetan ikusi ahal dena), logikoa dirudi bide-zoruan dauden geruza guztiak ezagutzen diren tarteak baino ez aztertzea. Ideia hau beste bide-agentziek ere jarraitu dute IRI-en progresioa eta beste parametroak modelizatzeko. Bide-zoruen hondatzea modelizatzearen lehenengo saiakeretan, 50eko hamarkadetako azken urteetan AASHO Road Testen gauzatutakoak (AASHO, 1962), guztiz ezagunak ziren bide-zoru egiturak neurtzen eta analizatzen dira. Geroago, *Long-Term Pavement Performance* (LTPP) programak Iparramerikan zehar (EEBBB eta Kanada barne) bide-zoruak aztertzen ditu eta datu asko daude egiturari buruz (materialak eta geruzen lodierak baino askoz gehiago), eta *Mechanistic-Empirical Pavement*

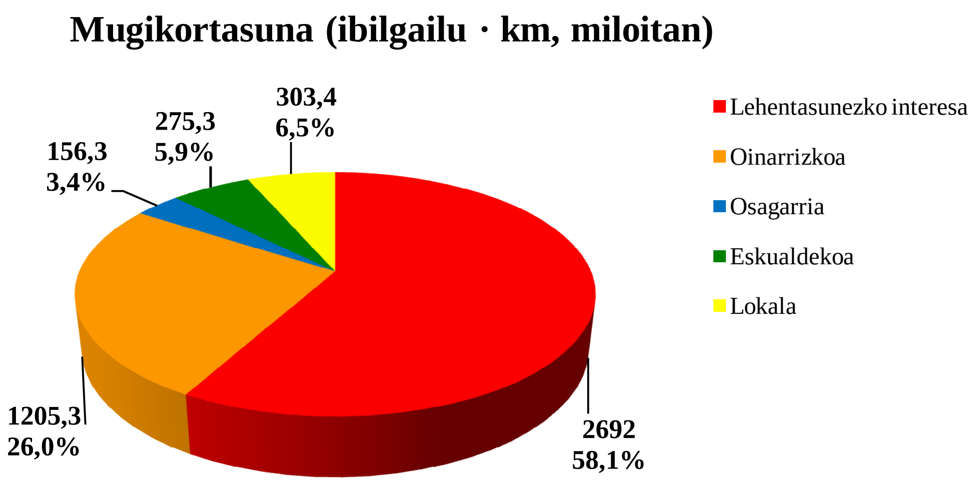
Design Guide (MEPDG) (AASHTO, 2008; 2015) gida garatzen ahalbidetu zuen, nahiz eta modelo horiek kalibratu izan behar tokiko egoeretara zeren eta globalki garatu zirelako (Wasem and Yuang, 2013). Horrez gain, Paterson (1987)-ek, lehenengo HDM-III-ko modeloak garatu zituenean, adierazi zuen erregulartasunaren progresioan denboran zehar fenomeno konplexua zela eta akats konposatu batzuen konbinazioan oinarritzen zen eta faktore hauen araberakoa zen: trafikoak eragindako deformazioa eta gurpil-arrastoaren sakoneraren bariantza, pitzadurek, batxeek eta konponketek sortutako gainazaleko akatsak eta adina eta ingurumen-faktoreen konbinazioa. Trafiko kargek tentsio-mailak sortzen dituzte bide-zoruaren geruzetan materialen gogortasuna eta lodieraren arabera.

Hori dela eta, guzti ezagunak diren bide-zoruen sekzioak baino ez dira sartuko IRI-en progresioaren kalkuluan. Komentatu den gisa, mota honetako bide-zoruek honelako informazioa daukate: bide-zoruaren egitura osoa, zerbitzuan jarri zen momentua (lanak amaitu ziren unea baino garrantzitsuagoa dena, laneko trafikoa baztergarria izan daitekeelako). Kontrako ikuspuntutik, ez da logikoa analizatzea, adibidez, bide-zoruaren sekzioaren azken 5 cm baino ezagutzen ez diren bide-zoru sekzioak. Behean dauden materialak guztiz ezezagunak dira, eta, nahiz eta modeloa kalkulatzeko datu berdinak sartuko liratekeen, bide-zoru mota oso ezberdinak badira, guztiz ezberdinak diren IRI progresioak izango lituzkete.

Datu-baseko bide-zoruaren sekzio osoa ezaguna den bide-zoru tarteak baino kontuan ez hartzearen erabakia errepide-maila lokaleko errepideak baztertzera behartzen du. BKSn datuak sartzean, 6. kapituluan deskribatutako xehetasunarekin sartu ziren lehentasuneko intereseko (gorria), oinarrizko (laranja), osagarriko (urdina) eta eskualdeko (berdea) errepide-mailako proiektuak (6.3 taula). Sare lokaleko errepideak ez ziren hain zehatz aztertu haien trafiko bolumenen eta haien garrantziagatik errepide-sare osoko funtzionamenduan. Errepide-sare lokaleko errepideak Bizkaiko Foru Aldundiak kudeatzen duen sarearen luzera totalaren % 46,2a izan (8.1 irudia), sare honen bidezko mugikortasuna totalaren % 6,5a baino ez da (8.2 irudia) (Pérez-Acebo, 2018). Sare honetako bide-zoru Egitura artxiboak ere lortu ahal dira bide-zoruak kudeatzeko sistematik (Gestivía), baina oso sekzio gutxi lortu ahal dira, errepide lokal bakoitzean gauzatutako proiektuen ikerketa burutu ez zelako. Bizkaiko Foru Aldundian, Garraio eta Herri Lanen Sailean, bide-zoruak kudeatzeko sistema globala garatzeko erabaki zen lehentasuna eta funtsak ematea errepide-sare garrantzitsuei.



8.1. irudia. Bizkaiko Foru Aldundiak kudeatzen dituen errepide-sareen luzerak 2016an (Diputación Foral de Bizkaia, 2017).



8.2 irudia. Bizkaiko Foru Aldundiak kudeatzen dituen errepide-sareen mugikortasuna 2016an(Diputación Foral de Bizkaia, 2017).

Horren ondorioz, nahiz eta ia errepide-sarearen erdia baztertuta egon, mugikortasunaren % 93,5a aztertuta egongo da erabiliko diren errepide-sareen bitartez.

Errepide-sare bakoitzean sartzen diren errepideak I. eranskinean erakusten dira, datu gehigarriekin:

1) Oraingo eta aurreko kodea, tartearen izendatzea eta kokapena.

2) Errepide mota, *Ley 37/2015, de 29 de septiembre, de Carreteras* (BOE, 2015) legearen arabera: bide-sariko autobideak, debaldeko bide-sareak, autobidea, errei anitzeko errepideak eta errepide konbentzionalak.

3) Errepide bakoitzean errepide-mota bakoitzaren luzera (errepide berean aldaketak egotekotan)

4) Errepide bakoitzean errepide-mota bakoitzaren hasierako eta amaierako PK-ak.

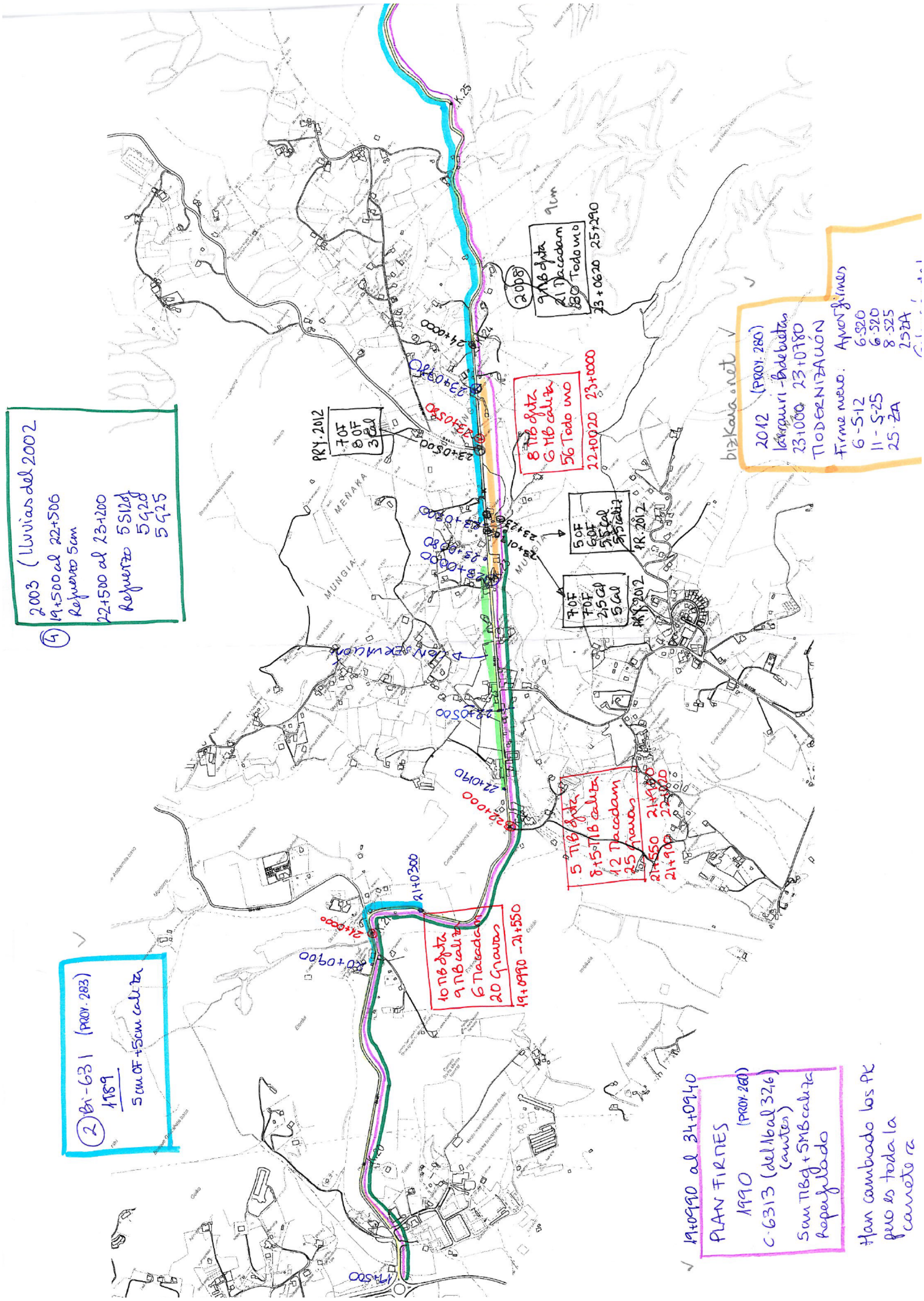
8.3.2. Analisi-metodologia

Aztertuko diren errepideak ezarrita daudenean eta IRI portaera-modeloan sartu ahal diren bide-zoru sekzioak zehaztu direnean, errepide bakoitzaren analisi indibiduala gauzatu zen. Helburu honetarako, aukeratutako errepide-sareetako errepide bakoitzean gauzatutako proiektu guztien ikerketa sakona burutu zen.

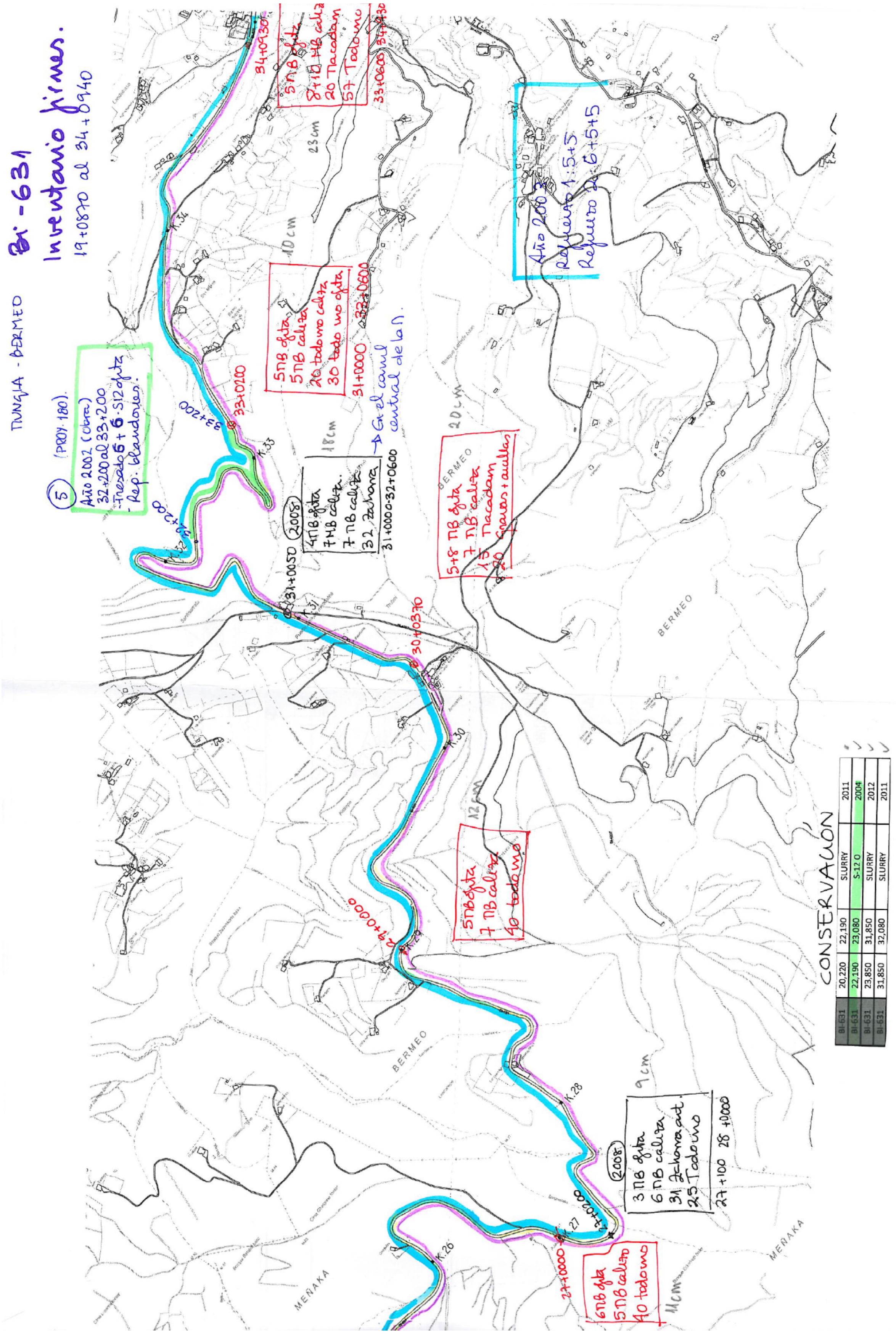
Lehenago azaldu den bezala, Bide-zoru Egitura artxiboak bide-zoruaren sekzio osoa ezaguna den tarteei buruzko informazioa ematen du, Hasierako bide-zorua bezala adierazita eta zerbitzuan jarri zen datarekin. Baita ere oraingo bide-zoruaren sekzioaren materialak eta geruza bakoitzeko lodierak aurkezten ditu. Hala ere, artxiboaren akatsetan azaldu zen bezala, ez du azaltzen gauzatu diren bitarteko mantentze- eta errehabilitazio-lanik oraingo egoerara heltzeko. Bitarteko proiektuak ez dira aipatzen, nahiz eta bide-zoruaren sekzioaren historian sartuta dago, oraingo lodierak kontuan hartzen direlarik.

Hori dela eta, ikerketa sakonagoa egin behar da, Bide-zoru Egitura artxiboa ikusteaz gain. Tesi hau burutzeko, “*UTE Agenda de Estado*” (Gestivía softwarean datuak sartzeko ingeniaritza enpresen batuketa) enpresen ingeniariek erabili zituzten zirriborroak eta eskemak eskuragarriak egon ziren. Zirriborro hauetan, bide-zoruak kudeatzeko sisteman proiektuak sartu behar zituzten ingeniariak adierazten zuten oharrak eta behin-behineko eskeman. 8.3 eta 8.4 irudiek erakusten dute behin-behineko eskema hauen adibidea erakusten dute BI-631 errepiderako (Mungia – Bermeo tartea). Errepidearen mapa bat erabiliz, Bizkaiko Foru Aldundiko artxiboetan aurkitutako proiektuak kokatzen eta adierazten dute errepidean zehar. Honelako datuak apuntatzen ziren: trafikorako zerbitzuan jartzen zen data, bide-zoruaren egitura osoa (proiektua Trazatu Berria bezala kontsideratzen batzen) edo zabaldutako geruza berriak (mantentze- eta errehabilitazio-lanen kasuan), eta hasierako eta amaierako PK-ak. Ondoren gauzatutako proiektuak ere apuntatze dira, geruza berriak, datak eta proiektua gauzatu den tartearen PK-ak adieraziz. Kolore ezberdinak aplikatzen dira eragindako luzerak azkar ikusi ahal izateko.

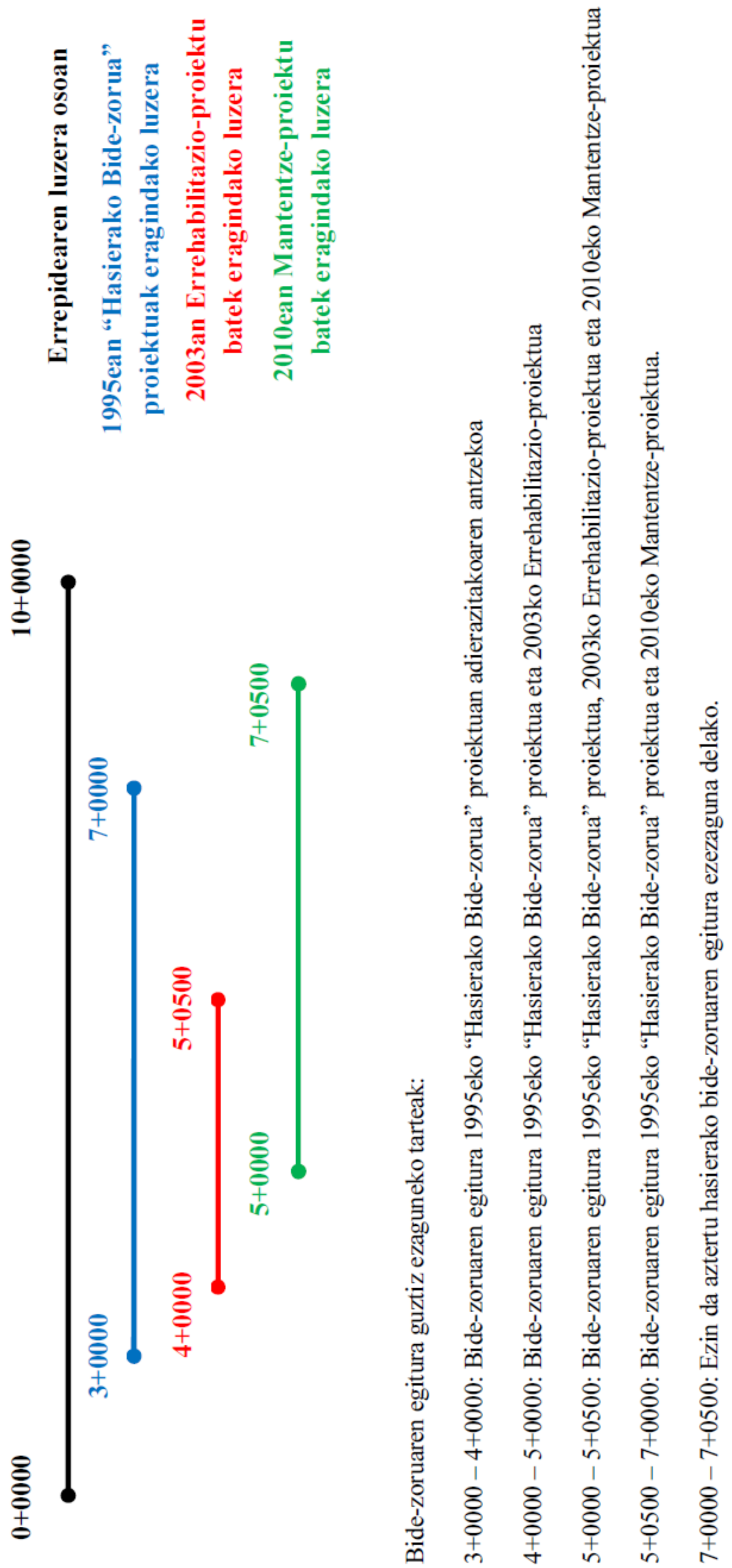
Kontuz ibili behar da tarteak aztertzean. Bide-zoru sekzio berria luzera zehatz batean gauzatu ahal da. Hala ere, mantentze- eta errehabilitazio-proiektuak ez dira gauzatzen luzera oso berean, baizik eta parte batean baino ez. Orduan, beste proiektu berri bat burutu ahal da, zeinek aurreko proiektuak eraginik ez zuen hasierako tartearen parte batean eragina duen eta aurreko proiektuak eragindako tartearen parte batean baino. Hori del eta, hasierako bide-zoruaren tartearen hasierako luzera 4 partetan zatituta egon ahal da bide-zoru egitura ezberdinekin (8.5 irudia).



8.3 irudia. "UTE Agenda de Estado"-ko ingeniariek erabilitako behin-behineko zirriborro eskematikoaren adibidea, BI-631 errepidean (Mungia – Bermeo tartea), errepidean gauzatutako proiektu guztiak aztertzeko.



8.4 irudia. UTE Agenda de Estado"-ko ingeniariek erabilitako behin-behineko zirriborro eskematikoaren adibidea, BI-631 errepidean (Mungia – Bermeo tartea), errepidean gauzatutako proiektu guztiak aztertzeko (jarraipena).



8.5 irudia. Errepide batean gauzatutako proiektuen adibidea eta aztertzeko lortutako sekzioak.

Azaldu den gisa, errepideen zirriborro eskematikoek ematen dituzten errepide bakoitzean gauzatutako errepideei buruzko pistak ematen dituzte. Hala ere, proiektu horiek Gestivíako proiektuen zerrendan egiaztatu behar dira. Lehenengo eta behin, egiaztatu behar da proiektua benetan burutu zen, zeren eta batzuetan proiektu bat gauzatzeko prest egon daiteke baina azkenean ez da burutzen edozein arrazoirengatik (diru falta, beste proiektu batez ordezkatua, etab.) Orduan, hasierako eta amaierako PK-ak, proiektu-mota (Trazatu berria, bide-zoru sekzio osoa emanez, edo Mantentze- eta Errehabilitazio-lana, M&R lana) eta datu-basea sartutako materialak eta lodierak. Batzuetan, behin-behineko eskemetan (8.3 eta 8.4 irudiak) agertzen diren balioak eta Gestivían sartutakoak ezberdinak izan ahal dira.

Errepide bakoitzean gauzatutako proiektuen analisi osoa egin denean, Errepide artxiboa prestatzen da Kontserbazio-eremua 1, 2 eta 3an dauden errepide bakoitzerako, II. eranskinean erakusten den bezala. Eranskin horretan errepide bakoitzeko Errepide artxiboa sartuta dago. Errepide bakoitzerako, aurretik determinatutako artxibo bat (8.2 taula) betetzen da, non honako informazio hau erregistratzen den:

- **Errepidearen izendatzea:** errepidearen kodea.
- **Ibilbidea:** Errepidearen izena edo konektatzen dituen hiriak
- **Puntu kilometrikoak (PK-ak):** Bizkaiko Foru Aldundiak kudeatutako tarteen hasierako eta amaierako Puntu Kilometrikoak adierazten dira.
- **Transferitutako tarteak:** Azaldu den gisa, batzuetan errepidearen tarte batzuek beste administrazio bati transferituta daude, errepideak herri bat zeharkatzen duenaren kasuan, eta tartearen hiriko naturagatik, administrazio lokalari transferitu zion (udalari)
- **Komentario orokorrak:** Errepidearen historia laburra ematen da. Aipatzen da errepidea existitzen ote zen Bizkaiko Foru Aldundiari transferitu baino lehen (1983) edo BFAk eraikitakoa den. Proiektu nagusiak adierazten dira, batez ere, hasierako bide-zoruak dituztenak.
- **Lan nagusiak:** Errepidean gauzatutako proiektu nagusiak erakusten dira dataz ordenatuta, Gestivíako proiektuaren kodea, zerbitzuan jarri zen data, eragindako tartea (PK-en bidez) eta proiektuan aipatutako geruzak eta haien lodierak.
- **IRI:** IRI-ari buruzko datuak aztertzen dira urtez urte. Kalkulurako IRI datuak erabili diren tarteak adierazten ditu (bide-zoruaren egitura osoa ezagutzen denean) eta neurtutako batez besteko IRI balioa kontsideratutako tartearentzat. Ohar batzuk gehitzen dira gezurrezko datuak ikusten badira. Adibidez, 8.5 irudiko 3+0000 – 4+0000 tartean, non 1995etik inolako proiekturik gauzatu ez denean, IRI datuak 2007tik 2011ra hobetzen direla ikusten bada (hobekuntza handia), datu horiek gezurreko datuak bezala kontsideratu ahal dira lanak gauzatu zirelako baina inolako proiekturik sartu ez delako datu-basean
- **Labainketarekiko erresistentzia:** 2011ko eta 2016ko labainketarekiko erresistentziako datuak komentatzen dira. Azalpen gehiago ematen da 9. Kapituluan.
- **Labainketarekiko erresistentzia errodadura geruzan 2016an:** Azalpen gehiago ematen da 9.

kapituluan.

Beraz, kalkuluan sartutako errepide bakoitzeko tarteen azalpen osoa ematen da eta, emaitza arraroen kasuan, posible da datuaren jatorria berrikusi, indibidualki aztertu, eta balioei edo jatorriari buruzko zalantzak egotekotan, baztertu datu horiek gezurrezkoak izateagatik. Horrez gain, datu batzuk baztertzeko arrazoiak ere azaltzen dira II. eranskineko Errepide artxiboetan.

8.4. Familia-modeloak IRI iragartzeko

Literaturan aurkitutako modeloetan ikusten den gisa, modelo ezberdinak proposatzen dira erregulartasunaren progresiorako bide-zoruaren sekzioa berria den ala mantentze- edo errehabilitazio-lana gauzatu den (George, 2000; AASHTO, 2008; 2015). Logikoa da suposatzea portaera ezberdinak egongo dira bide-zorua guztiz berria bada edo partzialki konponduta bada. Errodadura geruza (edo geruza gehiago) kendu arren eta beste batez (edo beste batzuez) material berdinarekin eta lodiera berriarekin ordezkatu arren, hondatze txarragoa erregistratzen da mantentzen diren geruzek aurreko trafikoko kargak ere jasan dituztelako. Efektu hau ikusi ahal da hainba modeloetan (European Commission, 1997; George, 2000; AASHTO, 2008; 2015, Busch *et al.*, 2010).

Hori del eta, IRI-en eboluzioa iragartzeko modeloak bi taldetan banatzea adostu zen. Alde batetik, **Trazatu Berria** bezala sailkatutako bide-zoruak, eta beste aldetik, **Mantentzea eta Errehabilitazioa (M&R)** bezala sailkatutako bide-zoruak.

Bide-zoru bat **Trazatu Berria** bezala sailkatzen da bide-zoruaren egitura osoa ezaguna denean eta mantentze- edo errehabilitazio-lanik gauzatu ez denean. Sekzio osoa ezaguna da nahaste bituminosozko geruzak eta oinarri eta azpi-oinarriko geruzak (azken bi hauek material pikortatuaz edo konglomeratzaile hidraulikoz trataturiko materialak dituztenean, ez nahaste bituminosorik) ezagunak direnean, geruza bakoitzean erabili den materialarekin eta haien lodierarekin. BFAk kudeatutako errepide-sarean ezin da aurkitu bide-zoru zurrunik, hormigoizko bide-zoruak dituztenak.

Zelaiguneari buruzko datuak jakin behar ez direla kontsideratu da, zelaiguneari buruzko informazioa daukaten proiektuak oso gutxi baitira. Proiektu batzuetan zelaiguneari buruz aurkitu ahal den informazio bakarra zelaigune-mota da, hau da, E1, E2 do E3, bide-zoruak dimentsionatzeko Espainiako arauen arabera (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b), eta oso kasu gutxitan aipatzen dira zelaigune horiek lortzeko erabili izan diren geruzak.

Orokorrean, Trazatu Berria bezala sailkatutako tarteak benetako errepide berriko trazatu berrietatik datoz, herriak edo hiriak ez zeharkatzeko gauzatzen diren saihesbideak bezala. Kasu hauetan, logikoa da trazatu berria gauzatu dela aurretik errepiderik ez zegoen lekuan. Oso kasu gutxitan, Trazatu Berri batek jarraitzen dio errepide baten trazatua. Kasu honetan, bide-zoruaren sekzio berreraikuntza osoa gauzatzen da, geruza bituminosoak eta oinarri eta azpi-oinarriko geruzak barne. Horrez gain, sekzioa “Hasierako Bide-zorua” bezala sailkatzen da datu-basean eta hala adierazten da Bide-zoru Egitura artxiboan.

8.2 taula. Errepide artxiboaren adibidea BI-712 errepiderako, II. eranskinean sartuta.

Errepidearen izendatzea	**BI-712**	
Ibilbidea	Basauri - Bolueta	
Puntu kilometrikoak (PK-ak):	**Nondik:** 387+0680	**Nora:** 388+0550
Transferitutako tarteak		
382+0530 – 384+0350: Intersection with BI-625 – Basauri 384+0350 – 387+0680: San Miguel and Basauri 388+0550 – 389+0050: Bolueta		
Komentario orokorrak:		
This road was a part of the N-625. Nowadays it is complementary road, with the main part crosses urban areas and it is transferred to municipalities. In the part managed by the Regional Government of Biscay, the oldest project conserved is from 1993 and detailed the different bituminous layers extended over existing pavements (PROJ-400). There is a maintenance work registered in 2007 in the entire road, where slurry was extended (PROJ-2008). Later, in 2008 a layer of 5 cm of S-12 was extended in 388+0240 – 388+0420 (PROJ-2009) and in 2015. A layer of 6 cm of S-12 was extended in 388+0100 – 388+0240 (PROJ-1756).		
Lan nagusiak:		
1993: PROJ-400 (01/06/1993) 387+0680 – 388+0100: AC 16 surf S 5 cm + AC 22 bin S 7 cm + AC 22 base G 12 cm + MG (ZA) 25 cm 388+0100 – 388+0550: AC 16 surf S2 5 cm + G20 7 cm 2007: PROY-2008 (01/06/2007) 387+0690 – 388+0550: LB2 2008: PROY-2009 (01/06/2008) 388+0240 – 388+0480: AC 16 surf S 5 cm 2015: PROY-1756 (30/10/2015) 388+0110 – 388+0240: AC 16 surf S 6 cm		
IRI:		
2000:		
Stretch 387+0680 – 388+0100: IRIm = 2,47.		
2004:		
Stretch 387+0680 – 388+0100: IRIm = 2,06. Sections with improvements are discarded. New IRIm = 2,59		
2007:		
Stretch 387+0680 – 388+0100: IRIm = 2,14 (After maintenance work)		
2011:		
Stretch 387+0680 – 388+0100: IRIm = 2,16		
2016:		
Stretch 387+0680 – 388+0100: IRIm = 2,62		
Labainketarekiko erresistentzia:		
2011:		
Stretch 387+0680 – 388+0100.		
2016:		
Stretch 387+0680 – 388+0100.		
Labainketarekiko erresistentzia errodadura geruzan 2016an		
2016		
388+0100 – 388+0240: PROJ-1756 (30/10/2015) AC 16 surf S 6 cm 388+0240 – 388+0480: PROJ-2009 (01/06/2008) AC 16 surf S 5 cm 388+0480 – 388+0550: PROJ-2008 (01/06/2007) LB2		

Bide-zoru bat **Mantentzea eta Errehabilitazioa (M&R)** bezala sailkatzen da material bituminoso gehigarria, geruza bat edo batzuetan, existitzen den bide-zoru baten gainean zabaltzen denean. Geruza batzuk fresatu ahal dira material berria zabaldu baina lehen, baina M&R lan bat gauzatu dela suposatuko da nahaste bituminosozko geruza bat zabaldu denean, bere lodiera gora behera.

Geruza berrien lodiera minimo edo maximoari dagokionez ez dago definiziorik errehabilitazio-lanen eta mantentze-lanen artean berezitzeko. Normalean, errehabilitazio-lanak erlazionatuta daude bide-zoruen egiturazko arazoekin eta hasierako egituraren erresistentzia berreskuratu nahi dute geruza berriak gehituz, orokorrean, geruza bat baino gehiago zabalduz. Adibidez, bide-zoruen errehabilitazioari buruzko Espainiako Arauak (Ministerio de Fomento, 2003a) adierazten du 5 eta 18 cm zabaldu behar dira bide-zoruaren egiturazko erresistentzia berreskuratzeko deflexioak handiak direnean. Aitzitik, mantentze-lanak normalean bide-zoruaren errodadura geruzaren ezaugarriak hobetzearekin erlazionatuta dago, batez ere labainketarekiko erresistentziarekin eta batzuetan, erregulartasunarekin. Orokorrean, geruza berri bakarra zabaltzen da, gainazaleko ezaugarriak errodadura geruzarekin baino ez baitaude erlazionatuta. Bide-zoruen dimentsionaketa eta materialen kalitatea kontrolatzeko (PG-3) Espainiako Arauek adierazten dute, errodadura geruzetarako, honako lodiera hauetan zabaldu behar direla materialak materialen arabera (8.3 taula).

8.3 taula. Nahaste bituminosozko errodadura geruzaren lodierak materialaren arabera Espainiako arauetan (Ministerio de Fomento, 2003b; 2015).

Material mota	**Izendatzea**	**Lodiera (cm)**
Hormigoi bituminosoa	AC16 surf S, AC16 surf D	5
	AC22 surf S, AC22 surf D	6
Nahaste etenak	BBTM 11A, BBTM 11B, BBTM 8A, BBTM 8B	2-3
Nahaste dranaitzaileak	PA 11, PA 16	4
Kare esneak	MICROF 11, MICROF 8, MICROF 5 (lehen LB1, LB2, LB3, LB4)	< 1,5

Normalean, kare-esneak aplikatzea erlazionatuta dago mantentze-lanekin, labainketarekiko erresistentzia eta egitura berreskuratu nahi delako.

Hurrengo azpi-ataletan familia-modelo bakoitzean sartutako informazioa deskribatzen da.

8.4.1. Mota guztietako bide-zoruetan sartutako informazioa

Errepide-tarte bakoitzerako, honako informazio hau sartu da analisirako (8.4 taula, Kontserbazio-eremu 1eko BI-631 errepidearen bi tarterekin adibide bezala):

- *A zutabea*, **Errepidearen izendatzea**: errepidearen kodea
- *B zutabea*, **Galtzada**: Galtzada nagusia, errepide konbentzionaletan, edo gorantz edo beherantz doana autobide, autobide eta errei anitzeko errepideetan. Lotune-adar baten galtzada ere izan daiteke, baina ez dago IRI daturik lotune-adarretan.

- *C zutabea*, **Hasierako PK-a**: tartearen hasierako Puntu Kilometrikoa
- *D zutabea*, **Amaierako PK-a:** tartearen amaierako Puntu Kilometrikoa
- *E zutabea*, **Datu-abiltzearen data zehatza**: ezaguna bada, datua hartu zen data zehatza jartzen da. Data zehatza ezagutzen ez bada, BFAko ingeniarien informazioa adierazten da, normalean udan.
- *F zutabea*, **Datu-biltzearen urtea**: IRI datua hartu zen urtea.
- *G zutabea*, **Datu-biltzearen urtea, zenbakitan**. Datu biltzearen data zenbakiz adierazten da. Adibidez, datua 2004ko uztailean bildu bazen, 2004. urtearen erdia adierazten du eta 2004,5 erregistratzen da. Bide-zoruaren benetako adina kalkulatzeko erabiltzen da.
- *H zutabea*, **IRI balioa**. Tarte horretan neurtutako IRI-en balio, ezarritako neurtze-tartearen hasierako eta amaierako PK-ena arabera, 6.21 taulan deskribatzen zen bezala. Ezkerreko eta eskuineko erreietarako balioak erregistratzen dira datu-basean, eta beraz, bi lerro sortzen dira beste zutabeetan informazio berarekin baina IRI balioa ezberdina izanik.

8.4 taula. IRI analizatzeko errepide-tarte guztietarako sartzen diren datuak (adibidearekin)

A	B	C	D	E	F	G	H
Errepidearen izendatzea	**Galtzada**	**Hasierako PK-a**	**Amaierako PK-a**	**Datu-abiltzearen data zehatza**	**Datu-biltzearen urtea**	**Datu-biltzearen urtea, zenbakitan**	**IRI balioa**
BI-631	BI-631 bakarra	34+0710	34+0810	03/07/2016	2016	2016,5	1,96
BI-631	BI-631 bakarra	34+0710	34+0810	03/07/2016	2016	2016,5	1,93
BI-631	BI-631 bakarra	34+0810	34+0910	03/07/2016	2016	2016,5	1,18
BI-631	BI-631 bakarra	34+0810	34+0910	03/07/2016	2016	2016,5	1,48

Horrez gain, tarte bakoitzerako, bai Trazatu Berria bezala bai Mantentzea edo Errehabilitazioa (M&R) bezala sailkatuta, hurrengo trafiko datu hauek sartzen dira (8.5 taula, Kontserbazio-eremu 1eko BI-631 errepidearen bi tarterekin adibide bezala)

8.5 taula. IRI analizatzeko errepide-tarte guztietarako trafikoari buruz sartzen diren datuak (adibidearekin)

T1	T2	T3	T4	T5
Datu-biltzearen urtearen AADT-a	**Datu-biltzearen urtearen H.AADT-a**	**Datu-biltzearen urtearen trafiko kategoria**	**Ibilgailu totalak**	**Ibilgailu astun totalak**
5.314	159	T31	2.394.000	74.866
5.314	159	T31	2.394.000	74.866

- *T1 zutabea*, **datu-biltzearen urtearen AADT-a**, datu-biltzearen urtean erregistratutako Eguneko Batez Besteko Intentsitatea (EBBI). Eguneko Batez besteko trafiko totala adierazten du, errepidearen bi noranzkoak kontsideratuz, bai errepide konbentzionaletan bai galtzada banandutako errepideetan.

- *T2 zutabea*, **datu-biltzearen urtearen H.AADT-a**, datu-biltzearen uretan erregistratutako Eguneko Batez Besteko ibilgailu astunen Intentsitatea. Ibilgailu astuna kontsideratzen da bere pisu total 3.500 kg baino handiagoa denean (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b). Ibilgailu astunen Eguneko Batez Besteko Intentsitatea proiektuko erreian, hau da, trafiko astun gehiago duen erreia. Orokorrean ez da jakiten zein den proiektuko erreia eta beraz, arauen arabera ezartzen da (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b). Bi erreiko errepide konbentzional batean, trafiko astuna berdin zatitzen da bi noranzkoetan, hau da errei bakoitzak trafiko astunen erdia hartzen duela kontsideratzen da (%50 - %50). Galtzada banandutako errepide batean, bi erreirekin galtzada bakoitzean noranzko bakoitzean, kanpoko erreiak (eskuinean dagoena) noranzko horretako trafiko astun osoa hartzen du. Bi galtzaden datua ematen bada (BFAko datuetan ohikoa den bezala), galtzada bakoitzeko eskuineko errei bakoitzak trafiko astunaren erdia jasotzen du. Galtzada banandutako errepidean, noranzko bakoitzeko galtzada bakoitzean hiru errei edo gehiagorekin, kanpoko erreiak (eskuinean dagoena) noranzko horretako trafiko astunaren % 85a hartzen du. Berriro ere, bi galtzadetako datua ematen bada (BFAko datuetan ohiko den bezala), galtzada bakoitzak trafiko astunaren erdia jasotzen du, eta datu horretatik, eskuineko erreiak %85a hartzen du

- *T3 zutabea*, **Datu-biltzearen urtearen trafiko kategoria**, datu-biltzearen urtean errepidearen trafiko kategoria adierazten du Ministerio de Fomento (2003b)-ren arabera. 8.6 taularen arabera ezartzen da.

8.6 taula. Trafiko astuneko kategoriak Espainian (Ministerio de Fomento, 2003b).

Trafiko kategoria	**H.AADT (ibilgailu astun/egun/errei)**	**Trafiko kategoria**	**H.AADT (ibilgailu astun/egun/errei)**
T00	H.AADT ≥ 4.000	**T31**	200 < H.AADT ≤ 100
T0	4.000 < H.AADT ≤ 2.000	**T32**	100 < H.AADT ≤ 50
T1	2.000 < H.AADT ≤ 800	**T41**	50 < H.AADT ≤ 25
T2	800 < H.AADT ≤ 200	**T42**	25 < H.AADT

- *T4 zutabea*, **ibilgailu totalak**, tartetik igaro diren ibilgailuen kopuru metatua, tartea zerbitzuan jarri zen momentutik (Trazatu Berria tarteen kasuetan), edo Mantentze- edo Errehabilitazio-lana amaitu zenetik (M&R tarteen kasuetan) datu-biltzearen momentura arte. 7. kapituluan adierazi den gisa, 2000. urtetik aurrera, *AADT*, ibilgailu astunen portzentajea, *H.AADT* eta trafiko kategoriak sartuta daude datu-basean. Aurreko urteetako datuak, 2000. urtea baino lehenago, BFAko aurreko dokumentuetan aurkitu ahal dira. Hala eta guztiz ere, 2000. urtea baino lehenagoko datuak behar direnean, %2ko hazkuntza tasa suposatu da, 90ko hamarkadarako balio zentzuduna dela egiaztatu dena (Departamento de Vivienda, Obras Públicas y Transportes, 2012; Pérez-Acebo, 2018). *AADT-eko* datuak erabiltzen direnez, zutabe honek sekzioa bi noranzkoetan zeharkatu duten ibilgailu kopuru totala adierazten du.

- *T5 zutabea*, **ibilgailu astun totalak**, tartetik igaro diren ibilgailu astunen (3.500 kg baino gehiago pisatzen duten ibilgailuak) kopuru metatuta, tartea zerbitzuan jarri edo mantentze- edo errehabilitazio-lana amaitu zenetik datu-biltzearen momentura arte. 2000. urtea baino aurreko datuak erabili behar izatekotan, hazkuntza-tasari buruz *T4* zutabean adierazitako ideia ere kontsideratu da. *H.AADT*-eko datuak erabiltzen direnez, zutabe honek sekzioa proiektuko erreitik, ibilgailu astun gehien duen erreia, zeharkatu duten ibilgailu astunen kopuru totala adierazten du, *T2* zutabeko suposaketekin.

8.4.2. Trazatu Berria bezala sailkatutako bide-zoruetan sartutako datuak

Trazatu Berria bezala sailkatutako tarteetan, aurretik deskribatutako datuez gain, honako informazio hau gehitzen da (8.7 taula, Kontserbazio-eremu 1eko BI-631 errepidearen bi tarte erakusten dira, adibide bezala).

- *I zutabea*, **irekitze-data**, tartea zerbitzuan jarri zen data zehatza.
- *J zutabea*, **irekitze-urtea**, tartea zerbitzuan jarri zen urtea
- *K zutabea*, **irekitze-urtea, zenbakiz**, tartea zerbitzuan jarri zen dataren kuantifikazioa, G zutaberako azaldu den bezala.
- *L zutabea*, **adina,** bide-zoruaren adina kalkulatzen da zutabe F-ri zutabe J kenduz (*L zutabea = F zutabea – J zutabea*). Datu-biltzearen urtearen eta zerbitzuan jarri zen urtearen arteko diferentzia adierazten du.
- *M zutabea*, **adina erreala**, bide-zoruaren adina zehatzago bat lortzen da G zutabeari K zutabea kenduz (*M zutabea = G zutabea – K zutabea*). Bide-zoruaren benetako adina adierazten du, urteetan azalduta, non 0,5 urtek 6 hilabete esan nahi du.
- *N zutabea*, **deskribapena**, bide-zoruaren egitura osoa eskematikoki deskribatzen da, geruza bakoitzaren materiala eta bere lodiera aipatuz, errodadura geruzatik hasita eta azpi-oinarriko geruzaren azken geruzan amaituz, zelaigunearen gainean dagoena. Zelaigunea ez da sartzen.
- *O zutabea*, **bide-zoru mota**, aurreko zutabearen arabera, bide-zorua malgua edo erdi-zurruna bezala sailkatzen da. Tesian zehar azaldu den bezala, ezin da bide-zoru zurrunik aurkitu BFAk kudeatzen duen errepide-sarean. Zementua edo zepa azpi-oinarria tratatzeko erabiltzen denean, bide-zoru erdi-zurruna kontsideratzen da. Bide-zoru erdi-zurrunetan aurkitu ahal diren materialak honako hauek dira: lurzoru eta zementua, hartxintxar eta zementua, zepa pikortsua eta beiraztatutako zepa. Bide-zoru malguetan, azpi-oinarria material pikortatuaz osatzen da, normalean zagor artifiziala edo hartxintxarra. Azpimarratu behar da *Guía para la actualización del inventario de firmes de la Red de Carreteras del Estado* (Ministerio de Fomento, 2011a) eta *Norma 6.1-IC Secciones de Firme* (Ministerio de Fomento, 2003b) gidetan bide-zoru malgu batek nahaste bituminosozko 15 cm baino gutxiago duena dela ezartzen da, eta nahaste bituminosozko 15 cm edo gehiago duena bide-zoru erdi-malgua dela ezartzen da. Azken berezitasun hau ez dela garrantzitsua adostu da eta bide-zoruak malguak eta erdi-zurrunak bezala baino ez dira sailkatu azpi-oinarriaren materialaren arabera.

- *P zutabea*, **geruza bituminosoen lodiera totala (cm)**, bide-zoruaren sekzioa osatzen duten geruza bituminoso guztien lodiera totala, cm-tan
- *Q zutabea*, **errodadura geruzaren lodiera (cm)**, bide-zoruaren egituraren errodadura geruzaren lodiera, cm-tan.
- *R zutabea*, **errodadura geruzaren materiala**, errodadura geruzan erabilitako material bituminosoa.
- *S zutabea*, **tarteko geruzaren lodiera (cm)**, bide-zoruaren egituraren tarteko geruzaren lodiera, cm-tan.
- *T zutabea*, **tarteko geruzaren materiala**, tarteko geruzan erabilitako material bituminosoa.
- *U zutabea*, **oinarriko geruzaren lodiera (cm)**, bide-zoruaren egituraren oinarrizko geruzaren lodiera, cm-tan.
- *V zutabea*, **oinarriko geruzaren materiala**, oinarrizko geruzan erabilitako material bituminosoa. Bide-zoruaren sekzioan bi geruza baino ez badaude, taulan sartzen dira errodadura eta tarteko geruza bezala, ez dago oinarrizko geruzarik. Bide-zoruaren egituran geruza bituminoso bakarra baldin badago, errodadura geruzan sartze da, eta ez dago oinarrizko eta tarteko geruzarik.
- *W zutabea*, **1. azpi-oinarriko geruzaren lodiera**, bide-zoruaren egituraren azpi-oinarriko 1. geruzaren lodiera, cm-tan- Orokorrena geruza ez bituminosoak oinarri eta azpi-oinarri geruzak deitzen dira. Hala era, geruza bituminosoen 3. Geruzatik berezitzeko, nahiago izan da lehenengo eta bigarren azpi-oinarriak deitzea geruza ez bituminosoei.
- *X zutabea*, **1. azpi-oinarriko geruzaren materiala**, 1. azpi-oinarriaren geruzan erabilitako materiala ez bituminosoa .
- *Y zutabea*, **2. azpi-oinarriko geruzaren lodiera**, bide-zoruaren egituraren azpi-oinarriko 1. geruzaren lodiera, cm-tan. Normalean, azpi-oinarri bat sartzen bat, baina bigarren geruza ez bituminosoa zabaldu zutenaren kasuan, zuzen zelaigunearen gainean zabaldutako geruza da..
- *Z zutabea*, **2. azpi-oinarriko geruzaren materiala**, 2. azpi-oinarriaren geruzan erabilitako materiala ez bituminosoa.

8.7 taula. Trazatu Berria bezala sailkatutako tarteetan sartutako datuak.

I	J	K	L	M	N	O
Irekitze-data	**Irekitze -urtea**	**Irekitze-urtea, zenbakiz**	**Adina**	**Adina erreala**	**Deskribapena**	**Bide-zoru mota**
01/05/2015	2015	2015,3	1	1,2	BBTM11A 3cm + AC 16 bin S 8 cm + GC 29 cm	Erdi-zurruna
01/05/2015	2015	2015,3	1	1,2	BBTM11A 3cm + AC 16 bin S 8 cm + GC 29 cm	Erdi-zurruna

P	Q	R	S	T	U	V
Geruza bituminosoen lodiera totala (cm)	**Errodadura geruzaren lodiera (cm)**	**Errodadura geruzaren materiala**	**Tarteko geruzaren lodiera (cm)**	**Tarteko geruzaren materiala**	**Oinarriko geruzaren lodiera (cm)**	**Oinarriko geruzaren materiala**
11	3	BBTM11A	8	AC 16 base S	-	-
11	3	BBTM11A	8	AC 16 base S	-	-

W	X	Y	Z
1. azpi-oinarriko geruzaren lodiera	**1. azpi-oinarriko geruzaren materiala**	**2. azpi-oinarriko geruzaren lodiera**	**2. azpi-oinarriko geruzaren materiala**
29	Hartxintxar eta zementua		
29	Hartxintxar eta zementua		

8.4.3. Mantentzea eta Errehabilitazio (M&R) bezala sailkatutako bide-zoruetan sartutako datuak

Tartearen identifikatzeko informazioaz gain, 8.4.1 atalean azaltzen den bezala A – H zutabeetan sartzen dena, eta trafikoari buruzko datuez gain (T1 – T5 zutabeak) M&R tarteetarako honako informazio hau sartu da modelizatzeko (8.8 taula, Kontserbazio-eremua 1eko BI-634 errepideko eta Kontserbazio-eremua 2ko BI-633 errepideko bi tarteak erakusten dira adibide bezala)

- *I zutabea*, **M&R lanaren data**, M&R lana amaitu den data zehatza.
- *J zutabea*, **M&R lanaren urtea**, M&R lana amaitu den urtea.
- *K zutabea*, **M&R lanaren urtea, zenbakiz**, M&R lana amaitu den dataren kuantifikazioa, G zutaberako azaldu den bezala.

- *L zutabea*, **adina,** M&R lanaren adina kalkulatzen da F zutabeari J zutabea kenduz (*L zutabea = F zutabea –J zutabea*). Datu-biltzearen urtearen eta M&R lana amaitu zen urtearen arteko diferentzia adierazten du.

- *M zutabea*, **adin erreala**, M&R lanaren adina zehatzago bat lortzen da G zutabeari K zutabea kenduz (*M zutabea = G zutabea – K zutabea*). Bide-zoruaren benetako adina adierazten du, urteetan azalduta, non 0,5 urtek 6 hilabete esan nahi du.

- *N zutabea*, **M&R lanaren deskribapena**, Mantentzea eta Errehabilitazio lanaren deskribapen eskematikoa, geruza bakoitzaren materiala eta bere lodiera aipatuz, errodadura geruzatik hasita. Adierazi behar da materiala fresatu den, egituraren benetako lodiera jakiteko.

- *O zutabea,* **geruza bituminoso berrien lodiera totala (cm)**, dauden geruzen gainean (geruza batzuk fresatu eta gero, edo ez) zabaldutako geruza bituminosoen lodiera total, cm-tan.

- *P zutabea,* **errodadura geruza berriaren lodiera (cm)**, dauden geruzen gainean (geruza batzuk fresatu eta gero, edo ez), zabaldutako errodadura geruzaren lodiera, cm-tan. Kare-esne bat zabaldu denean, adierazitako lodiera 0,5 cm da. Nahiz eta lodiera maximoa 1,5 cm izan, 8.3 taulan adierazten den bezala, BFA bide-zoruak kudeatzeko sistemak ez du lodiera totalaren handitzerik kare-esnea zabaltzen denean. Errodadura geruza berria agertzen da, Kare-esnea (LB2), baina dagoen bide-zoruaren gainean kare-esnea baino zabaltzen ez bada, lodiera berea adierazten da, baina kare-esneko errodadurarekin.

- *Q zutabea*, **errodadura geruza berriaren materiala**, dagoen bide-zoruaren gainean zabaldutako errodadura geruzaren materiala

- *R zutabea*, **dagoen bide-zoruaren deskribapena**, dagoen bide-zoruaren egitura, gauzatzen den M&R lana baino lehen, eskematikoki deskribatuta.

- *S zutabea*, **bide-zoru mota**, aurreko zutabearen deskribapenaren arabera, bide-zorua malgua edo erdi-zurruna sailkatzen da, Trazatu Berriko O zutabean azaldutako irizpideak jarraituz

- *T zutabea*, **dagoen bide-zoruaren data**, dagoen bide-zoruaren gauzatzearen data zehatza. Trazatu Berria edo M&R tartea izan daiteke eta beraz, zerbitzuan jarri zen data edo bide-zoru sekzio horretan azken M&R lana gauzatu zen data agertzen da.

- *U zutabea*, **dagoen bide-zoruaren urtea**, aurreko zutabearen dataren urtea

- *V zutabea*, **dagoen bide-zoruaren urtea, zenbakiz**, aurreko dataren kuantifikazioa familia-modelo guztien *G* zutabean azaldu den bezala.

- *W zutabea*, **aurreko lanaren adina**, azken lanaren adina (bai eraikuntza berria bai M&R lana) kalkulatzen da J zutabeari U zutabea kenduz (*W zutabea = J zutabea –U zutabea*). Tartearen azken bi jardueren arteko denbora-tartea adierazten du.

- *X zutabea*, **aurreko lanaren adin erreala**, azken lanaren adina zehatzagoa (bai eraikuntza berria bai

M&R lana) kalkulatzen da K zutabeari V zutabea kenduz (*X zutabea = K zutabea –V zutabea*). Tartearen azken bi jardueren arteko benetako denbora-tartea adierazten du, urteetan azalduta, non 0,5 urtek 6 hilabete esan nahi du

- *Y zutabea*, **aurreko lana**, aurreko lanaren natura, Trazatu Berria eta M&R lana berezituz.
- *Z zutabea*, **lanen historia**, tartearen lanen eta jardueren historia osoa, definitzen dena eta aurrekoa barne.
- *AA zutabea*, **dagoen bide-zoruaren geruza bituminosoen lodiera totala (cm)**, dagoen bide-sekzioaren geruza bituminoso guztien lodiera, M&R lana baino lehen, cm-tan.
- *AB zutabea*, **dagoen bide-zoruaren azpi-oinarriko geruzen lodiera totala (cm)**, dagoen bide-sekzioaren azpi-oinarrizko geruza ez bituminoso guztien lodiera, M&R lana baino lehen, cm-tan. Egon ahal diren bi geruzak sartuta.
- *AC zutabea*, **dagoen bide-zoruaren azpi-oinarriaren materiala**, dagoen bide-sekzioaren azpi-oinarrizko geruza ez bituminoso guztien materialak, M&R lana baino lehen. Bi material badaude biak adieraziko dira.
- *AD zutabea*, **bide-zoru berriaren deskribapena**, sortzen den bide-zoruaren egitura berria, M&R lana gauzatu ondoren zegoen bide-zoruaren gainean, kontuan hartuz zeuden geruzak, fresatu ahal izan direnak eta berriak
- *AE zutabea*, **bide-zoru berriaren geruza bituminosoen lodiera totala (cm)**, bide-zoruaren egitura berria osatzen duten geruza bituminoso guztien lodiera totala, cm-tan.
- *AF zutabea*, **bide-zoru berriaren errodadura geruzaren lodiera (cm),** bide-zoruaren egitura berriaren errodadura geruzaren lodiera, cm-tan. Kare-esnea zabaldu bada, ez du adierazten 0,5 cm, = eta P zutabeetan adierazi den bezala, baizik eta kare-esnearen azpian dagoen geruzaren lodiera.
- *AG zutabea*, **bide-zoru berriaren errodadura geruzaren materiala**, bide-zoruaren egitura berriaren errodadura geruzaren materiala. Kare esnea zabaldu bada, aurreko zutabean azaldu den bezala, kare-esnea aipatzen da, nahiz eta lodiera bere azpian dagoen geruzarena izan.
- *AH zutabea*, **bide-zoru berriaren tarteko geruzaren lodiera (cm)**, bide-zoruaren egitura berriaren tarteko geruzaren lodiera, cm-tan. Kare-esnea errodadura geruzan adierazi bada, tarteko geruzaren lodiera errodadura geruzan aipatzen da. Hori dela eta, kasu honetan 3. geruza bituminosoaren lodiera adierazten da, kare-esne eta beste baten ostean.
- *AI zutabea*, **bide-zoru berriaren tarteko geruzaren materiala**, bide-zoruaren egitura berriaren tarteko geruzaren materiala, errodadura geruzan kare-esnearen presentziaren irizpideak kontuan hartuz.
- *AJ zutabea*, **bide-zoru berriaren oinarriko geruzaren lodiera (cm)**, bide-zoruaren egitura berriaren oinarrizko geruzaren lodiera, cm-tan, errodadura geruzan kare-esnea egotearen

posibilitatea kontuan hartuz.

- *AK zutabea*, **bide-zoru berriaren oinarriko geruzaren materiala**, bide-zoruaren egitura berriaren oinarrizko geruzaren material. Errodadura geruzan kare-esnea badago, geruza bituminosoen 4. geruzari erreferentzia egiten dio, egotekotan.

- *AL zutabea*, **bide-zoru berriaren azpi-oinarriko geruzen lodiera totala (cm)**, azpi-oinarriko geruza guztien lodiera totala, cm-tan. Bi geruza egotekoan bien lodieren batuketa adierazi behar da.

- *AM zutabea*, **bide-zoru berriaren azpi-oinarriko geruzen materialak**, azpi-oinarriko geruzetan dauden materialak, M&R lanak gauzatu baino lehen zeudenak.

8.4.4. Datuen tratamendua eta familia bakoitzeko datu kopuru eskuragarri

8.3.1 atalean azaldu den bezala, sare-maila lokaleko errepideak (sare horia) ez dira kontuan hartu IRI portaera modelizatzeko errepide mota hau ez zen hain zehatz aztertu beste sare-mailak bezala bere garrantzi txikiagatik. Bizkaiko Foru Aldundiko ingeniarien esfortzuak sare-maila garrantzitsuenetan zentratuta zeunden: lehentasuneko intereseko sarea (sare gorria), oinarrizko sarea (sare laranja), sare osagarria (sare urdina) eta eskualdeko sarea (sare berdea) (6.3 taula). Hori dela eta, nahiz eta errepide-sare totalaren % 46,2a ez da sartzen analisian, aztertutako sareetatik Bizkaiko lurraldearen mugikortasunaren % 93,5ak zirkulatzen du, eta beraz, errepide nagusi guztiak sartuta daude.

IRI-a modelizatzeko aztertu diren errepide-motari dagokionez, ezarri zen bi erreiko errepide konbentzionalak aztertzen direla (errepide konbentzionalak, Espainiako *Ley 37/2015, de 29 de septiembre, de Carreteras* (BOE, 2015)). Galtzada banandutako errepideetako galtzada batean, 8.4.1 atalean *T2* zutaberako komentatu den bezala, ibilgailu astun gehienek eskuineko erreitik zirkulatzen dute eta oso gutxi beste erreietatik. Beraz, galtzadaren portaera IRI-en aurrean ez da errepide konbentzional batean dagoenaren antzekoa. Errepide konbentzional batean, galtzadaren bi erreiek karga berdinak dituzte. Hori dela eta, tesi honetan IRI-en eboluzioaren modeloak errepide konbentzionaletan baino ez da proposatu nahi.

Horren ondorioz, Kontserbazio-eremu 1, 2 eta 3ko errepideak baino analizatu dira IRI modeloetarako. Kontserbazio-eremua 4 Bilbo inguruan dauden autobide eta autobiek osatua dago.Bi erreiko errepide konbentzionaleko tarte oso gutxi daude Eremu 4an (5 km baino gutxiago, N-634 errepidea kontuan hartu gabe, non oso tarte gutxitan bide-zoru sekzio osoa ezaguna den) eta beraz, ez dira analizatzen.

8.8 taula. Mantentzea eta Errehabilitazioa (M&R) bezala sailkatutako tarteetan sartutako datuak

I	J	K	L	M	N	O	P	Q
M&R lanaren data	**M&R lanaren urtea**	**M&R lanaren urtea, zenbakiz**	**Adina**	**Adin erreala**	**M&R lanaren deskribapena**	**Geruza bituminoso berrien lodiera totala (cm)**	**Errodadura geruza berriaren lodiera (cm)**	**Errodadura geruza berriaren materiala**
01/06/2010	2010	2010,5	6	6	BBTMA 11A 3 cm + AC 16 bin S 5 cm	8	3	BBTM 11A
01/08/2014	2014	2014,6	2	1,9	Kare-esnea	0,5*	0,5*	LB2

R	S	T	U	V	W	X
Dagoen bide-zoruaren deskribapena	**Bide-zoru mota**	**Dagoen bide-zoruaren data**	**Dagoen bide-zoruaren urtea**	**Dagoen bide-zoruaren urtea, zenbakiz**	**Aurreko lanaren adina**	**Aurreko lanaren adin erreala**
AC 22 surf S 6 cm + AC 22 base G 11 cm + Material pikortatua 25 cm	Malgua	01/06/1992	1992	1992,5	18	18
AC 16 surf S 5 cm + AC 16 surf S 6cm + AC 22 base G 11 cm + Material pikortatua 25 cm	Malgua	01/01/2012	2012	2012	2	2,6

Y	Z	AA	AB	AC	AD
Aurreko lana	**Lanen historia**	**Dagoen bide-zoruaren geruza bituminosoen lodiera totala (cm)**	**Dagoen bide-zoruaren azpi-oinarriko geruzen lodiera totala (cm)**	**Dagoen bide-zoruaren azpi-oinarriaren materiala**	**Bide-zoru berriaren deskribapena**
M&R	Trazatu berria – M&R – M&R	17	25	Material pikortatua (ZA)	BBTM 11A 3 cm + AC 16 bin S 5 cm + AC 22 surf S 6 cm + AC 22 base G 11 cm + Material pikortatua 25 cm
Trazatu berria	Trazatu berria – M&R	22	25	Material pikortatua (ZA	LB2 + AC 16 surf S 5 cm + AC 16 surf S 6cm + AC 22 base G 11 cm Material pikortatua 25 cm

AE	AF	AG	AH	AI	AJ	AK	AL	AM
Bide-zoru berriaren geruza bituminosoen lodiera totala (cm)	**Bide-zoru berriaren errodadura geruzaren lodiera (cm)**	**Bide-zoru berriaren errodadura geruzaren materiala**	**Bide-zoru berriaren tarteko geruzaren lodiera (cm)**	**Bide-zoru berriaren tarteko geruzaren materiala**	**Bide-zoru berriaren oinarriko geruzaren lodiera (cm),**	**Bide-zoru berriaren oinarriko geruzaren materiala**	**Bide-zoru berriaren azpi-oinarriko geruzen lodiera totala (cm)**	**Bide-zoru berriaren azpi-oinarriko geruzen materialak,**
25	3	BBTM 11A	5	AC 16 bin S	17	AC 22 surf S/AC 22 base G	25	MG (ZA)
22	5	LB2	6	AC 16 surf S	11	AC 22 base G	25	MG (ZA)

Bide-zoruaren egitura osoa ezaguna den eremu 1, 2 eta 3ren tarteak analizatu direnean, 100 m-ko tarte guztiak batera aztertzen saiatu zen. Aldagai azaltzaile berdinak dituen tarte baten barruan oso bariantza handia ikusten zen. Adibidez, 2 km-ko tarte homogeneo batean, bide-zoru sekzio berearekin proiektu beretik eta trafiko bolumen berdina (bai trafiko totala bai trafiko astuna), 100 m-ko 10 balio eskuragarri daude. Tarte horretan, IRI datuen bariantza handia nabaritu zen, bide-zoru ezaguneko tarte guztien datuak erlazionatzea ezinezkoa eginez. Hori dela eta, batez besteko IRI kalkulatzea erabaki zen, ezaugarri bereak dituen tarte bakoitzerako, hau da, bide-zoruaren egituran, bide-zoruaren adinean (egitura dator proiektu beretik) eta trafiko bolumenetan balio bereak izan behar ditu. Orduan, adibidean, ezaugarri berdineko 100 m-ko 20 azpi-tarteen batez bestekoa kalkulatzen da. Prozedura hau, batez besteko balioa kalkulatzea ezaugarri bereak dituen tartean logikoa da eta normala zeren eta literaturan proposatutako modelo deterministakoak batez besteko IRI-en balio iragartzen saiatzen dira, aldagai azaltzaile batzuetatik eta ez dagoen bariantza ezta tartearen barruan dagoen balioen heina. Bariantza hau oso garrantzitsua da eta probabilitatezko modeloen helburu nagusietako bat kontsideratzen da, 4. kapituluan azaldu den moduan. Bide-zoruak probabilitatezkoak direla esaten da (Tjan and Pitaloka, 2005; Abaza, 2016a, Li *et al.*, 1997; Hong, 2014) eta aldakortasun honek egiaztatzen du. Hala ere, probabilitatezko modeloak erabiltzen dira aldagai independente gutxi daudenean. Baina informazio zehatza badago (Bizkaiko errepide-sarean bezala), analisi deterministakoa gomendatzen da. Horren ondorioz, ezaugarri bereko tarteen batez bestekoa kalkulatzen da eta beraz, batez besteko IRI-a iragarriko da. Gainera, analisia osatzeko, tarte homogeneo baten barruan , desbideratze estandarra edo tipikoa kalkulatzen da (8.13 ekuazioa). Hala, aldakortasunari buruzko informazio gehiago aztertu ahal da.

$$s = \sqrt{\sigma^2} = \sqrt{\frac{\sum_{i=1}^{N}(x_i - \mu)^2}{N}} \qquad [8.13]$$

Non x_i 100 m-ko tarte bakoitzaren IRI datua da, μ IRI-en batez bestekoa da datu guztietarako eta N da datu guztien kopurua kontsideratutako tarte homogeneoan. 8.13 ekuazioan, populazioko desbideratze estandarra aukeratu zen eta ez lagineko desbideratze estandarra (*N*-1izendatzailean) datuek ezaugarri bereko tartearen populazio totala adierazten dutelako. Ez da populazio batetik ateratako lagina baizik eta populazio osoa.

Ezaugarri bereko Tarte bakoitzeko batez besteko IRI-a kalkulatu ondoren, Eremu 1, 2 eta 3tik ateratako behaketen kantitate totala 8.9 taulan erakusten da.

Errepide konbentzionaletako 186 Trazatu Berria tarteetatik, 105 tarte bide-zoru malgukoak dira eta 81 bide-zoru erdi-zurrunekoak dira. Galtzada banandutako 20 Trazatu Berria tarte guztiek bide-zoru erdi-zurruna daukate. Errepide konbentzionaletan dauden 112 tarteetatik, 72 tarteek bide-zoru malgua daukate eta 40k erdi-zurruna. Galtzada banandutako 30 M&R tarteek bide-zoru zurruna daukate.

8.9 taula. Kontsideratutako informazioan balio ezberdinak dituzten tartean, IRI modelizatzeko aukeratuak, Kontserbazio-eremu 1, 2 eta 3tik.

	Eremu 1					**Eremu 2**					**Eremu 3**			
	Trazatu berria		**M&R**			**Trazatu berria**		**M&R**			**Trazatu berria**		**M&R**	
Road	**2e**	**2g**	**2e**	**2g**	**Road**	**2e**	**2g**	**2e**	**2g**	**Road**	**2e**	**2g**	**2e**	**2g**
BI-631	4				BI-623	5		8	18	BI-624				
BI-634	2		2		BI-633	32		16		BI-625	14	8	13	12
BI-635	5		12		BI-638					BI-630				
BI-735	2		2		BI-732	3				BI-712	2		3	
BI-737	3				BI-2224	8		1		BI-745				
BI-2101					BI-2301	2		5		BI-2521				
BI-2120	3		2		BI-2405	8		2		BI-2522	1			
BI-2121	8		1		BI-2543	4		16		BI-2617				
BI-2122	2				BI-2632					BI-2625	3			
Bi-2153	1				BI-2636					BI-2701	7		13	
Bi-2235	6		2		N-240	18		1		BI-2757	7			
Bi-2237	2		2		N-636					BI-2794	3			
Bi-2238	6		5							N-639	12		2	
BI-2704	12		4											
BI-2713	1													
BI-2731		4												
Guztira	57	4	32		**Guztira**	80	8	49	18	**Guztira**	49	8	31	12

Trazatu berria, errepide konbentzionalak, guztira	186
Trazatu berria, galtzada banandutako errepideak, guztira	20
M&R tarteak, errepide konbentzionalak, guztira	112
M&R tarteak, galtzada banandutako errepideak, guztira	30
GUZTIRA	**358**

Oharra: 2e = bi erreiko errepide konbentzionalak. 2g = Galtzada banandutako errepideak.

8.5. Trazatu Berria tarteetarako proposatutako IRI portaera-modeloak

8.5.1. Modelizazioan kontuan hartutako aldagaiak

Bizkaiko errepide-sarean IRI portaeran eragina izan ahal duten faktore bezala, modeloan sartzen diren aldagai independenteak deskribatzeko, 8.1 taulan aurkeztutako aldagai-talde berdinak erabiliko dira.

8.2 atalean azaldu zen gisa, hainbat aldagai identifikatu ziren erregulartasunaren eboluzioan eragina duten faktore bezala. 8.1 taulan zerrendatutako faktoreetatik, Bizkaiko errepide-sarean IRI-a iragartzeko aukeratu direnak deskribatzen dira, zein datu erabiltzen diren, bide-zoruak kudeatzeko sistemaren datu-basean dagoen informazio eskuragarriaren arabera.

8.5.1.1. Adina

Bide-zoruaren adina erabilitako ohiko faktorea da erregulartasuna modeloetan eta beraz, kalkuluan sartu zen bi eratan. *L* zutabean balioak, ***Age***, zerbitzuan jarri zen urte naturalaren (edo M&R lana amaitzearen urtearen) eta datu-biltzearen urtearen diferentzia adierazten du. Data horiek urtean zehar aldatu ahal direnez, M zutabea, adina erreala, ***R.Age***, adinaren balio zehatzago bat adierazten du, data zehatzak kontuan hartzen baitira. Adina urtetan azalduta dago, frakzio dezimal batean, non 0,5 urtek 6 hilabete adierazten duen. Gainera, errepidea zerbitzuan jartzen denean edo M&R lana amaitzen denean datu-biltzearen urte berdinean, *age* faktoreak 0 adierazten du, errepidea zerbitzuan jartzen dela esan nahi duela suposatu ahal da. Hala ere, *R.Age* bide-zoru berri (edo konpondutakoaren) adin erreala adierazten du.

8.5.1.2. Trafiko bolumenak

5. kapituluan eta 8.2 atalean aipatu zen trafiko bolumenaren garrantzia bide-zoruaren hondatze-faktore bezala. Errepidearen egituran aplikatutako kargak adierazten dute. Orokorrean, *Equivalent Single Axle Load* (*ESAL*) (Ardatz Sinpleko Karga Baliokidea) erabiltzen da ibilgailuen hainbat pisutako kalteak karga estandar batek eragiten duen kalte bihurtzeko. Espainian, *ESAL*-ak 13 t-ko pisua dauka. Sekzio batetik igaro diren *ESAL*-en kopurua determinatzeko, hainbat ibilgailuen pisuak jakin behar dira eta transferentzia funtzio bat erabiltzen da eragindako kaltea bihurtzeko, normalean laugarren graduko parabolaren itxura duena. Hala eta guztiz ere, sekzio bakoitza igarotzen duten ibilgailu ezberdinen kopurua ezezaguna da Bizkaian. Trafiko datuak ibilgailu arina eta ibilgailu astunak baino ez ditu berezitzen. Ibilgailu astuna kontsideratzen da bere pisua 3.500 kg baino handiago denean (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b). Neurketa batzuk egin ziren pisuen banaketa ezagutzeko baina errepide zehatz batzuetako da. Errepide horietan, data horiek erabili ahal dira sekzioa zeharkatu duten *ESAL*-en kopuru totala kalkulatzeko, baina beste errepideetan, balio horiek estrapolatu behar izango lirateke. Ondorioz, bihurtzeko ekuazioak errepide-sare osoan erabili ordez, zentzudunagoa dirudi datu eskuragarriak zuzen erabili: trafiko totalaren bolumena eta astunen trafiko bolumena

8.4.1 atalean azaldu den moduan, modeloan sartutako datuak dira: Eguneko Batez Besteko Intentsitatea (***AADT***) ibilgailu/egun-etan (*T1 zutabea*), ibilgailu astunen Eguneko Besteko Intentsitatea (***H.AADT***) ibilgailu astun/egun-etan proiektu-erreian (*T2 zutabea*), Trafiko kategoria Ministerio de Fomento (2003b)-ren arabera datu-biltzearen urtean (*T3 zutabea*), sekziotik igaro diren ibilgailu kopuru metatuta totala, ***TotVeh*** (*T4 zutabea*) eta sekziotik igaro diren ibilgailu astunen kopuru metatuta total, ***TotH.Veh*** (*T5 zutabea*). Bi zutabe hauetarako, tartearen zerbitzuan jartzearen data eta datu-biltzearen data zehatzak erabiltzen dira. 2000. urtetik *AADT*, ibilgailu astunen portzentajea, *H.AADT* eta trafiko kategoria datu-basean eskuragarri daude. Aurreko urteetako datuak BFAko dokumentazioan aurkitu ahal da. Hala ere, 2000. urteko baino lehenagoko datuak behar direnean, trafikoaren hazkuntza tasa % 2koa izan dela suposatu da. Balio hori zentzuduna izateak egiaztatu du 90eko hamarkadan (Departamento de Vivienda, Obras Públicas y Transportes, 2012, Pérez-Acebo, 2018).

8.5.1.3. Egiturazko parametroak

8.1 taulan ikusten den bezala, egiturazko parametroak erabiltzen dira IRI modeloetan, normalean "*Structural Number*" (SN)-en bidez edo bere aldakuntzak. Hala ere, hau ez da Espainian erabiltzen den parametroa eta ez dago eskuragarri datu-basean, lortzeko testak egin ez zirelako. Hori dela eta, bide-zoru baten kargak jasateko egiturazko ahalmena sartzen da hondatze-modeloan geruza bakoitzean dagoen materiala eta geruza bakoitzaren lodiera adieraziz, 8.4.2 eta 8.4.3 atalean azaldu den gisa. Geruza bituminosoentzat, 3 geruza suposatu dira: errodadura geruza, tarteko geruza eta oinarriko geruza, Espainiako arauen irizpideak jarraituz. Bi geruza bituminoso baino ez badaude, errodadura eta tarteko geruza bezala erregistratzen dira. Oinarriko eta azpi-oinarriko geruza ez bituminosoentzat, 1. azpi-oinarria eta 2. Azpi-oinarria deitzen dira, oinarriko geruza bituminosotik berezitzeko. Normalean bide-zoru malguetan azpi-oinarriko geruza bakarra dago, baina bide-zoru erdi-zurrunetan, azpi-oinarria bi geruzaz osatu ahal da. Ondorioz, bi geruza posible sartzen dira.

Geruza bituminosoen lodiera totala, ***TotBit***, sartzen da geruza bituminosoen kontribuzioa egituraren erresistentziarako ebaluatzeko parametroa bezala, cm-tan azalduta. Hala eta guztiz ere, parametro honek ez islatzen material bakoitzaren ezaugarri ezberdinak, eta ez du berezitzen hormigoi bituminosoen (AC), nahaste etenen (BBTM) eta nahaste drainatzaileen (PA) artean. Azaldu den bezala, kare-esneen lodiera (< 1,5 cm) ez dago erregistratuta BKSn lodiera gehigarri bezala. Geruza bituminosoen natura ezberdina islatzeko asmoz, parametro bat sortzen da, nahaste bituminosoen Egiturazko Erresistentzia (*Structural Strength*, *SS*), $\boldsymbol{SS_{Bit}}$, eta kalkulatzen da geruza bituminosoen lodiera eta dauden materialen Young-en moduluaren biderketaren batuketa bezala, 8.14 ekuazioan adierazten den moduan.

$$SS_{bit} = \sum_{i=1}^{n} Bth_i \cdot E_i \qquad [8.14]$$

Non

SS_{bit}: Geruza bituminosoen Egiturazko Erresistentzia (*Structural Strength*), geruza bituminoso bakoitzaren kontribuzioa kargak eusteko islatzen duena
Bth_i : geruza bituminoso bakoitzaren lodiera, cm-tan
E_i : material bituminoso bakoitzaren Young-en modulua, 8.10 taulan erakusten dena.
n : bide-zoru sekzioan dauden geruza bituminosoen kopuru totala

8.10 taula. Young-en modulua material bituminoso batzuetarako (Departamento de Vivienda, Obras Públicas y Transportes, 2012)

Materialak	Young-en modulua (MPa)
Hormigoi bituminosoak (AC): Trinkoak (D) eta erdi-trinkoak (S)	6.000
Hormigoi bituminosoak (AC): lodiak (G)	5.000
Nahaste etenak (BBTM A and B) eta nahaste drainatzaileak (PA)	4.000

Azpi-oinarriaren kontribuzioa islatzeko, antzeko parametroa sortu da, SS_{sub}, 8.15 ekuazioarekin kalkulatuta.

$$SS_{sub} = \sum_{i=1}^{n'} Sth_i \cdot E_i \qquad [8.15]$$

Non

SS_{sub}: azpi-oinarriko geruzen Egiturazko Erresistentzia (*Structural Strength*), azpi-oinarriaren geruza bakoitzak kargak eusteko kontribuzioa islatzen duena

Sth_i : azpi-oinarriaren geruza ez bituminoso bakoitzaren lodiera, m-tan

E_i: geruza bakoitzeko Young-en modulua, 8.11 taulan lortu ahal dena

n': bide-zoruaren sekzioan daude geruza ez bituminosoen kopuru totala

8.11 taula. Hainbat material ez bituminosoren Young-en modulua (Departamento de Vivienda, Obras Públicas y Transportes, 2012)

Materiala	**Young-en modulua (MPa)**
Lurzoru eta zementua material pikortatuarekin	8.000
Lurzoru eta zementua zagor artifizialarekin	12.000
Hartxintxar eta zementua	22.000
Zagor artifiziala	250*
Zepa pikortsua	10.000*
Beiratutako zepa	10.000*

* Balio hurbilduak dira

Bide-zoruaren sekzioaren erresistentziaren ideia globala izateko, kalkulatutako geruza bituminosoen eta geruza ez bituminosoen kontribuzioak batzen dira beste parametro batean, Egiturazko Erresistentzia Totala, (*Structural Strength total*), $\boldsymbol{SS_{tot}}$, honela kalkulatzen dena:

$$SS_{tot} = SS_{bit} + SS_{sub} \qquad [8.16]$$

Zelaigunearen parametroei dagokienez lehenago komentatu da so datu gutxi erregistratuta daudela BKSn. Horren ondorioz, ez dira zelaigunearen parametrorik sartu.

8.5.1.4. Beste parametroak

8.1 taulan beste parametroak zerrendatu ziren eta jarraian komentatzen dira:

- **Akatsak**. Errepide batean akatsen presentziak, batez ere pitzadurak eta gurpil-arrastoa, erregulartasuna handitzea suposatzen du eta erregulartasunaren eboluzioa iragartzeko balio du. Hala era, pitzadurak eta erregulartasunari buruzko datuak datu-basean ez daude osorik eta erabaki zen ez sartzea. Gainera, modelo enpirikoa garatu nahi da, teoria mekanizistak ekidinez.
- **Klima**. Azaldu den gisa, Bizkaiko probintzia txikia da (2.217 km^2) eta klima berea dauka azalera

osoan (klima ozeanikoa). Beraz, aldakuntza txikiak ikusi ahal dira probintzia zatitu ahal den eremuen artean. Aldakuntza horiek, tenperatura edo prezipitazioei dagozkienez. ez daukate %10 baino handiagoko diferentziarik. Klima sartzea eragina duen faktore bezala logikoa da modelo globala garatzen denean, HDM-III eta HDM-4n, edo MEPDG-ek proposatzen dituenak bezala (AAASHTO, 2008; 2015), non oso klima ezberdinak aurkitu ahal dira. Bizkaiko kasuan, esan daiteke klima zehatz eta espezifiko batera (klima ozeanikoa) garatzen ari dela.

- **Hasirako IRI**-a. Errepidearen hasierako IRI-a ez dago sartuta datu-basean. Hasierako IRI maximoak, errepidea trafikorako ireki baino lehen ezin dira pasatu (Ministerio de Fomento, 2001; 2004a; 2008; 2011b; 2015) baina proiektu bakoitzeko balio zehatz ez daude sartuta datu-basean.
- **Beste parametroak.** Modelo batzuk datu geometrikoak erabiltzen dituzte (erreiaren zabalera) edo datu funtzionala (hiri barneko edo hiri arteko errepidea). Erreiaren ohiko zabalera 3,50 m denez errepide guztietan eta tarte guztiak hiri arteko errepidetakoak direnez, ez dira beste parametrorik aukeratu.

Literaturan, IRI portaera-modeloak bide-zoru malguentzat eta bide-zoru erdi-zurrunentzat tradizionalki bananduta kontsideratu dira. Hori dela eta, IRI portaera-modelo bat bide-zoru malguentzat eta IRI portaera-modelo bat bide-zoru erdi-zurrunentzat garatu nahi dira.

8.5.2. IRI portaera modeloak bide-zoru malguetako errepideetarako

IRI eboluzio modeloak bide-zoru malguetarako honako aldagai independenteak erabiltzen dira: ***Age***. Ez da ***R.Age***, ***TotBit***, $\boldsymbol{SS_{bit}}$, $\boldsymbol{SS_{tot}}$, $\boldsymbol{SUB_{cm}}$, ***AADT***, ***H.AADT***, ***TotVeh*** eta ***TotH.Veh***. $\boldsymbol{SS_{su}}$ ez da kontsideratu bide-zoru malguetan, material pikortatu berdina erabiltzen da: zagor artifiziala, eta horren ondorioz, bere balio SUB_{cm}.-ri proportzionala da. Aitzitik, SS_{tot} aldagaia sartzen da SS_{bit} et SS_{sub}-en konbinaziotik datorrelako.

Hasieran, mendeko aldagaiaren eta aldagai independenteen esplorazio-analisia gauzatzen da (8.12 eta 8.13 taula). Aldagai kuantitatiboek ez dute banaketa normalik jarraitzen (8.14 taula).

Aldagai independenteen eta mendeko aldagaiaren arteko korrelazioa konprobatu zen Pearson-en korrelazio-koefizientearen bitartez, *R*, korrelazioaren esangura adieraziz (8.15 taula). Mendeko aldagaiaren eta aldagai independente bakoitzaren arteko korrelazioa erakusten da, baina ez aldagai independenteen artekoa.

8.12 taula. Aldagaien esplorazio-analisia Trazatu Berria tarteetan bide-zoru malguarekin. Lehenengo partea

		IRI	Age	R.Age	TotBit	SUBcm	SSbit	SStot
Batez bestekoa		2,411	9,85	9,76	16,74	26,09	91829	98350
Errore estandarra		0,079	0,631	0,63	0,407	0,821	2125	2106
%95ko KTak	**Goiko**	2,255	8,6	8,51	15,94	24,46	87614	94173
	Beheko	2,567	11,1	11,01	17,55	27,71	96043	102527
Bariantza		0,650	41,803	41,85	17,404	70,81	474201099	465836058
Desbideratze tipikoa		0,806	6,466	6,47	4,172	8,415	21776	21583
Minimoa		1,114	0	0,1	8	12	44000	49000
Maximoa		5,475	26	26	33	55	192000	198250
Anplitudea		4,361	26	25,9	25	43	148000	149250
Kuartileko heina		1,130	9	8,9	8	0	30000	22500
Simetria		1,021	0,688	0,692	0,386	2,451	0,843	0,819
Kurtosia		0,986	-0,27	-0,268	1,379	6,301	3,653	3,923

8.13 taula. Aldagaien esplorazio-analisia Trazatu Berria tarteetan bide-zoru malguarekin.. Jarraipena.

		AADT	Age	TotVeh	TotH.Veh
Batez bestekoa		5937,1	242,7	19370,7	797,1
Errore estandarra		351,3	18,6	1731,8	77,0
%95ko KTak	**Goiko**	5240,4	205,9	15936,6	644,4
	Beheko	6633,8	279,5	22804,9	949,8
Bariantza		12960008,9	36160,3	314891873,3	622596,5
Desbideratze tipikoa		3600,0	190,2	17745,2	789,0
Minimoa		508	17	21,51	0,67
Maximoa		20284	1268	94659,69	3787,65
Anplitudea		19776	1251	94638,18	3786,98
Kuartileko heina		4100	237	21563,66	1037,02
Simetria		0,889	1,805	1,622	1,498
Kurtosia		2,015	6,795	3,264	2,451

8.14 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat Trazatu Berria tarteetan bide-zoru malguarekin.

Aldagaiak	**Kolmogorov-Smirnov**[a]			**Shapiro-Wilk**		
	Estatistikoa	**Askatasun-graduak**	**Esangura (p-balioa)**	**Estatistikoa**	**Askatasun-graduak**	**Esangura (p-balioa)**
IRI	0,114	105	0,002	0,924	105	< 0,001
Age	0,156	105	< 0,001	0,941	105	< 0,001
R.Age	0,15	105	< 0,001	0,941	105	< 0,001
TotBit	0,22	105	< 0,001	0,92	105	< 0,001
SUBcm	0,437	105	< 0,001	0,583	105	< 0,001
SSbit	0,18	105	< 0,001	0,921	105	< 0,001
SStot	0,175	105	< 0,001	0,921	105	< 0,001
AADT	0,081	105	0,085	0,94	105	< 0,001
H.AADT	0,118	105	0,001	0,865	105	< 0,001
TotVeh	0,147	105	< 0,001	0,856	105	< 0,001
TotH.Veh	0,156	105	< 0,001	0,85	105	< 0,001

[a] Lilliefors Esangura zuzenketa

8.15 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) Trazatu Berria tarteetan bide-zoru malguarekin.

Aldagai independenteak	**Korrelazioa IRI-ekin (R)**	**Korrelazioren esangura (bi aldekoa)**
Age	0,509	< 0,001
R.Age	0,512	< 0,001
TotBit	-0,531	< 0,001
SUBcm	0,319	0,001
SSbit	-0,475	< 0,001
SStot	-0,448	< 0,001
AADT	-0,024	0,810
H.AADT	-0,055	0,574
TotVeh	0,380	< 0,001
TotH.Veh	0,308	0,001

Ikusten den bezala, korrelaziorik onenak *Age*, *R.Age* eta *TotBit*-rekin lortzen dira. Korrelazioak SUB_{cm}, SS_{bit}, SS_{tot}, *TotVeh* eta *TotH.Veh*–rekin ere oso altuak dira. Aitzitik, *AADT* eta *H.ADDT* -k oso korrelazio baxuak erakusten dituzte, oso esangura edo adierazgarritasun baxuarekin. Hurrengo pausoan, aldagaien transformazio posibleak aztertu ziren. Mendeko aldagaia eta aldagai independente bakoitzaren arteko erlazioa hoberen doitzen duen kurben analisitik, aldagaien transformazio batzuk proposatzen dira. 8.16 taulak erakusten du aldagai independente bakoitzak eta IRI (mendeko aldagaia) korrelazio onena lortzen duten kurbak erakusten ditu. Batzuetan, kurba koadratikoa edo kubikoak hoberen doitzen du baina ez dute errepikatzen literaturan ikusi den patroia. Beste batzuetan, korrelazio linealaren eta besteen artekoak determinazio-koefizientean hobekuntza oso txikia bada ($\Delta R^2 < 0{,}05$), modelo lineala mantendu da, aldagai independentea ez dela transformatu adieraziz.

8.16 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duen ekuazioak Trazatu Berria tarteetan bide-zoru malguarekin.

Aldagai independente	Ekuazio-mota	Modeloaren laburpena					Parametroen estimazioak			
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
Age	Potentziala	0,277	39,494	1	103	< 0,001	1,778	0,026		
R.Age	Esponentziala	0,28	40,019	1	103	< 0,001	1,78	0,026		
TotBit	Logaritmikoa	0,304	45,022	1	103	< 0,001	7,153	-1,702		
SUBcm	Lineala	0,101	11,634	1	103	0,001	1,615	0,031		
SSbit	Logaritmikoa	0,257	35,653	1	103	< 0,001	21,744	-1,696		
SStot	Logaritmikoa	0,225	29,842	1	103	< 0,001	22,038	-1,711		
AADT	Alderantzizkoa	0,079	8,891	1	103	0,004	2,231	527,196		
H.AADT	Alderantzizkoa	0,088	9,997	1	103	0,002	2,23	17,713		
TotVeh	Koadratikoa	0,228	15,072	2	102	< 0,001	2,364	-1,43E-05	4,71E-10	
TotH.Veh	Koadratikoa	0,147	8,816	2	102	< 0,001	2,354	0	2,05E-07	

8.14 taulan, hainbat eraldaketa egin daitezke aldagai independente bakoitzaren eta IRI-en (mendeko aldagaia) arteko korrelazioa hobetzeko. *Age*-k hobetzen du bere korrelazioa eraldaketa potentzial baten bitartez, baina *R.Ag*e erakusten du korrelazio hobeago bat eraldaketa esponentzial baten bidez, 8.17 ekuazioaren itxurarekin. *Age* ezin da eraldatu transformazio esponentzial baten bidez, trazatu berria datu-biltzearen urte beran gauzatu bazen, bere balioa 0 da. Horren ondorioz, *IRI* eta adina erlazionatzeko, *R.Age* erabiltzea nahiago izan da, adinaren balio zehatzagoa dena.

$$IRI = 1{,}78 \cdot e^{0{,}026 \cdot R.Age} \qquad [8.17]$$

TotBit, SS_{bit} eta SS_{tot} aldagaietarako transformazio logaritmikoa proposatzen da R-ren balioa hobetzeko. AADT eta H.AADT, alderantzizko eraldaketa gomendatzen da, baina balioa ez da asko hobetzen eta erlazio baxuenak dituzten erlazioak dira ($p = 0{,}02$ and $p = 0{,}04$). *TotVeh* eta *TotH.Veh* aldagaietarako, transformazio koadratikoa aplikatzen da, puntuen dispertsio-grafikoa ikusita, kasu bietan datuak doitzeko kurba apropos da. Beraz, aldagaiak eraldatu ziren 8.14 taulan azaltzen den moduan. IRI eta *R.Age* aldagaien arteko erlazioa 8.17 ekuazioa jarraitzen duenez eta aldagai independentea (*R.Age*) ez eraldatzeko, IRI mendeko aldagai eraldatzea nahiago izan da logaritmo natural baten bidez, 8.18 ekuazioan azaltzen den erlazioa garatuz.

$$LnIRI = A + B \cdot R.Age \qquad [8.18]$$

IRI eta aldagai independenteen arteko erlazioa lortu nahi denez, 8.17 ekuazioaren egitura jarraituz, aurretik eraldatutako aldagai independenteak eraldatzea derrigorrezkoa da logaritmo naturalaren bidez, 8.19 ekuazioaren itxurako erlazioa sortzeko:

$$\mathrm{Ln}IRI = A + B * \mathrm{R.Age} + C * \mathrm{Ln}(\textit{Eraldatutako aldagaia 1}) + D * \mathrm{Ln}(\textit{Eraldatutako aldagaia 2}) \quad [8.19]$$

IRI portaerarako ekuazioa garatua denean, 8.19 ekuazioan azaltzen den bezala, 8.20 ekuazioa bezalako adierazpen ohiko batera eraldatuko litzateke

$$IRI = A' + B' * \mathrm{e}^{(B''*R.Age)} + C' * \textit{Eraldatutako aldagaia 1} + D' * \textit{Eraldatutako aldagaia 2} \quad [8.20]$$

Eraldatutako aldagaiei logaritmo naturala aplikatu ondoren, lortutako korrelazioak mendeko aldagai berriarekin, Ln*IRI*, 8.17 taulan erakusten dira.

8.17 taula. Korrelazioa eraldatutako mendeko aldagaiaren (LnIRI) eta eraldatutako aldagai independenteen artean (8.16 taulan proposatzen den moduan)(Pearson-en R koefizientea) Trazatu Berria tarteetan bide-zoru malguetan.

Aldagai independenteak	Korrelazioa IRI-ekin (R)	Korrelazioren esangura (bi aldekoa)
Age	0,526	< 0,001
R.Age	0,529	< 0,001
Ln(LnTotBit)	-0,550	< 0,001
Ln(SUB1cm)	0,218	0,026
Ln(LnSSbit)	-0,508	< 0,001
Ln(LnSStot)	-0,478	< 0,001
Ln(TotVeh^2)	0,242	0,013
Ln(TotH.Veh^2)	0,218	0,026

Mendeko aldagaia LnIRI da

Ikusten den bezala, IRI eta aldagai independenteen arteko erlazioa hobetzen da, Ln(*TotVeh*^2) eta Ln(*TotH.Veh*^2)-rako salbu. Beraz, metatutako trafikoarekin erlazionatutako aldagaiak eraldatu gabe beharrezkoa da.

Eraldatutako aldagaiekin eta Ln*IRI* mendeko aldagaiarekin, erregresio lineal anizkoitzaren modeloa, SPSS v24 programaren Pausoz Pauso Aurrera funtzioaren bitartez, kalkulatu zen. Ekuazio bat adierazi du, 8.21 ekuazioa, esangura globala duena (*F* testaren p-balioa 0,05 baino txikiagoa da) eta aldagai guztiak esanguratsuak dira indibidualki (*t* testaren p-balio 0,05 baino txikiagoa da).

$$LnIRI = Int + B \cdot R.Age + C \cdot Ln(LnTotBit) \quad [8.21]$$

Proposatutako modeloaren Bariantzaren analisi osoa 8.18 taulan erakusten da. Determinazio koefizientearen balioa (R^2) 0,419 da eta doitutako R^2 0,408 da.

8.18 taula. SPSS programaren Pausoz Pauso Hautaketa funtzioak proposatutako erregresio lineal anizkoitzaren modeloaren bariantzaren analisia (ANOVA) Trazatu Berria tarteetan bide-zoru malguetan.

Bariantzaren analisia						
Iturria	**Karratuen batuketa**	**Askatasun-graduak**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**
Modeloa	4,373	2	2,186	36,797	< 0,001	1,260
Errorea	6,061	102	0,059			
Total zuzenduta	10,434	104				
Estimazioaren errore estandarra		**R**	**R^2**	**R^2_{adj}**	**Kolinealtasunaren estatistikoak**	
					Tolerantzia	**VIF**
0,24376		0,647	0,419	0,408	0,847	1,180
Parametroen estimazioak						
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioa**	**p-balioa**	**%95eko konfiantza-tarteak**	
Konstantea	2,005	0,290	6,922	< 0,001	1,430	2,579
R.Age	0,018	0,004	4,533	< 0,001	0,010	0,026
Ln(LnTotBit)	-1,328	0,266	-4,993	< 0,001	-1,855	-0,800

Ln*IRI* mendeko aldagai bezala mantenduz, modelo posible gehiago probatu ziren determinazio-koefiziente (R^2) hobeagoa lortzeko helburuarekin, baldin eta erlazioa esanguratsua den (*F* testaren *p*-balioa 0,05 baino txikiagoa) eta sartutako aldagai guztien koefizienteak esanguratsuak diren (*t* testaren *p*-balioa 0,05 baino txikiagoa). Aldagai eskuragarrien hainbat konbinazio egiaztatzen dira. Erregresio lineala anizkoitzaren analisiaren beste suposaketak (kolinealtasuna, homoszedatizitatea, etab.) ere egiaztatzen dira modeloaren egokitasuna jakiteko. 8.19 taulak aurkezten du analizatutako modelo batzuk, sartutako aldagaiak, lortutako R^2 eta R^2_{adj} (doitutako R^2) eta modeloei buruzko komentario eta ohar batzuk adieraziz.

8.19 taula. Proposatutako erregresio lineal anizkoitzeko modeloak IRI portaerarako Trazatu Berria tarteetan bide-zoru malguetan mendeko aldagaia LnIRI izanik

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
LnIRI = Int + R.Age + Ln(LnSStot) + Ln(TotH.Veh^2)	0,378	0,360	Ln(TotH.Veh^2)-ren esangura baxua (p = 0,823)
LnIRI = Int + R.Age + Ln(LnSStot) + Ln(TotVeh^2)	0,378	0,359	Ln(TotVeh^2) -ren esangura baxua (p = 0,918)
LnIRI = Int + R.Age + Ln(LnSSbit) + Ln(TotH.Veh^2)	0,391	0,373	Ln(TotH.Veh^2) -ren esangura baxua (p = 0,780)
LnIRI = Int + R.Age + Ln(LnSSbit) + Ln(TotVeh^2)	0,391	0,372	Ln(TotVeh^2) -ren esangura baxua (p = 0,941)
LnIRI = Int + R.Age + Ln(LnTotBit) + Ln(TotVeh^2)	0,420	0,403	Ln(TotH.Veh^2) -ren esangura baxua (p = 0,612)
LnIRI = Int + R.Age + Ln(LnTotBit) + Ln(TotH.Veh^2)	0,423	0,406	Ln(TotH.Veh^2) -ren esangura baxua (p = 0,356)

Note: Int: Konstantea

Ikusten den gisa, Ln(Ln*TotBit*) aldagaiak Ln(LnSS_{tot}) edo Ln(LnSS_{bit}) aldagaiek baino korrelazioa hobeagoa ematen du. Beraz, geruza bituminosoen lodiera osoa egiturako erresistentzia islatzen duen proposatutako aldagaiek baino hobeto erlazionatzen da lortutako IRI-ekin. Gainera, Ln(LnSS_{bit}), azpi-oinarriko geruzaren lodiera osoa kontuan hartzen duen parametroak are korrelazio txarragoa. Ondorioztatu ahal da geruza bituminosoak bide-zoruaren egituraren erresistentziaren arduradunak direla eta material pikortatuak oso gutxi kolaboratzen du, IRI portaeran eraginik ez duen faktorea izanik.

8.19 taularen azken modeloak lortu ahal den modelorik onena adierazten du, Ln(*TotH.Veh^2*) aldagaiaren esangura baxua gora behera (p = 0,356). Mendeko aldagaia IRI-a den modelo bat aurkezteko, *R.Age* kurba onenak jarraituz eraldatu zen, 8.17 ekuazioan azaldutakoa. Eraldatutako aldagai honekin, *ExpR.Age* izenekoa, erregresio lineal anizkoitzeko modelo gehiago analizatu ziren, 8.20 taulan erakutsitakoak.

8.20 taula Analizatutako erregresio lineal anizkoitzeko modeloak IRI portaerarako Trazatu Berria tarteetan bide-zoru malguetan mendeko aldagaia IRI izanik

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
IRI = Int + ExpR.Age + LnSStot + TotVeh^2	0,410	0,392	Aldagai guztiak esanguratsuak dira (p < 0,05) CI = 145
IRI = Int + ExpR.Age + LnSStot + TotH.Veh^2	0,394	0,376	Aldagai guztiak esanguratsuak dira (p < 0,03) CI = 148
IRI = Int + ExpR.Age + LnSSbit + TotH.Veh^2	0,405	0,380	Aldagai guztiak esanguratsuak dira (p < 0,03) CI = 138
IRI = Int + ExpR.Age + LnSSbit + TotVeh^2	0,415	0,398	Aldagai guztiak esanguratsuak dira (p < 0,05) CI = 135
IRI = Int + ExpR.Age + LnTotbit + TotVeh	0,411	0,393	TotVeh-ren esangura ertaina (p=0,18) CI = 39
IRI = Int + ExpR.Age + LnTotbit + TotH.Veh	0,416	0,398	TotH.Veh-ren esangura ertaina (p=0,1) CI=41
IRI = Int + ExpR.Age + LnTotBit + TotVeh^2	0,442	0,425	Aldagai guztiak esanguratsuak dira (p < 0,06)
IRI = Int + ExpR.Age + LnTotBit + TotH.Veh^2	0,437	0,420	Aldagai guztiak esanguratsuak dira (p < 0,05)

Note: Int: Konstantea. CI = Condition Index

8.20 taularen modeloen laburpenetik, *ExpR.Age* beti esanguratsua dela ikusi ahal da, Ln*TotBit* aldagaiarekin LnSS_{tot} edo LnSS_{bit} aldagaiekin baino korrelazio hobeagoak lortu ahal dira ikus daiteke. Gainera, zerbitzuan jarri zen momentutik pasatu diren ibilgailu total eta ibilgailu astunen kopuru metatua esanguratsua da transformazio koadratikoarekin. Lehenengo bi aldagaian finkoak bezala hartuta, *ExpR.Ag*e eta Ln*TotBit,* trafiko metatutako aldagaien hainbat konbinazio probatu dira eta analisiaren laburpena 8.21 taulan erakusten da.

8.20 taula eta 8.21 taula konparatzen denean, ondorioztatu ahal da taula bakoitzeko modelorik onenak taula bakoitzean agertzen diren azkenak dira. 8.20 taulako azken modeloak erabiltzen du sekzioa zeharkatu duten ibilgailu astunen kopuru metatua eta bere determinazio-koefizientea (R^2) 0,437 da, 8.21 taulako azken modeloak sekzioa zeharkatu duten ibilgailu kopuru metatua (edozein motako ibilgailua) erabiltzen du eta bere determinazio-koefizientea (R^2) 0,472 da, iragartze hobeagoa ematen.

8.21 taula Analizatutako erregresio lineal anizkoitzeko modeloak IRI portaerarako Trazatu Berria tarteetan bide-zoru malguetan mendeko aldagaia IRI izanik trafiko metatuaren aldagaien hainbat konbinaziorekin.

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotH.Veh^2	0,442	0,420	Esangura baxua,TotVeh^2 (p=0,33), TotH.Veh (p=0,741)
IRI = Int + ExpR.Age + LnSStot + TotVeh^2 + TotH.Veh^2	0,410	0,386	Esangura baxua,TotVeh^2 (p=0,107), TotH.Veh (p=0,860)
IRI = Int + ExpR.Age + LnSSbit + TotVeh^2 + TotH.Veh^2	0,415	0,392	Esangura baxua TotH.Veh (p=0,986)
IRI = Int + ExpR.Age + LnTotbit + SUB1cm + TotVeh^2 + TotH.Veh^2	0,444	0,415	Esangura baxua,TotVeh^2 (p=0,554), TotH.Veh (p=0,646), SUB1cm (p=0,627)
IRI = Int + ExpR.Age + TotVeh^2 + TotH.Veh^2	0,308	0,287	Esangura ertaina TotH.Veh^2 (p=0,151)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotH.Veh	0,445	0,423	Esangura baxua,TotH.Veh (p=0,407)
IRI = Int + ExpR.Age + LnTotbit + TotVeh + TotH.Veh	0,416	0,392	Esangura baxua,TotVeh (p=0,363), TotH.Veh (p=0,874)
IRI = Int + ExpR.Age + LnTotbit + TotVeh + TotH.Veh^2	0,443	0,421	Esangura baxua,TotVeh (p=0,303)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotH.Veh^2 + TotVeh + TotH.Veh	0,480	0,448	Esangura baxua,TotH.Veh^2 (p=0,397), TotH.Veh (p=0,256)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotVeh + TotH.Veh	0,476	0,449	Esangura baxua, TotH.Veh (p=0,410)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotVeh	0,472	0,451	Aldagai guztiak esanguratsuak dira ($p < 0,05$)

Oharra: Int: Konstantea. CI = Condition Index

Tradizionalki, IRI portaera-modelo deterministakoek ibilgailu astunen kopuru metatua erabiltzen dute aldagai bezala zeren eta ibilgailu astunak bide-zoruaren hondatzearen arduradunak direla leporatzen zaielako haien pisu handiagogatik, bide-zoruaren gainean karga handiagoak aplikatuz. *ESAL* (*Equivalent Single Axle Load*) parametroa erabiltzeak ibilgailu bakoitzaren baliokidetasuna islatu nahi du *ESAL* kopuru bezala, ibilgailu bakoitzaren eragina bide-zoruaren hondatzean kalkulatzeko (8.22 ekuazioa).

$$ESAL_{128} = K \cdot (P_i/128)^{\alpha} \qquad [8.22]$$

Non $ESAL_{128}$ kN-eko (edo 13 t-ko) Ardatz Sinple baten Karga Baliokidea den eta adierazten du 128 kN (edo 13t-ko) ardatz sinpleren kopuruak, P_i (kN-tan) karga totala duen ardatz batek sortuko duen kalte berdina; P_i da ardatz bakoitzeko pisua, K ardatz sinplerako 1 da, 1,4 ardatz bikoitzetarako eta 2,3 ardatz hirukoitzetarako eta α i 4 da bide-zoru malguetan eta 8 bide-zoru zurrunetan.

ESAL-en bidez ondorioztatu ahal da ibilgailu arinek ez dute sortzen kalte handiak bide-zoruan. Ibilgailu astun baten efektua ibilgailu arinen mila batzuek sortuko dutenaren antzekoa da.

IRI eta ibilgailu guztien bolumen metatuaren arteko korrelazio hobeagoaren azalpena izan daiteke trafikoaren aldagai guztien arteko korrelazioa (*TotVeh* and *TotH.Veh* eta haien transformazio koadratikoak) (8.22 taula). Horrek egiten du posible batak bestea ordezkatu ahal izatea. Hala ere, 8.21 taulan ikusi den bezala, ibilgailu totalen eta ibilgailu astunen bolumenak sartzen badira, ibilgailu astunen aldagaia ez da esanguratsua. Aldagaien arteko korrelazio altu hau sortzen da errepideetako ibilgailu astunen portzentajeak antzekoak direlako, trafiko totalaren %5 eta 8aren artean dagoelarik. Horrez gain, behaketen kopuru totala (105) ez da handia, eta horrek ere eragina dauka IRI eta ibilgailu totalen arteko korrelazio hobeagoan.

8.22 taula. Trafiko metatuaren aldagaien arteko korrelazioa (Pearson-en R koefizientea)

	TotVeh	TotH.Veh	TotVeh^2	TotH.Veh^2
TotVeh	1	0,925	0,857	0,923
TotH.Veh	0,925	1	0,924	0,843
TotVeh^2	0,857	0,924	1	0,910
TotH.Veh^2	0,923	0,843	0,910	1

Horren ondorioz, nahiz eta ibilgailu guztien kopuru metatua erabiltzen duen ekuazioak korrelazio hobeagoa eman, ibilgailu astunen kopuru metatua daukan ekuazioa aukeratu da IRI eboluzioa iragartzeko (8.23 ekuazioa):

$$IRI = 0{,}680 \cdot e^{(0{,}026 \cdot R.Age)} - 1{,}411 \cdot LnTotBit + 0{,}079 \cdot 10^{-6} \cdot VehTotPes^{2} \qquad [8.23]$$

Non:

IRI : iragarritako batez besteko IRI balio (m/km), adina, trafikoa eta geruza bituminosoen lodierako aldagaien balio bereak dituen tarterako bide-zoru malgu berrietan

R.Age : bide-zoruaren adina, zerbitzuan jarri zen data zehatzetik kalkulatu nahi den momentura arte, urtetan, frakzio dezimaletan azalduta, non 0,5 jartzeak adierazi nahi du 6 hilabete.

TotBit: bide-zoru malguan dagoen geruza bituminosoen lodiera totala, cm-tan.

VehH.Tot:Kontsideratutako denbora-tartean sekziotik igarotako ibilgailu astunen kopuru metatua proiektuko erreitik (galtzadan ibilgailu astun gehiago igarotzen diren erreian) ireki zenetik kalkulatu nahi den momentura arte. Orokorrean, noranzko biek astunen trafiko berdina daukate, totalaren erdia.

Mota honetako ekuazioa beste herrialde eta beste eskualdeetako errepideetan erabil daitekeela pentsatzen da erregulartasunaren eboluzioan eragina duten oinarrizko faktoreek osatzen baitute. Alde batetik, azpi-oinarriaren geruzaren materiala berea da bide-zoru malguetan, material pikortatua, bide-zoruaren egiturazko erresistentzia geruza bituminosoek proportzionatzen dute. Beraz, geruza bituminosoen lodiera totalak erregulartasunaren progresioan efektu negatiboa dauka. Beste aldetik, bide-zorua hondatzen duten ohiko kanpoko faktoreak adina, martxan jarri zen momentutik iragarri nahi den momentura arte, eta proiektuko erreitik igaro diren ibilgailu kopuru metatua. 8.23, 8.24, 8.25 eta 8.26 taulak eta 8.6 eta 8.7 irudiek modeloaren analisi estatistiko osoa erakusten dute.

8.23 taula. 8.23 ekuazioaren Bariantzaren analisia (ANOVA) Trazatu Berria tarteetan bide-zoru malguetan errepide konbentzionaletan.

Bariantzaren analisia								
Iturria	**Karratuen batuketa**	**Askatasun-graduak**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**	**Estimazioaren errore estandarra**	**R**
Modeloa	29,509	3	9,836	26,114	<0,001	1,448	0,6137	0,661
Errorea	38,043	101	0,377				**R^2**	**Adj. R^2**
Total zuzenduta	67,552	104					0,437	0,420

Parametroen estimazioak							Kolinealtasunaren estatistikoak	
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioae**	**p-balioa**	**%95eko konfiantza-tarteak**		**Tolerantzia**	**VIF**
Intercept	5,353	0,968	5,531	<0,001	3,433	7,272		
ExpR.Age	0,382	0,19	2,011	0,047	0,005	0,759	0,584	1,714
LnTotBit	-1,411	0,254	-5,544	<0,001	-1,916	-0,906	0,82	1,219
TotH.Veh^2	0,079	0,031	2,568	0,012	0,018	0,141	0,685	1,459

8.24 taula. 8.23 ekuazioaren modeloaren aldagai independenteen arteko Pearson-en R korrelazio-koefizienteak

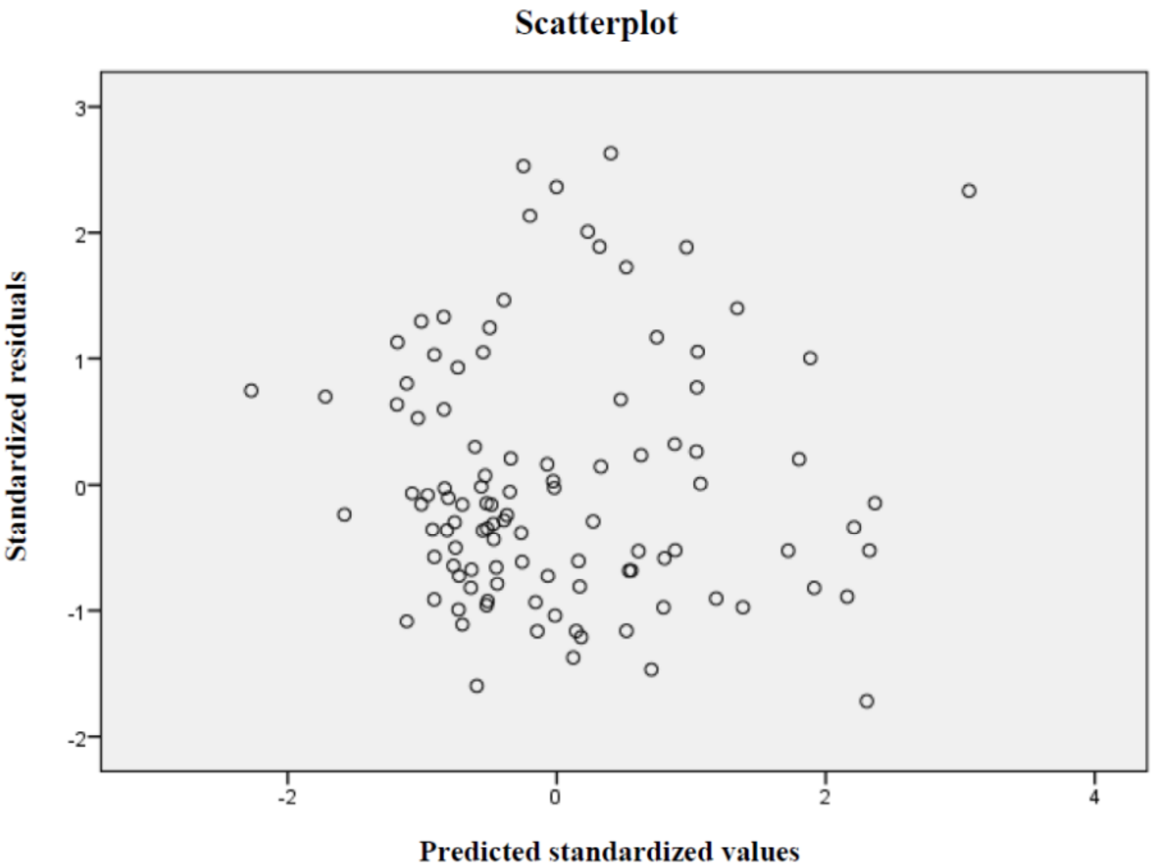
		VehH.Tot^2	LnTotBit	ExpR.Age
Korrelazioak	VehH.Tot^2	1	-0,177	-0,557
	LnTotBit	-0,177	1	0,419
	ExpR.Age	-0,557	0,419	1
Kobariantzak	VehH.Tot^2	0,001	-0,001	-0,003
	LnTotBit	-0,001	0,065	0,02
	ExpR.Age	-0,003	0,02	0,036

8.25 taula. 8.23 ekuazioaren kolinealtasunaren diagnosia

Dimentsioa	**Balio propioa**	**Correlation index**	**Bariantzaren proportzioak**			
			Konstantea	**VehTot^2**	**LnTotBit**	**ExpR.Age**
1	3,29	1	0	0	0	0,02
2	0,686	2,19	0	0	0	0,67
3	0,022	12,312	0	0,51	0,11	0,2
4	0,002	36,842	1	0,49	0,89	0,11

8.26 taula. 8.23 ekuazioaren erroreen estatistikoak

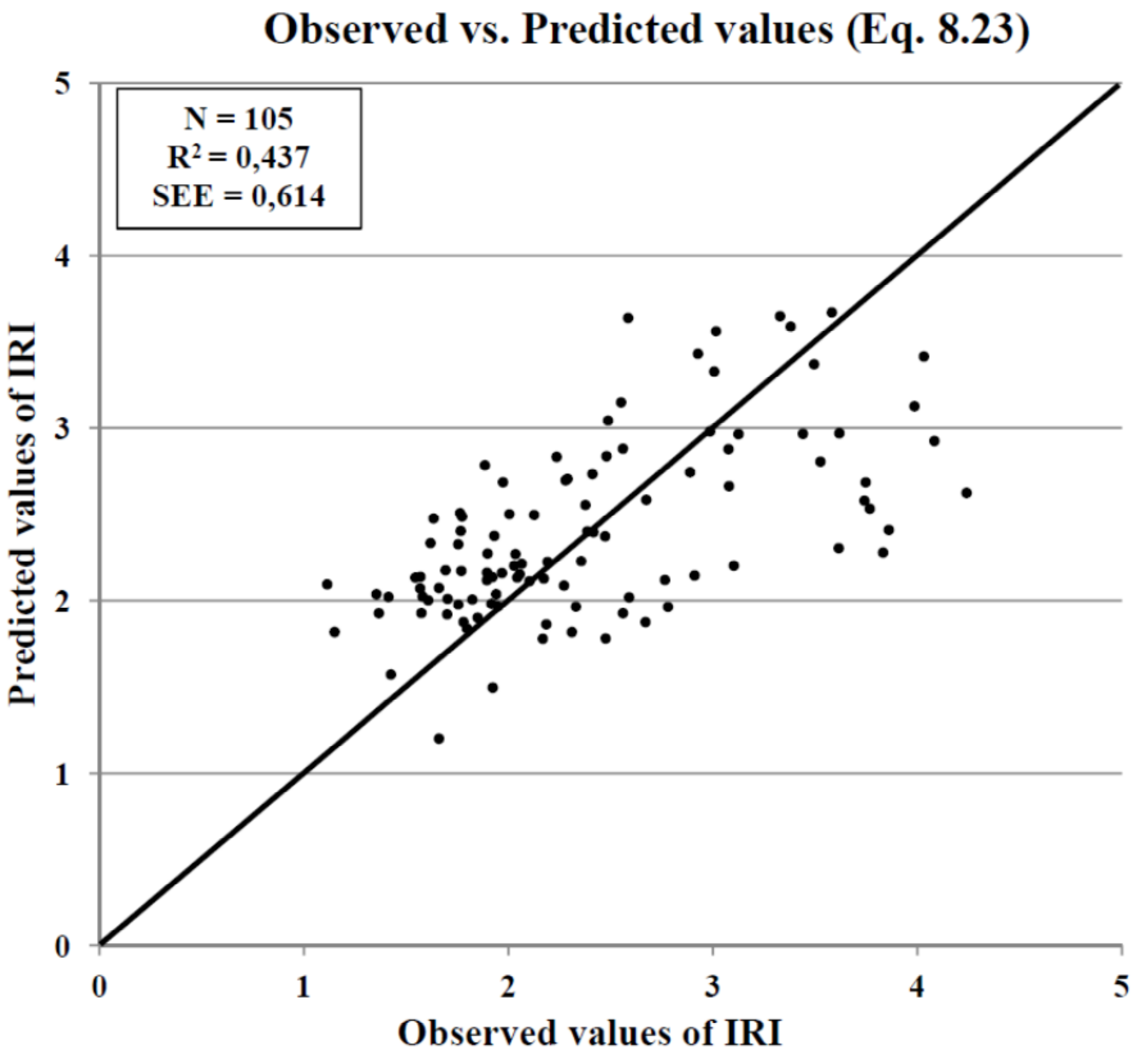
	Minimoa	Maximoa	Batez bestekoa	Desbideratze estandarra	N
Iragarritako balioa	1,201	4,044	2,411	0,533	105
Errorea	-1,054	1,615	0,000	0,605	105
Iragarritako balio estandarra	-2,271	3,065	0,000	1,000	105
Errore estandarra	-1,718	2,631	0,000	0,985	105



8.6 irudia. 8.23 ekuazioaren iragarritako balio estandarrak iragarritako errore estandarrekiko grafikoa.

Erregresio lineal anizkoitzeko modelo batek bete behar dituen hipotesiak, 7.5.3 atalean azaldutakoak, egiaztatzen dira. Doitutako determinazio-koefizientea 0,420 da. Ikusten den bezala, *F* testak 0,001 baino txikiagoko *p*-balio bat ematen du (8.23 taula), proposatutako erlazioa benetakoa den adieraziz. Parametroen koefizienteen Student-en *t* testek erakusten dute esanguratsuak direla eta ez direla nulua (%95eko adierazgarritasuneko konfiantza-tarteek ez dute 0 sartzen). Durbin-Watson estatistikoa 1,448 da, erroreen independentzia egiaztatuz, ez dago auto-korrelaziorik. Homoszedastizitatea 8.6 irudian egiaztatzen da, non ez dira patroirik nabaritzen. *Condition Index* bat 30 baino handiagoa da (8.25 taula), multikolinealtasunaren arazoak adierazi ahal dituena. Hala eta guztiz ere, modeloa azaltzen duten aldagaien arteko korrelazio handirik ez dago (*Age* eta *TotH.Veh* erlazionatuta egon daitezke, baina haien Pearson-en R korrelazio-koefizientea -0,557 da, eta Bariantzaren Inflazioaren Faktorea (*Variane Inflation Factor*) baxuak dira (VIF <

10). Horren ondorioz, ondorioztatu ahal da ez dagoela multikolinealtasuna. Erroreen banaketa normala da, haien batez bestekoa 0 da eta haien bariantza 1etik hurbil dago (8.23 taula). 8.7 irudiak erakusten du neurtutako balioak eta iragarritako balioak, puntuak diagonaletik hurbil daudela erakutsiz.



8.7 irudia. Neurtutako datuak eta iragarritako datuak 8.23 ekuazioren bidez.

8.5.3. IRI portaera-modeloak bide-zoru erdi-zurrunetarako

IRI iragartzeko Trazatu Berria tarteetan bide-zoru erdi-zurrunekin errepide konbentzionaletan, 81 behaketa eskuragarri daude Bizkaiko Kontserbazio Eremu 1, 2 eta 3tik (8.9 taula). Kasu honetan SS_{sub} sartzen da SUB_{cm},-ren ostean, bide-zoru erdi-zurrun batean bi geruza egon daitezkeelako.

Lehenengo eta behin, aldagaien esplorazio-analisia gauzatzen da (8.27 eta 8.28 taula). 8.29 taulak adierazten du mendeko aldagaiaren (IRI) banaketa normala duela.

8.27 taula.. Aldagai kuantitatiboen (mendekoaren eta independenteen) esplorazio-analisia Trazatu Berria tarteetan bide-zoru erdi-zurrunekin. Lehenengo partea.

		IRI	Age	R.Age	TotBit	SSsub	SSbit	SStot
Batez bestekoa		2,123	7,11	6,898	13,840	398247	76840	475086
Errore estandarra		0,057	0,603	0,601	0,376	20262	1846	20541
%95ko KTak	**Goiko**	2,009	5,91	5,703	13,09	357925	73166	434208
	Beheko	2,237	8,31	8,093	14,59	438569	80513	515965
Bariantza		0,267	29,45	29,214	11,461	33253932020	276036420	34177298690
Desbideratze tipikoa		0,517	5,427	5,405	3,385	182357	16614	184871
Minimoa		1,075	1	0,7	8	205000	48000	268000
Maximoa		3,615	29	28,6	25	748000	131000	844000
Anplitudea		2,540	28	27,9	17	543000	83000	576000
Kuartileko heina		0,773	8	8,13	3	388000	26000	379000
Simetria		0,416	1,413	1,399	0,817	0,635	0,537	0,535
Kurtosia		-0,045	2,897	2,796	0,756	-1,206	0,086	-1,252

8.28 taula. Aldagai kuantitatiboen (mendekoaren eta independenteen) esplorazio-analisia Trazatu Berria tarteetan bide-zoru erdi-zurrunekin. Jarraipena

		AADT	Age	TotVeh	TotH.Veh
Mean		9378,62	581,12	18804,02	1182,07
Standard error		865,457	58,56	2127,83	148,17
%95ko KTak	**Beheko**	7656,3	464,59	14570	887
	Goiko	11100,93	697,66	23039	1477
Bariantza		60670231,49	277767,14	366739978	1778322
Desbideratze tipikoa		7789,11	527,04	19150	1334
Minimoa		911	19	1012,56	20,53
Maximoa		36185	2533	102496,75	6957,42
Anplitudea		35274	2514	101484,18	6936,89
Kuartileko heina		9905	775	20912,8	1352,8
Simetria		1,405	1,195	1,914	1,888
Kurtosia		1,838	1,739	4,481	4,125

Mendeko aldagaia (IRI) eta aldagai independente bakoitzaren arteko korrelazio aztertzen da Pearson-en *R* korrelazio-koefizientearen arabera (8.30 taula). Bide-zoru malguetan bezala, *Age*, *R.Age* eta *TotBit* aldagaiek korrelazio ona daukate IRI-arekiko. Berriro ere, *TotBit* aldagaiak SS_{bit} aldagaiak baino korrelazio hobeagoa dauka, nahiz eta azken honek material bituminosoen arteko diferentziak hobeto islatzen duen. SS_{sub} eta SS_{tot} aldagaiek are korrelazio baxuagoak daukate, esanguratsuak ez direnak (*p*-balioa > 0,05). Kasu honetan *AADT* eta *H.AADT* aldagaiek korrelazio altua daukate eta, beste aldetik, *TotVeh* eta *TotH.Veh* aldagaiak korrelazio baxuenak (R < 0,10).

8.29 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat Trazatu Berria tarteetan bide-zoru erdi-zurrunekin

Aldagaiak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)
IRI	0,082	81	0,200*	0,979	81	0,192
Age	0,13	81	0,002	0,878	81	< 0,001
R.Age	0,126	81	0,003	0,883	81	< 0,001
TotBit	0,215	81	< 0,001	0,91	81	< 0,001
SUB1cm	0,158	81	< 0,001	0,94	81	0,001
SSbit	0,263	81	< 0,001	0,827	81	< 0,001
SStot	0,222	81	< 0,001	0,86	81	< 0,001
AADT	0,164	81	< 0,001	0,854	81	< 0,001
H.AADT	0,177	81	< 0,001	0,862	81	< 0,001
TotVeh	0,185	81	< 0,001	0,802	81	< 0,001
TotH.Veh	0,192	81	< 0,001	0,79	81	< 0,001

[a] Lilliefors Esangura zuzenketa

* Hau da beheko muga benetako esangurarako

8.30 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) Trazatu Berria tarteetan bide-zoru erdi-zurrunekin

Aldagai independenteak	Korrelazioa IRI-ekin (R)	Korrelazioren esangura (bi aldekoa)
Age	0,388	< 0,001
R.Age	0,390	< 0,001
TotBit	-0,384	< 0,001
SSbit	-0,302	0,006
SSsub	-0,134	0,231
SStot	-0,160	0,154
AADT	-0,407	< 0,001
H.AADT	-0,299	< 0,001
TotVeh	-0,072	0,525
TotH.Veh	-0,003	0,979

Aldagai independente bakoitza hobeto doitzen duen kurkak lortu ziren, korrelazioa hobetzeko transformazio posibleak jakiteko asmoz (8.31 taula).

8.31 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duen ekuazioak Trazatu Berria tarteetan bide-zoru erdi-zurrunetan.

Aldagai independente	Ekuazio-mota	Modeloaren laburpena					Parametroen estimazioak			
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
R.Age	Potentziala	0,234	24,117	1	79	< 0,001	1,668	0,133		
TotBit	Lineala	0,148	13,671	1	79	< 0,001	2,934			
Sssub	Lineala	0,018	1,455	1	79	0,231	2,275	-3,81E-07		
SSbit	Lineala	0,091	7,918	1	79	0,006	2,844	-9,38E-06		
SStot	Lineala	0,026	2,07	1	79	0,154	2,335	-4,47E-07		
AADT	Lineala	0,166	15,72	1	79	< 0,001	2,376	-2,70E-05		
H.AADT	Lineala	0,089	7,743	1	79	0,007	2,293	0		
TotVeh	Koadratikoa	0,026	1,042	2	78	0,358	2,075	7,37E-09	-1,27E-16	
TotH.Veh	Koadratikoa	0,003	0,132	2	78	0,877	2,096	5,19E-05		

8.31 taulan adierazitako eraldakuntzak gauzatu dira. *R.Age* aldagaia eraldatu zen ekuazio potentzial baten bidez, 8.24 ekuazioan erakusten den gisa:

$$PotR.Age = 1{,}668 \cdot R.Age^{0{,}133} \quad [8.24]$$

PotR.Age aldagai berriarekin eta eraldatutako $TotVeh^2$ eta $TotH.Veh^2$ aldagaiekin erregresio lineal anizkoitzeko modelo saiatu zen SPSS v24 programaren Pausoz Pauso Hautaketa funtzioaren bitartez. Lortutako soluzioa 8.25 ekuazioaren aldagaiak erabiltzen zituen. Modeloaren balioak 8.32 taulan erakusten dira.

$$IRI = PotR.Age + VehTot + TotBit \quad [8.25]$$

Erregresio lineal anizkoitzeko modelo gehiago frogatu ziren determinazio-koefiziente (R^2) hobeago bat lortzeko, baina egiaztatuz erlazioa esanguratsua dela, sartutako aldagaien koefizienteak esanguratsuak direla eta erregresio lineal anizkoitzeko modeloaren hipotesi guztiak betetzen direla (7. kapitulua). 8.33 taulak erakusten du analizatutako modelo interesgarrienen laburpena.

Ikusten den bezala, badirudi proposatutako SS_{bit}, SS_{sub} eta SS_{tot} aldagaiak ez dute islatzen geruzen ezaugarri ezberdinak. Beharbada, Young-en modulua ez da parametro onenik SNC antzeko parametro bat kalkulatzeko edo beste aldetik, hartutako moduluak ez dira zuzenak. Hala ere, bide-zoru erdi-zurrunen egon ahal diren geruzek ezaugarri ezberdinak dituzte eta honek IRI portaeran eragina izan behar du. Hori del eta, Erregresio Lineal Orokor Anizkoitzeko (GLM) modelo batzuk frogatu ziren. Bertan, azpi-oinarriaren geruza posibleak faktore bezala sartu ziren. Ezin dira faktore bakarra bezala sartu azpi-oinarrian bi geruza zabaldu daitezkeelako. Horren ondorioz, honako maila hauek SUB1 eta SUB2 faktoreetarako sortu ziren, datu-basean agertzen diren materialen arabera (8.34 taula).

8.32 taula. 8.25 ekuazioaren erregresio lineal anizkoitzaren modeloaren bariantzaren analisia (ANOVA) Trazatu Berria tarteetan bide-zoru erdi-zurrunetan

Bariantzaren analisia								
Iturria	**Karratuen batuketa**	**Askatasun-graduak**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**	**Estimazioaren errore estandarra**	**R**
Modeloa	8,089	3	2,696	15,657	< 0,001	1,602	0,415	0,616
Errorea	13,261	77	0,172				**R^2**	**R^2_{adj}**
Total zuzenduta	21,350	80					0,379	0,355

Parametroen estimazioak							Kolinealtasunaren estatistikoak	
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioa**	**p-balioa**	**%95eko konfiantza-tarteak**		**Tolerantzia**	**VIF**
Konstantea	0,090	0,567	0,158	0,875	-1,040	1,219		
PotR.Age	1,256	0,235	5,345	< 0,001	0,788	1,724	0,652	1,534
TotVeh^2	-8,622E-9	0,000	-2,804	0,006	0,000	0,000	0,621	1,610
LnTotBit	-0,032	,015	-2,118	0,037	-0,062	-0,002	0,820	1,219

8.33 taula. Proposatutako erregresio lineal anizkoitzeko modeloak IRI portaerarako Trazatu Berria tarteetan bide-zoru erdi-zurrunetan

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
IRI = Int + PotR.Age + SSbit + SSsub + TotH.Veh + TotVeh	0,387	0,346	Esangura baxua, SSbit (p=0,13), SSsub (p=0,32)
IRI = Int + PotR.Age + TotH.Veh^2 + SSsub + TotBit	0,324	0,288	Esangura baxua, TotH.Veh^2 (p=0,34), SSsub (p=0,94)
IRI = Int + PotR.Age + TotH.Veh^2 + TotBit	0,324	0,297	Esangura baxua, TotH.Veh (p=0,34)
IRI = Int + PotR.Age + TotH.Veh + TotBit + SSsub	0,325	0,289	Esangura baxua, TotH.Veh (p=0,31), SSsub (p=0,93)
IRI = Int + PotR.Age + LnTotBit + TotH.Veh	0,318	0,292	Esangura baxua, TotH.Veh (p=0,328)
IRI = Int + PotR.Age + LnTotBit + TotVeh	0,371	0,347	Esangura baxua, Int(p=0,82), IC=43
IRI = Int + PotR.Age + SSbit + TotVeh	0,361	0,337	Esangura ertaina, SSbit(p=0,137)
IRI = Int + PotR.Age + TotBit + TotVeh	0,384	0,360	Aldagai guztiak esanguratsuak dira (p<0,05)
IRI = Int + PotR.Age + TotBit + TotH.Veh	0,330	0,304	Esangura baxua, TotH.Veh (p=0,289)

Oharra: Int: Konstantea.

8.34 taula. Azpi-oinarriko 1. geruzako material posibleak (SUB1) eta 2. geruzako material posibleak (SUB2) bide-zoru erdi-zurrunetan.

Azpi-oinarriko 1. geruza (SUB1)	**Azpi-oinarriko 2. geruza (SUB2)**
Hartxintxar eta zementua (12)	Zagor artifiziala (11)
Zementu-lurzorua (13)	Zementu-lurzorua (13)
Zepa pikortsua (14	Beiratutako zepa (15)
	Ezer ez (20)

8.35 taulak aurkezten du Erregresio Lineal Orokor Anizkoitzeko (GLM) modeloen laburpena, non faktoreen hainbat konbinazio erabiltzen diren, eta, sartutako faktoreak nahi den bezala konbinatu ahal dira. SPSS v24 programak faktore eta koaldagaien konbinazio espezifikoa ahalbidetzen du. Karratuen batuketa II motakoa da. Azpi-oinarriko geruzen lodierak, *SUB1cm* eta *SUB2cm*, lehenengo eta bigarren geruzarako hurrenez hurren, ere sartu dira aldagai kuantitatibo bezala.

8.35 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak IRI portaerarako Trazatu Berria tarteetan bide-zoru erdi-zurrunetan

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
IRI = Int + PotR.Age + TotBit + TotVeh + SUB1(f)	0,422	0,384	Esangura baxua, TotBit (p=0,775)
IRI = Int + PotR.Age + TotBit + TotVeh + SUB1(f) + SUB2(f)	0,464	0,404	Esangura baxua, TotBit (p=0,998), SUB2 (p=0,143)
IRI = Int + PotR.Age + TotVeh + SUB1(f) + SUB2(f)	0,464	0,413	Esangura baxua, SUB2 (p=0,134), Int(p=0,1)
IRI = Int + PotR.Age + TotVeh + TotBit + SUB1(f) + SUB2(f) + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,470	0,376	Esangura baxua, TotBit (p=0,89), SUB1 (p=0,96), SUB2 (p=0,28) eta biderketak
IRI = Int + PotR.Age + TotVeh + TotBit + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,460	0,391	Esangura baxua, TotBit (p=0,69), eta bideketak (p>0,12)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,459	0,398	Esangura baxua, SUB2*SUB2cm (p=0,38), eta Int (p=0,41)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm + SUB2(f)	0,463	0,404	Esangura baxua, SUB2(f) (p=0,29), eta Int (p=0,285)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,466	0,381	Esangura baxua, + SUB1(f)*SUB1cm (p=0,983) SUB2(f)*SUB2cm (p=0,673)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB2(f)	0,465	0,405	Esangura ertaina, Int (p=0,146)
IRI = Int + PotR.Age + TotVeh + TotBit + SUB1(f)*SUB2(f)	0,465	0,397	Esangura baxua, TotBit (p=0,995), Int (p=0,257)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm*SUB2(f)*SUB2cm	0,386	0,337	Esangura baxua, biderketa (p=0,271)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm*TotBit + SUB2(f)*SUB2cm*TotBit	0,459	0,399	Esangura baxua, SUB2(f)*SUB2cm*TotBit (p=0,184), Int (p=0,426)
IRI = Int + PotR.Age + TotVeh^2 + SUB1(f)*SUB1cm*TotBit + SUB2(f)*SUB2cm*TotBit	0,438	0,375	Konstantea zero izan ahal da (p=1,00)
IRI = Int + PotR.Age + TotH.Veh + SUB1(f)*SUB1cm*TotBit + SUB2(f)*SUB2cm*TotBit	0,394	0,327	Esangura baxua, TotH.Veh (p=0,39), Int (p=0,563)

8.35 taulan ikusten den moduan, azpi-oinarren geruzen lodierak sartzean, aldagai horiek ez dira esanguratsuak modeloan. Material mota eta bere lodiera konbinatu dira *TotBit* baina beste aldagaiak esangura baxua lortu zuten. Bide-zoru malguetan bezala, trafikoarekin erlazionatutako aldagaien hainbat konbinazio saiatu ziren. Saiakera hauek 8.36 taulan laburtzen dira.

8.36 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak IRI portaerarako Trazatu Berria tarteetan bide-zoru erdi-zurrunetan Trafiko bolumenekin erlazionatutako aldagaien hainbat konbinaziotarako

Proposatutako modeloak	R^2	R^2_{adj}	Komentarioak eta oharrak
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB2(f)*TotBit	0,464	0,396	Esangura baxua, Intercept (p=0,243)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f)	0,503	0,441	Esangura baxua, Kontantea (p=0,318)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f)*TotBit	0,498	0,427	Esangura baxua, Intercept (p=0,274)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + TotBit + SUB1(f)*SUB2(f)	0,504	0,433	Esangura baxua, Intercept (p=0,382), TotBit (p=0,884)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + LnTotBit + SUB1(f)*SUB2(f)	0,505	0,435	Esangura baxua, Intercept (p=0,344), TotBit (p=0,602)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + LnTotBit + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,498	0,426	Esangura baxua, Intercept (p=0,850), SUB2(f)*SUB2cm (p=0,480), LnTotBit (p=0,989)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,498	0,434	Esangura baxua, Intercept (p=0,801), SUB2(f)*SUB2cm (p=0,464)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB1cm + SUB2(f)	0,500	0,436	Esangura baxua, Intercept (p=0,616), SUB2(f) (p=0,423)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)	0,481	0,446	Esangura baxua, Intercept (p=0,446)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f) + SUB2(f)	0,501	0,446	Esangura baxua, Intercept (p=0,125), SUB2(f) (p=0,409)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f) + SUB2(f)*SUB2cm	0,500	0,444	Esangura baxua, Intercept (p=0,199), SUB2(f)*SUB2cm (p=0,440),
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f) + SUB1(f)*SUB2(f)*LnTotBit	0,514	0,419	Esangura baxua, biderketak(p > 0,7)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f)*LnTotBit	0,504	0,434	Esangura baxua, Konstantea (p=0,338)

8.36 taulan erakusten den gisa, aldagai guztiak esanguratsuak izanik korrelazio onenak dituzten ekuazioak taularen bigarren eta azken ekuazioak dira. Ekuazio bietan, bide-zoruaren adina, sekziotik igaro diren ibilgailu totalen kopuru metatua eta ibilgailu astunen kopuru metatua eta azpi-oinarrizko geruzetan zabaldu ahal diren materialak kontuan hartzen duen faktore bat. Bigarren modeloak, 8.36 taularen azken ekuazioak, geruza bituminosoen lodiera ere erabiltzen du, azpi-oinarrizko geruzen material bituminosoak kontuan hartzen duen faktorearekin konbinatuta. Hala ere, aldagai gehigarri hau modeloan sartu arren, modelo biak antzeko determinazio-koefizienteak dituzte (0,503 eta 0,504). Hala eta guztiz ere, lehenengo modeloak aldagai gutxiago bat daukanez, doitutako R2 (7.41 ekuazioa) handiagoa da lehenengo ekuazioan. Horrez gain, aldagaien esangura handiagoa da lehenengo modeloan. Hori dela eta, lehenengo modeloa (8.36 taularen bigarren ekuazioa) proposatzen da errepide konbentzionaleko bide-zoru erdi-zurrun berrietarako. 8.26

ekuazioaren bidez definitzen da:

$$IRI = 0{,}217 + 2{,}237 \cdot R.Age^{0{,}133} - 2{,}208 \cdot 10^{-5} \cdot TotVeh + 1{,}738 \cdot 10^{-4} \cdot TotH.Veh + SUB_f \qquad [8.26]$$

non

IRI : iragarritako batez besteko International Roughenss Index (m/km) adina, trafiko eta azpi-oinarriko balio bereak dituen tarterako bide-zoru erdi-zurrun berrietan.

R.Age : bide-zoruaren adina, zerbitzuan jarri zen data zehatzetik kalkulatu nahi den momentura arte, urtetan, frakzio dezimaletan azalduta, non 0,5 jartzeak adierazi nahi du 6 hilabete.

TotVeh : kontsideratutako denbora-tartean sekziotik igarotako ibilgailu totalen kopuru metatua ireki zenetik kalkulatu nahi den momentura arte bi erreiko errepide konbentzionaleko bi noranzkoetan, mila ibilgailu astunetan azalduta

VehH.Tot: kontsideratutako denbora-tartean sekziotik igarotako ibilgailu astunen kopuru metatua proiektuko erreitik (galtzadan ibilgailu astun gehiago igarotzen diren erreian) ireki zenetik kalkulatu nahi den momentura arte, mila ibilgailu astunetan azalduta. Orokorrean, noranzko biek astunen trafiko berdina daukate, hau da, totalaren erdia.

SUBf: azpi-oinarrian zabaldu ahal diren material ez bituminosoen konbinazio posibleak kontuan hartzen duen faktorea. Bere balioa 8.37 taulatik lortu ahal da.

8.37 taula.8.26 ekuazioan sartu beharreko SUBf faktorearen balioak azpi-oinarrian sartutako material ez bituminosoen arabera

		Azpi-oinarriaren 1. geruzako materialak		
		Hartxintxar eta zementua	**Zementu-lurzorua**	**Zepa pikortsua**
Azpi-oinarriaren 2. geruzako materialak	**Zagor artifiziala**	-0,367	-	-0,198
	Zementu-lurzorua	-0,532	-	-
	Beiratutako zepa	-	-	0,038
	Ezer ez (ez dago 2. Geruza)	-0,330	-0,649	0

8.37 taulan agertzen diren konbinazioak erabil daitezke. Baliorik ez badago, konbinazio hori ez zen aurkitu Bizkaiko errepide-sarean.

8.38 taulak erakusten du 8.26 ekuazioaren modeloaren subjektu-arteko efektuen testa eta aldagai bakoitzaren esangura ikusi daiteke. Konstantea esangura txikia dauka (p-balioa = 0,318). Hala ere, mantentzea nahiago da baztertzen bada, determinazio-koefizientea baxuagoa delako. 8.39 taulak 8.26 ekuazioaren modeloaren parametroen estimatzaileak (koefizienteak) erakusten ditu

8.38 taula. 8.26 ekuazioaren modeloaren Subjektu-arteko Efektuen testa

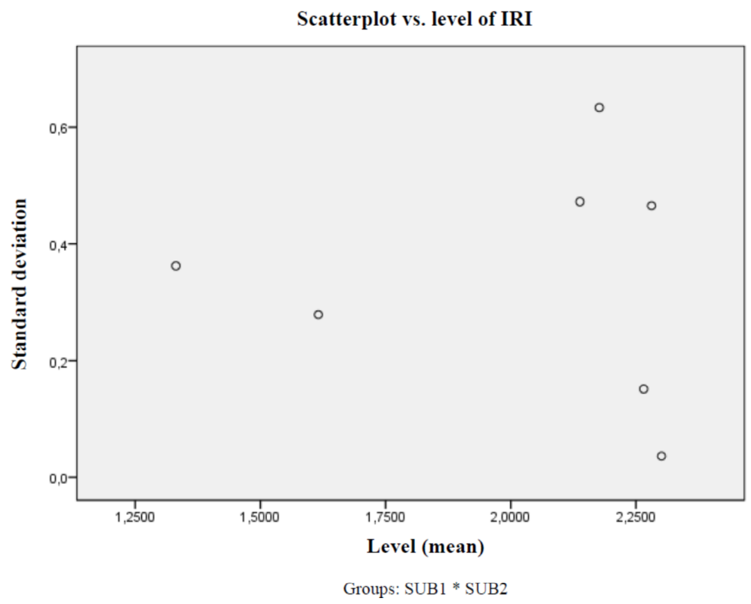
Jatorria	Karratuen batuketa III mota	Askatasun-graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	10,749	9	1,194	7,999	<0,001	0,503	71,993	1,000
Konstantea	0,151	1	0,151	1,012	0,318	0,014	1,012	0,168
PotR.Age	4,127	1	4,127	27,643	<0,001	0,28	27,643	0,999
TotVeh	2,541	1	2,541	17,017	<0,001	0,193	17,017	0,983
TotH.Veh	0,825	1	0,825	5,522	0,022	0,072	5,522	0,640
SUB1(f)*SUB(2)	3,126	6	0,521	3,49	0,004	0,228	20,938	0,929
Errorea	10,601	71	0,149					
Total	386,44	81						
Total zuzendua	21,35	80						

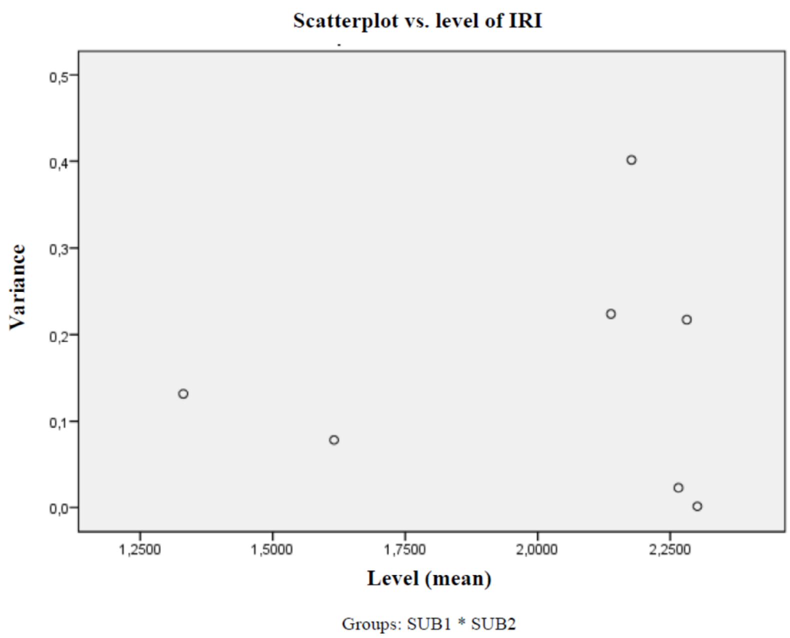
8.39 taula. 8.26 ekuazioaren modeloaren parametroen estimazioak.

Parametroak	B	Errore estandarra	t	Esang	%95 konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Goiko			
Konstantea	-0,217	0,518	-0,419	0,677	-1,25	0,816	0,002	0,419	0,07
PotR.Age	1,341	0,255	5,258	0	0,832	1,849	0,28	5,258	0,999
TotVeh	-2,21E-05	5,35E-06	-4,125	0	-3,28E-05	-1,14E-05	0,193	4,125	0,983
TotH.Veh	1,74E-04	7,40E-05	2,35	0,022	2,63E-05	3,21E-04	0,072	2,35	0,64
[SUB1=12] * [SUB2=11]	-0,367	0,235	-1,565	0,122	-0,835	0,101	0,033	1,565	0,339
[SUB1=12] * [SUB2=13]	-0,532	0,305	-1,744	0,086	-1,14	0,076	0,041	1,744	0,405
[SUB1=12] * [SUB2=20]	-0,33	0,133	-2,49	0,015	-0,595	-0,066	0,08	2,49	0,69
[SUB1=13] * [SUB2=20]	-0,649	0,166	-3,91	0	-0,98	-0,318	0,177	3,91	0,971
[SUB1=14] * [SUB2=11]	-0,198	0,126	-1,566	0,122	-0,449	0,054	0,033	1,566	0,339
[SUB1=14] * [SUB2=15]	0,038	0,243	0,156	0,876	-0,447	0,523	0	0,156	0,053
[SUB1=14] * [SUB2=20]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau errredundatea delako

8.8 irudiak mailazko dispertsio-diagramak erakusten ditu eta informazio grafikoa proportzionatzen du bariantzaren homogeneotasunari buruz eta batez bestekoaren neurriaren eta bariantzen neurrien arteko edozein erlazio posiblea sumatzen ahalbidetzen du. Mailazko bariantzak berdinak ez direnez, 8.8 et a8.9 irudietako puntuak ez daude horizontalean alienatuta.

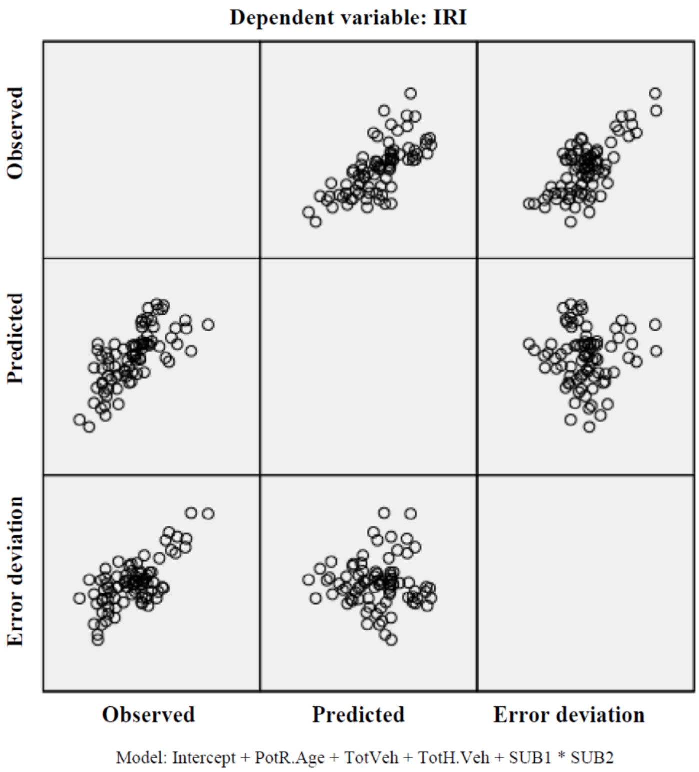




8.8 irudia. 8.26 ekuazioaren mailazko dispertsio-diagrama 8.26 a) Desbideratze estandarra, b) Bariantza

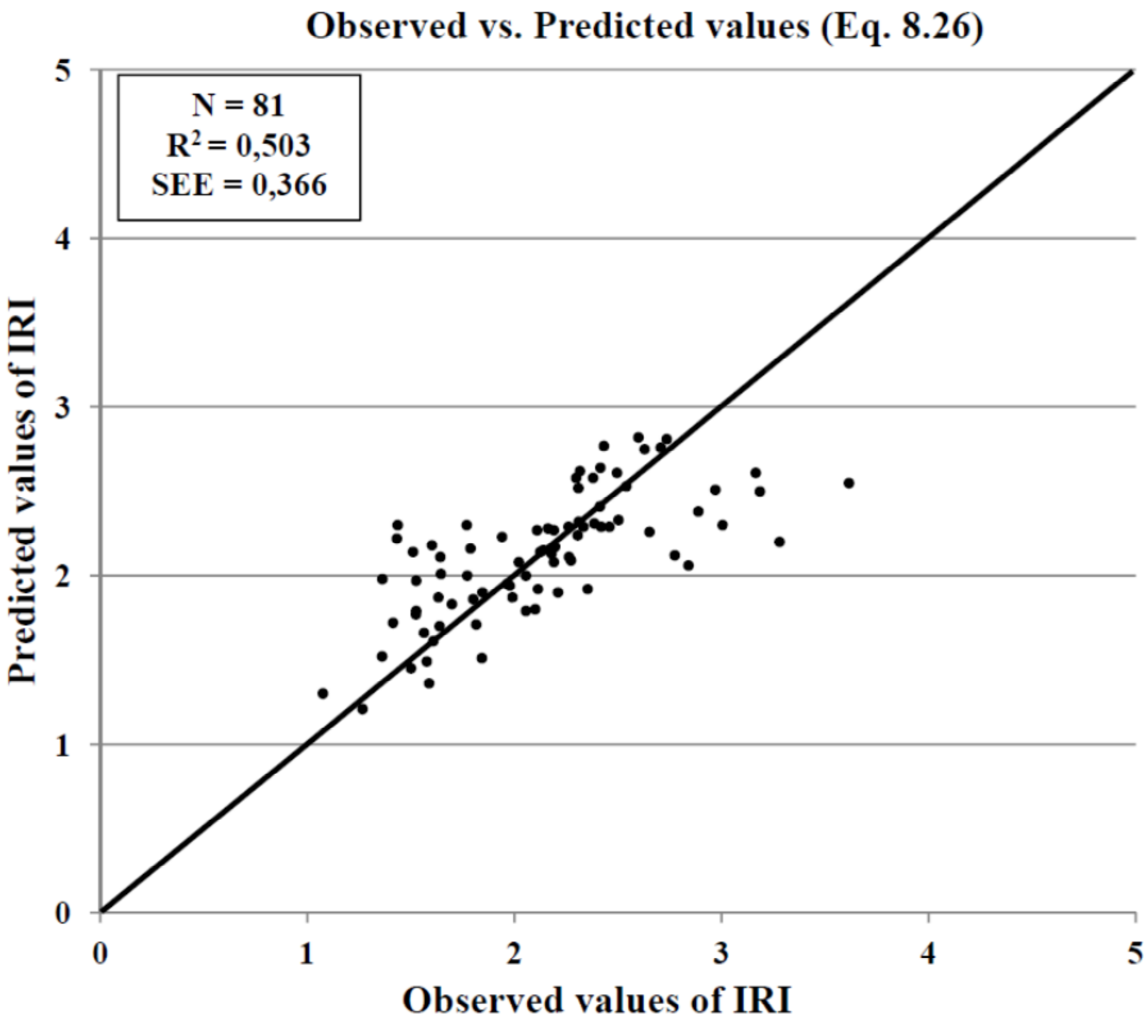
8.9 irudiko erroreen grafikoa erroreak ausazkoak direla eta haien artean independenteak direla ikusten ahalbidetzen du. Iragarritako balioak eta errore estandarren grafikoa ausazkoa denez (ez dago patroirik), erroreak independenteak dira. Erroreen bariantzak homogeneoak dira errore estandarren dispertsioa antzekoa da iragarritako balio guztien zehar. Iragarritako balioen eta neurtutako balioen grafikoak patroi lineala erakusten du, determinazio-koefizientean islatzen dena.

8.10 irudian 8.26 ekuazioaren bitartez iragarritako balioen eta neurtutako balioen grafikoa detaile handiagoz erakusten da.



8.9 irudia. Errore (estandarren, behatutako balioen eta 8.26 ekuazioaren bidez iragarritako balioen grafikoa

Bide-zoru zurrunetako errepide konbentzional berrietan IRI iragartzeko proposatutako modeloak bide-zoruaren adina, ibilgailu total eta ibilgailu astunen kopuru metatua eta faktore bat bide-zoru zurrunaren azpi-oinarria osatu ahal dituzten geruza batean edo bitan sartu ahal diren material ezberdinak kontuan hartzeko. Modelo horretan, bide-zoruaren gaineko kargak ibilgailu total eta ibilgailu astunen konbinazio batek adierazten ditu. Ibilgailu astunen koefizienteak ikur positiboa dauka eta ibilgailu totalen koefizienteak ikur negatiboak. Horrek azpimarratzen du ibilgailu astunen eragina handiagoa bide-zoruaren erregulartasunean, espero ahal zen gisa. Bide-zoruaren egitura erdi-zurruna kontuan hartzeko, azpi-oinarrian dauden materialak kontsideratzen dituen faktore bat behar da. Faktore horrek ez du behar geruza horien lodierak indarrean dauden arauak jarraituz diseinatzen baitira bide-zoruak. Modeloan zapi-oinarriko geruzen materiala eta geruza bituminosoen lodiera kontuan hartzen zituen faktore bat ere erabili ahal zen. Hala ere, modelo alternatibo horrek ez du hobekuntzarik erakusten eta bide-zoruen diseinua ezarritako arauen arabera gauzatzen denez, ez zela beharrezkoa erabaki zen.



8.10 irudia. Neurtutako datuak eta iragarritako datuak 8.26 ekuazioren bidez.

8.6. Mantentzea eta Errehabilitazioa tarteetarako proposatutako IRI portaera-modeloak

"Mantentze eta Errehabilitazio" bezala sailkatutako bide-zoru malguen eta erdi-zurrunen kopuru txikiak direla eta, konfiantzako modelo bat garatzeko daturik ez daudela kontsideratu da. Horren ondorioz, galtzada banandutako tarteak ere aztertzea beharrezkoa izango da datu eskuragarri nahikoa izateko. Ideia hau etorkizuneko ikerketa linea bezala sartu da 10.2 atalean.

8.7. Ondorioak

Bizkaiko errepide-sareko errepide konbentzional berrietarako IRI portaera modeloak garatu dirá bide-zoru malguarekin eta bide-zoru erdi-zurrunarekin. Bide-zoru malgu eta erdi-zurrunetarako modelo indibidualak garatu ziren haien portaera ezberdina kontuan hartuz.

Bide-zoru malguetarako, erregresio lineal anizkoitz bat hartu zen, zeinek bide-zoruaren adina eraiki zenetik, geruza bituminosoen lodiera eta sekziotik igarotako ibilgailu astunen kopuru metatua erabiltzen dituen, literaturan ohiko faktoreak direlarik.

Bide-zoru erdi-zurruneko errepide konbentzionaletarako, IRI iragartzeko proposatutako ekuazioak bide-zoruaren adina, sekziotik igarotako ibilgailu total eta astunen kopuru metatuak eta bide-zoru zurrunaren azpi-oinarria osatzen dituen geruza batean edo geruza bietan sartu ahal diren material ezberdinak kontuan hartzen duen faktore bat.

9. kapitulua. SCRIM koefizientearen portaera-modeloak Bizkaiko errepideetarako

9.1. Sarrera

Kapitulu honetan SCRIM koefizientearen portaera-modeloak Bizkaiko errepide-sarerako garatzen dira. Bide-zoruko marruskadurari buruzko ikerketek frogatu dute, errepide berri bat zerbitzuan jartzen denean, hasierako hobekuntza agertzen dela marruskaduran agregakinak biltzen dituen betuna kentzen delako eta agregakinak trafikoaren aurrean agertzen dira. Orduan, puntu maximo baten ostean (5.31 irudia), azalera oso azkar leuntzen da, hasieran labainketarekiko erresistentziaren galera ratio handiarekin eta geroago ratio motelago batez, oreka puntu batera arte. Oreka fasean, balioak konstanteak dira eta urte-sasoiko aldakuntzak erregistratzen dira, neguan balio altuenekin eta udan baxuagoak.

Modelo batzuk proposatu dira urte-sasoiko aldakuntzak kuantifikatzeko baina interesanteagoa da marruskadura balioak iragartzea haien balio minimoan daudenean, haien egoera txarrenean ezagutzeko. Literaturan aurkitu den bezala (5.5 atala), labainketarekiko erresistentzia errepideetan bi aldagaien araberakoa izaten da: ibilgailu astunen trafikoa (hainbat definizioren arabera azalduta) eta agregakinaren Azeleratutako Leuntze Koefizientea (PSV, *Polished Stone Value*) (Szatkowski and Hosking, 1972; Roe and Hartshorne, 1998; Transit New Zealand, 2002a; NZTA, 2013a). Azken aldagai horrek agregakinaren leunketaren aurreko erresistentzia adierazten du. Lehenengo aldagaiak agregakinean leuntzeko aplikatutako kargak adierazten ditu. Laborategiko entseguak joera berdina erakutsi dute: hasieran leuntze azkarra, geroago leuntze motelagoa eta azkenik, oreka puntua, non leuntze-ziklo gehiagok ez du lortzen labainketarekiko erresistentzia murriztea. Kurba hori agregakinen ezaugarrietatik kalkulatu ahal da, hala nola, gradazioa, tamaina, etab.

Lehenengo eta behin, Bizkaiko errepide berrietarako labainketarekiko erresistentziaren hasierako modeloa garatu zen, neguan hartutako SCRIM datuekin. Bigarrenez, udako marruskadura datuekin hainbat modelo garatu nahi ziren. 8. Kapituluan aurkeztutako familiak, Trazatu berriak eta Mantentzea eta Errehabilitazio tarteak kontsideratzen dira analisia hasteko.

9.2. Labainketarekiko erresistentziaren modeloa Bizkaiko errepide berrietan neguko datuekin

2016ko labainketarekiko datuak (eta IRI-eko datuak) lortu baino lehen, 2017an jaso zirenak, 2011ko neguan neurtutako datuen bitartez labainketarekiko erresistentziaren modelo bat garatu zen. 6. kapituluan komentatu denez, BFAk datu-biltze kanpainak gauzatu zituen 2000n, 2002an (partzialki(, 2007an eta 2011n. 2000ko, 2004ko eta 2007ko labainketarekiko erresistentziaren datu-biltzeen datak ez dago erregistratuta. BFAko ingeniariak esaten dute udan neurtu zela, baina ez dago data zehatzik datu-basean eta ondorioz, ezin dira datu horiek erabili konfiantzaz kalkulatzeko. 2011n hartutako datuak 2011ko otsailean eta martxoan hartu zirela erregistratu da, marruskadura balioak haien balio maximoetan daudenean (Burchett and Rizengers 1980; Echaveguren and Solminihac, 2011).

Iragartzeko modelo hau garatzean, aurreko egituren edo errehabilitazio edo mantentze-lanen eragina ekiditeko, errepide berrietako 16 tarteak (hiri artekoak), azken 25 urteetan eraikita, aukeratu ziren analisirako (Pérez-Acebo *et al.*, 2017a). Tarte hauek ez daukate errehabilitazio edo mantentze-lanik 2011 urtera arte. 9.1 taulak erakusten ditu aukeratutako errepide tarteak haien ezaugarri nagusiekin. Errepide guztiak errepide konbentzionalak dira, hau da, bi erreiko errepideak

9.1 taula. Labainketarekiko erresistentziaren modelorako aukeratutako errepide tarteak neguko datuekin (Pérez-Acebo et al. 2017a).

Errepidearen izendatzea (kodea)	**Errepide-sarea**	**Errodadura geruza**	**Bide-zoru mota [a]**	**Luzera (km)**	**Zerbitzuan jarri zen data**
N-240 (I)	Lehentasuneko interesa	AC 16 surf S	EZ	0,70	24/05/2002
BI-625	Oinarrizkoa	AC 16 surf S	ML	1,04	01/10/1994
BI-633 (I)	Oinarrizkoa	AC 16 surf S	ML	1,34	08/01/2003
BI-633 (II)	Oinarrizkoa	AC 16 surf S	ML	2,535	07/08/2002
BI-647	Oinarrizkoa	AC 16 surf S	ML	1,40	01/06/2007
BI-2121	Eskualdekoa	AC 16 surf S	ML	0,975	01/07/2000
BI-2238 (I)	Eskualdekoa	AC 16 surf S	EZ	1,02	01/06/2009
BI-2238 (II)	Eskualdekoa	AC 16 surf S	EZ	1,55	23/10/2009
BI-2224	Eskualdekoa	AC 16 surf S	ML	0,08	01/06/2007
BI-2405	Eskualdekoa	AC 16 surf S	EZ	1,570	23/10/2009
BI-2522	Eskualdekoa	AC 16 surf S	EZ	1,300	01/06/1997
BI-2713 (I)	Eskualdekoa	AC 16 surf S	EZ	0,08	01/11/2003
BI-732	Osagarria	BBTM 11A	EZ	0,70	01/10/2005
BI-2713 (II)	Eskualdekoa	BBTM 11A	EZ	2,260	03/08/2005
N-240 (II)	Lehentasuneko interesa	PA 11	SR	1,71	19/07/2002
BI-633 (III)	Oinarrizkoa	PA 11	ML	1,42	18/09/2000

[a] Bide-zoru mota: ML, malgua; EZ, erdi-zurruna

Tarte batzuk, zeharkako sekzio bererarekin, azpi-tarteetan zatitu ziren trafiko bolumenen arabera, trazatuan zehar bidegurutze batzuk zeudelako. Horren ondorioz, 23 sekzio ezberdin trafiko bolumen ezberdinekin aztertu ziren.

Errodadura geruzarako 3 material bituminoso daude: nahaste etenak (BBTM 11A), hormigoi bituminosoko nahaste erdi-trinkoak (AC surf S) eta nahaste drainatzaileak (PA 11). Bide-zoruaren egiturari dagokionez, bide-zoru malguak (ML) eta erdi-zurrunak (EZ) aztertzen dira.

Labainketarekiko erresistentzia iragartzeko honako aldagai hauek sartu ziren analisian eragina izan ahal duten faktoreak bezala:

- ***Age***. Errepidearen eraikuntzatik igarotako urtean 2011ra arte sartu zen faktore eragingarria bezala. 2011 eta eraikuntzaren urtearen arteko kenketa bezala kalkulatu zen. Errepide bakoitza denboran zehar momentu ezberdin batean eraiki zenez, datu-biltzea gauzatzean errepide bakoitzak adina ezberdina zeukan.

- **Geruza bituminosoen lodiera totala (*TotBit*).** Geruza bituminosoen lodiera ere sartu zen faktore eragingarria bezala, cm-tan. Sartutako sekzio guztiak hasierako bide-zoru bezala sartu denez, haien egitura osoa ezaguna da.

- **Eguneko Batez Besteko Intentsitatea (*AADT*).** Eguneko Batez Besteko Intentsitatea, sekziotik igaro diren ibilgailu total guztien batez bestekoa eguneko adierazten duena, ere sartu zen marruskaduraren analisian.

- **Ibilgailu astunen Eguneko Batez Besteko Intentsitatea (*H.AADT*).** Ibilgailu astunen eguneko trafikoa ere sartu zen. Espainian, 8. kapituluan luze komentatu denez, ibilgailu astuna izateko 3.500 kg baino gehiago pisatu behar du ibilgailuak (Ministerio de Fomento, 2003b). Errepide bakoitza *H.AADT*-en arabera, (ibilgailu astun/egun/errei) proiektuko erreian errepidea zerbitzuan jartzen denean (9.2 taula) (Ministerio de Fomento, 2003b).

9.2 taula. Espainiako trafiko kategoriak (Ministerio de Fomento, 2003b).

Trafiko kategoria	**H.AADT (ibilgailu astun/egun/errei)**	**Trafiko kategoria**	**H.AADT (ibilgailu astun/egun/errei)**
T00	H.AADT ≥ 4.000	**T31**	200 < H.AADT ≤ 100
T0	4.000 < H.AADT ≤ 2.000	**T32**	100 < H.AADT ≤ 50
T1	2.000 < H.AADT ≤ 800	**T41**	50 < H.AADT ≤ 25
T2	800 < H.AADT ≤ 200	**T42**	25 < H.AADT

- **Beharrezko PSV (PSV_{req}).** 5. kapituluan azaldu den bezala, PSV-a ezarri da labainketarekiko erresistentziaren iragartzeko modeloetan oinarriko faktore bezala. Agregakinen erresistentzia leuntzearen aurrean adierazten du (Szatkowski and Hosking, 1972; Roe and Hartshorne, 1998; Transit New Zealand, 2002a; NZTA, 2013a). Hala era, Bizkaiko errepide bakoitzeko agregakinen PSV ezezaguna da. Espainiako arauak ezartzen dute eskatutako PSV minimoa erabilitako materiala eta trafiko kategoriaren arabera. Aukeratutako errepideen proiektuen orduan (9.1 taula), AC 16 surf S nahastetarako 0,50 balio minimoa ezartzen zen T0 eta T1 kategorietarako, 0,45 T2 kategoriarako eta 0,40 T31-T42 kategorietarako. Nahaste etena eta drainatzaileentzat, 0,50 ezartzen zen minimo bezala T0 – T2 tarteetarako eta 0,45 T31-T42 tarteetarako (Ministerio de Fomento, 2001, 2004a; Ministerio de Obras Públicas y Urbanismo, 1989b). Gaur egun, arauak ezartzen du PSV-ren balioak 0 – 100 eskala batean azaldu behar dela (Ministerio de Fomento, 2015) eta eskala hau da modelizatzeko erabili zena.

Mendeko aldagaiari buruz, labainketarekiko erresistentziaren balio, SCRIM koefizientearen bidez lortzen dena, erabili zen. Ikerketa honetan erabilitako 23 tarteen batez besteko balioa kalkulatu zen. Baina datu hauek neguan lortu ahal den labainketarekiko erresistentziaren balio maximoa adierazten dute. Interesanteagoa da udako balioak jakitea, balioak haien minimoetan daudenean eta haien bariantza minimoa denean, Erresuma Batuko bide-agentziak erabiltzen duen *Mean Summer SCRIM Coefficiente* (MSSC) bezala, udan neurtutako

Side-force coefficient (SFC)-en (zeharkako marruskadura koefizientea) 3 balioen batez bestekoa (5.4.5.5 atala) (Hosking and Woodford, 1976b). Bizkaiko datuei dagokienez, 200, 2004 eta 2007an datuak udan hartu zirela suposatuz, urte-sasoiko aldakuntza geruza mota bakoitzerako kalkulatu ahal da (9.3 taula).

9.3 taula. SFC aldakuntza Bizkaian eta Gipuzkoan (Navarro et al. 2011).

Errodadura mota	**Aldakuntza heina Bizkaian**	**Batez besteko SFC Bizkaian**	**Aldakuntza heina Gipuzkoan**	**Batez besteko SFC Gipuzkoan**	**Proposatutako murrizpena**
BBTM 11A	14,7	65	18	55	13,5
PA-11	11,3	60,1	17	54	13
AC16 surf S	17,1	54,7	12,5	50	9

Aldakuntzen hein hauek (negutik udara) Gipuzkoan gauzatutako ikerketa baten datuekin konparatu ziren, non SCRIM koefizientearen balioak bildu zirren aurretik konpondutako errepideetan 3 hilero 2 urtez (Navarro *et al*., 2011). Gipuzkoa Euskal Autonomia Erkidegoko beste probintzia bat da, kostaldean ere kokatuta, Bizkaiko klima berea duenak (Klima ozeanikoa). Navarro *et al*. (2011)-ek hasierako jaitsiera erregistratu zuten lehenengo 12 hilabetetan eta handik aurrera urte-sasoiko aldakuntzak, balio baxuena ekainean erregistratuz (ez ziren egin neurketarik uztailean ezta abuztuan ere) (9.3 taula).

Ikusten den gisa, Bizkaian neurtutako aldakuntza heina Gipuzkoan lortutakoaren antzeko da (Navarro *et al*., 2011). Ez dira berdinak zeren eta Bizkaiko ikerketan aztertzen diren tarteak ez dira datu-biltze guztietan (2000, 2004 eta 2007an) neurtu. Urte-sasoiko aldakuntza hauek Echaveguren and Solminihac (2011)-ek proposatutako heinaren barruan daude hormigoi bituminosorako, 8 eta 25en artean. Errodadura geruza mota bakoitzerako proposatutako murrizpena, neguko datutik udako balio minimora, 9.3 taularen azken zutabean erakusten da. Gipuzkoan neurtutako aldakuntzaren 3/4 da, Gipuzkoan errepidea martxan jarri zenetik neurtzen hasten zenez. Hori dela eta, 2011ko neguan aukeratutako tarteen batez besteko balioa murrizten da 6.3 taularen azken zutabeko kantitatea, errodadura geruzaren materialaren arabera, *Mean Summer SCRIM Coefficient*–a (*MSSC*) lortzeko.

Orduan, erregresio lineal anizkoitza gauzatu zen *MSS*C mendeko aldagai bezala eta bere balioan eragina izan ahal duten aldagai independente posibleak: *AADT*, *H.AADT*, *Age*, geruza bituminosoen lodiera (*TotBit*) eta beharrezko *PSV* (PSV_{req}). Aldagai bakoitzaren eragina ebaluatu zen SPSS programaren Aurrera pausoz pauso funtzioarekin. Erregresio analisia gauzatzen denean, hipotesi batzuk onartzen dira, 7. Kapituluan laburbilduta daudenak.

Eraldaketa batzuk gauzatu ziren aldagai independentearen banaketa normala lortzeko eta korrelazio handiago izateko mendeko aldagai eta aldagai independentearen artean. Nahiz eta *MSS*C-en eta *AADT* eta *H.AADT*-en artean korrelazioa egon, aldagai independenteak eraldatu dira natural logaritmoa, erro karratua eta alderantzizkoa aplikatuz, beste ikertzaileen ideiak jarraituz (5.69 eta 5.71 ekuazioak(eta beste egileek marruskadura datuak aztertzean aplikatutako antzeko teknikak (Ongel *et al*., 2009) datuen banaketa normala lortzeko. 9.4 taulak aldagaien arteko Pearson-en korrelazio-koefizienteak erakusten ditu, esangura altua duten erlazioak adieraziz.

Erregresio anizkoitzaren analisiaren ondoren, lortu ahal zen emaitzarik onenak baztertzen zituen *AADT*, *Age* eta *TotBit*. Horren ondorioz, 9.1 ekuazioa proposatzen zen labainketarekiko erresistentzia iragartzeko %95 maila-konfiantzarekin (Pérez-Acebo *et al.*, 2017a).

$$MSSC = 30{,}19 - 0{,}82 \cdot \sqrt{H.AADT} + 0{,}76 \cdot PSV_{req} \qquad [9.1]$$

H.AADT ibilgailu/egun/errei-tan eta PSV_{req} 0tik 100era doan eskala batean adierazten da. MSSC ere 0tik 100era doan eskala batean adierazten da.

Ikusi den bezala, modeloan erabiltzen diren aldagai bakarrak *H.AADT*-ren erro karratua eta beharrezko PSV. 9.5 taulan erakusten den bezala,modeloaren determinazio-koefizientea (R^2) 0,696 da, ekuazioa 23 sekzioen bariantzaren %70a azaltzeko gai dela adieraziz. Durbin-Watson estatistikoa 1,499 da, erroreen independentzia frogatuz. 9.1a irudiak erroreen distribuzioa normala dela erakusten du. 9.1b irudiak erakusten du erroreak ez dute patroirik jarraitzen, homoszedastizitatea egiaztatuz. *Variance Infation Factor* (*VI*F) indizeak 1,17ko balio dauka, kolinealtasunaren arazorik ez dagoela egiaztatuz. *F* testak korrelazioa benetakoa da ($p < 0{,}01$) and aldagai independentearen koefizienteak Studen-en *t* test baten bidez errealak direla ikusten da (ez dira zero izango) (9.5 taula).

Proposatutako modeloak ez du kontsideratzen bide-zoruaren adina, *MSSC*-rekin korrelazio baxua zeukana ($R = 0{,}06$), TRL-ren ideiak indartuz, eta 5.31 irudiaren itxura egiaztatuz. Oreka fasean, aldakuntza bakarrak urte-sasoikoak dira eta balio minimoa konstante mantentzen da urteetan zehar. Horren ondorioz, beste ikertzaileek garatutako formulak zertifikantzen dira ere (Rezaei and Masad, 2013; Kassem *et al.*, 2013). Beraz, frogatzen da, leuntze-ziklo kopuru baten ostean, asintotako balio bateraino heltzen da. Bide-zoruak 2 urte baino gehiago baditu, adina ez da eragina duen faktore bat.

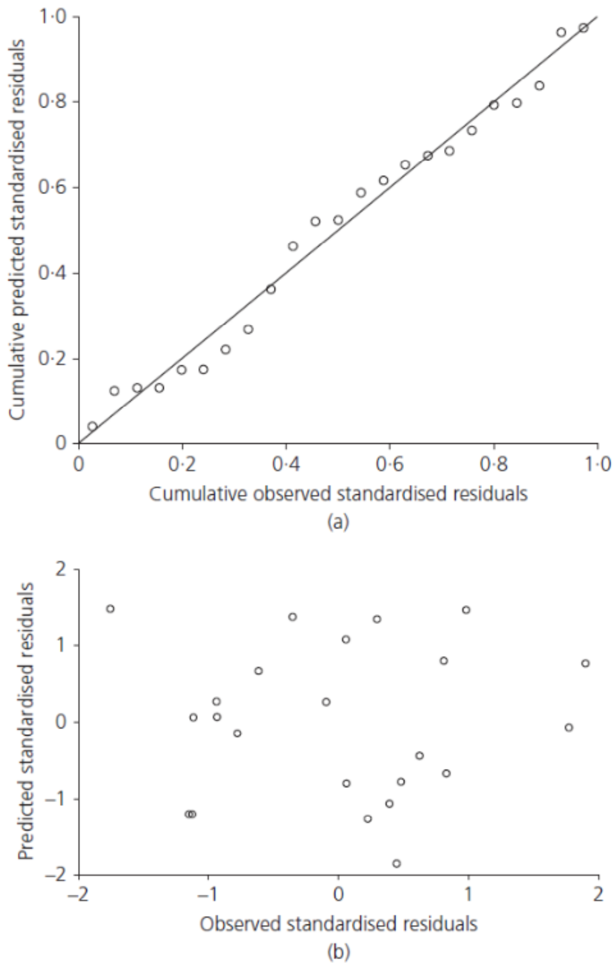
9.4 taula. Aldagaien arteko korrelazioak (Pearson-en R korrelazio-koefizientea) (Pérez-Acebo et al., 2017a)

	MSSC	AADT	H.AADT	Ln(AADT)	Ln(H.AADT)	Inv(AADT)	Inv(H.AADT)	Sqrt(AADT)	Sqrt(H.AADT)	PSV_{req}	Age	Tot Bit
MSSC	1,00											
AADT	-0,66**	1,00										
H.AADT	-0,75**	0,70**	1,00									
Ln(AADT)	-0,63**	0,90**	0,65**	1,00								
Ln(H.AADT)	-0,73**	0,75**	0,89**	0,86**	1,00							
Inv(AADT)	0,47*	-0,63**	-0,48*	-0,89**	-0,79**	1,00						
Inv(H.AADT)	0,49*	-0,59**	-0,56**	-0,86**	-0,85**	0,98**	1,00					
sqrt(AADT)	-0,66**	0,98**	0,69**	0,97**	0,82**	-0,76**	-0,726**	1,00				
sqrt(H.AADT)	-0,77**	0,74**	0,98**	0,75**	0,96**	-0,62**	-0,693**	0,77**	1,00			
PSV_{req}	0,02	0,22	0,41	0,25	0,37	-0,37	-0,409	0,22	0,38	1,00		
Age	-0,06	-0,13	0,05	0,00	0,16	-0,12	-0,225	-0,07	0,10	-0,12	1,00	
Tot Bit	-0,33	0,09	0,47**	0,29	0,59**	-0,41	-0,527**	0,18	0,55**	0,20	0,18	1,00

*$p < 0.05$; ** $p < 0.001$

9.5 taula. Modeloaren Bariantzaren analisia (ANOVA) (Pérez-Acebo et al., 2017a)

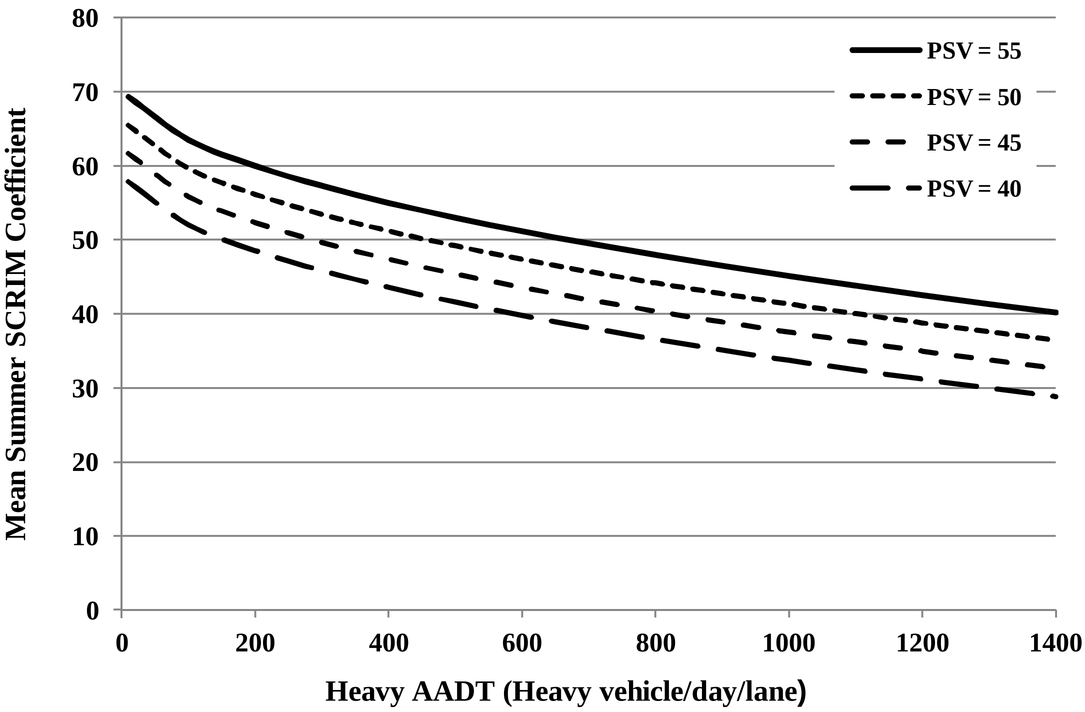
Bariantzaren analisia						
Iturria	**Askatasun-graduak**	**Karratuen batuketa**	**Batez besteko karratuak**	***F* balioa**	***p*-balioa**	**Durbin-Watson**
Modeloa	2	985,122	492,561	22,932	< 0.001	1,499
Errorea	20	429,581	21,479			
Total zuzenduta	22	1414,704				
Estimazioaren errore estandarra	**Mendeko aldagaiaren batez besteko**	**Aldakuntza-faktorea**	***R***	**R^2**	**R^2_{adj}**	***VIF***
6,69166	51,1484	13,08	0,834	0,696	0,666	1,172
Parametroen estimazioak						
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	***t* balioa**	***p*-balioa**	**%95eko konfiantza-tarteak**	
Konstantea	30,188	12,394	2,436	0,024	4,335	56,042
Sqrt(H.AADT)	-0.824	0,122	-6,771	< 0,001	-1,077	-0,570
PSV	0,759	0,280	2,713	0,0013	0,175	1,342



9.1 irudia. Proposatutako modeloaren erroreen analisia: a) erroreen banaketa normalaren doikuntza, b) neurtutako errore estandarren eta iragarritako errore estandarren arteko grafikoa (Pérez-Acebo et al., 2017a)

Horrez gain, geruza bituminosoen lodiera osoak ez du eraginik labainketarekiko erresistentzia, espero ahal zen bezala, marruskadura gainazaleko ezaugarria denez. *MSSC*-rekiko korrelazioa ez da oso baxua (*R* = -0,33) (9.4 taula) baina arrazoia da ibilgailu astunen trafiko handiko bide-zoruak geruza bituminosozko lodiera handiarekin diseinatzen direla. Erregresio-analisiak modelotik baztertzen du aldagai hau. Gainera, errodadura erabilitako materiala (drainatzailea, etena eta hormigoi bituminosokoa) ez dauka garrantzi handirik. Balio maximo eta minimoaren arteko diferentzia kalkulatzeko baino ez da kontuan hartu behar. Bide-zoruaren egitura mota, malgua edo erdi-zurruna, ez zuen aldaketarik erakutsi proposatutako modeloan. Bide-zoru egiturak egituraren arabera biltzean, antzeko modeloak lortzen ziren.

Beste modelo baten antzera, 9.1 ekuazioak lortutako *SFC* minimoa iragartzen ahalbidetzen du eskatutako *PSV* baterako Adibidez, Bizkaiko Foru Aldundiak labainketarekiko erresistentziarako atalase altuagoa jartzen badu, *PSV* handiagoa eskatu behar da errodadura geruzan erabiltzen diren agregakinetan (9.2 irudia). Azpimarratu behar da modeloa *H.AADT*-eko 1.400 ibilgailu/egun/errei baino balio txikietarako baino ezin da erabili,bi erreiko errepide konbentzionaletarako ibilgailu astunen Eguneko Batez Besteko Intentsitatea.



9.2 irudia. Mean Summer SCRIM Coefficient (MSSC)-en iragarpena H.AADT eta PSV_{req}-ren bitartez (Pérez-Acebo et al., 2017a)

9.3. Labainketarekiko erresistentziaren modeloak Bizkaiko errepideetan udako datuekin

Aurreko atalean, labainketarekiko erresistentziaren modeloa garatu zen *Mean Summer SCRIM Coefficient*–a (*MSSC*) iragartzeko neguko datuekin. Kasu horretan, urte-sasoiko aldakuntza estimatu behar zen, eta neguko marruskadurari kendu behar zitzaion, neguan marruskadura bere maximoan dagoelako.

Labainketarekiko erresistentziaren modelo zehatzago bat garatzeko 2016ko udako datu errealak erabili ziren.

Komentatu den bezala, labainketarekiko erresistentzia bere minimora heltzen da udan 5.4.5.5 atalean azaldu diren arrazoiengatik, eta gainera, bere bariantza baxuagoa da (Cenek *et al*., 1999). Hori dela eta, Erresuma Batuko bide-agentziak ezarri duen bezala, marruskadura datuak udan biltzea gomendatzen da (Britain, 2015; Highway Agencies, 2015). Horren ondorioz, BFAk 2016an datuak bildu zituen udan eta datu horiek Bizkaiko errepideetarako labainketarekiko erresistentziaren iragartzeko modelo zehatzago bat lortzeko erabiliko dira.

9.3.1. Labainketarekiko erresistentziaren modeloa guztiz ezagututako bide-zoruekin

Bizkaiko errepide-sareko labainketarekiko erresistentzia iragartzeko modeloa garatzeko, erregulartasunaren portaerarako gauzatutako errepideen analisia, non bide-zoruaren sekzio osoa ezaguna den, berriro erabili da. BKSren datu-basean guztiz erregistratutako bide-zoru egiturak identifikatzeko burututako proiektuen analisia jarraitu zen eta IRI datuen analisia kontsideratu zen. Hala, tarte batzuk baztertu ahal ziren IRI balio hobeagoak bazituzten eta mantentze- edo errehabilitazio-lanik burutu ez baziren. Irizpide hauek jarraituz, guztiz ezagututako bide-zoruaren egiturako tarteak identifikatu ziren, hasierako bide-zoruarekin eta mantentze- eta errehabilitazio-lan posibleak barne. Beraz, II. eranskineko Errepide artxiboetan errepide bakoitzerako “*IRI*” atalean burutu zen antzeko lana egin da “*Skid resistance*” atalean, non bide-zoru sekzioa ezaguna den tarteak identifikatzen dira, haien hasierako eta amaierako PK-ak (Puntu Kilometrikoak) adieraziz. Bi atal daude. 2016an guztiz ezagunak diren bide-zoruen sekzioen tarteak 2016 atalean adierazten dira. Horrez gain, 2011ko datu-biltzearen data ezaguna denez, batez ere 2011 neguan (2011ko otsailean eta martxoan), erabaki zen 2011n ezagunak ziren bide-zoruen sekzioen tarteak ere adieraztea. Hala, posiblea da konparatzea neguan lortutako labainketarekiko erresistentzia maximoa (neguan, 2011ko datuekin) eta udan neurtutako labainketarekiko erresistentziaren balio minimoa (udan, 2016ko datuekin). Hala ere, benetako konparaketa gauzatzeko, egitura berea mantendu behar da 2011tik. Garrantzia 2016ko datuetan dagoenez, 2016ko sekzio berea 2011n zeukaten tarteak zerrendatzen dira, eta horren ondorioz, 2011ko tartearen zerrenda askoz laburragoa da. 2011ko tarteak “2011” atalean zerrendatuta daude.

Guztiz ezaguna den bide-zoruaren egiturako tarteak aukeratzearen arrazoia zen bide-zoruaren egiturak, malgua edo erdi-zurruna, geruzen kopuruek, haien lodierek eta erabilitako materialek benetan eragina zeukaten marruskadura eskuragarrian. 2011ko datuekin gauzatutako aurreko lanak (Pérez-Acebo *et al*., 2017a) eta literaturaren berrikuspenak (Flora, 2009; Prang *et al*., 2012)adierazten dute ez dagoela korrelaziorik bide-zoruaren egituraren eta labainketarekiko erresistentziarekin. Orduan, emaitzek erakusten badute bide-zoruaren egiturazko ezaugarriak ez dutela eraginik labainketarekiko erresistentzian, ikerketa beste ikuspuntuz burutuko da.

Hori del eta, labainketarekiko erresistentziaren analisian erabilitako informazioa jarraian deskribatuko da (9.6 taula), IRI analisiarekin datu batzuk partekatzen direlarik:

- *Tartearen identifikazioa*: IRI analisirako erabilitako zutabe berdinak erabiltzen dira, 8.4.1 atalean deskribatutakoak: 8.4 taularen A – G zutabeak. Horietan adierazten da: errepidearen izendatzea (A zutabea), galtzada (B zutabea), hasierako eta amaierako PK (C eta D zutabeak, hurrenez hurren) eta E to G zutabeak, datu-biltzearen data zehatza, urtea eta urtea frakzio dezimalean sartzeko, bide-zoruaren benetako adina kalkulatzeko.

- *Labainketarekiko erresistentziaren datuak*.: SCRIM koefizientea (*S1 zutabea*) kontsideratutako 20 m-ko tartearentzat, 6.6.2 atalean azaldu zen bezala, 0tik 100erako eskalan adierazia; eta Egitura (*Texture*) (*S2 zutab*ea), mm-tan- Mean Profile Depth (Profilaren Batez besteko Sakonera), laserrean oinarritutako aparatu baten bidez lortuta (9.6 taula).

9.6 taula. BI-631 errepidearen tarte batzuen adibidea tartearen identifikazioarekin eta labainketarekiko erresistentziaren datuekin.

A	B	C	D	E	F	G	S1	S2
Errepidearen izendatzea	**Galtzada**	**Hasierako PK-a**	**Amaierako PK-a**	**Datu-biltzearen data zehatza**	**Datu-biltzearen urtea**	**Datu-biltzearen urtea, zenbakitan**	**SCRIM koefizientea**	**Egitura (mm)**
BI-631	BI-631 bakarra	23+0334	23+0353	30/06/2016	2016	2016,5	38	0,42
BI-631	BI-631 bakarra	23+0354	23+0373	30/06/2016	2016	2016,5	33	0,39
BI-631	BI-631 bakarra	23+0374	23+0393	30/06/2016	2016	2016,5	38	0,45
BI-631	BI-631 bakarra	23+0394	23+0413	30/06/2016	2016	2016,5	38	0,45

- *Trafiko datuak*: IRI-ren analisirako erabilitako informazio berdina, 8.5 taulan erakusten den bezala: datu-biltzearen urteko datuak, orokorrean 2016koa, honako datu hauekin: Eguneko Batez Besteko Intentsitatea (AADT) (*T1* zutabea); ibilgailu astunen Eguneko Batez Besteko Intentsitatea (H.AADT) (*T2* zutabea) Ministerio de Fomento (2003b)-n ezarritako proiektuko erreiari buruzko irizpideak jarraituz; trafiko kategoria (*T3* zutabea); sekziotik igarotako ibilgailu totalen kopuru metatua errepidea zerbitzuan jarri zenetik (Trazatu Berria tarteen kasuan) edo Mantentze- eta Errehabilitazio-lana amatu zenean (M&R tarteetan) datu-biltzearen momentura arte (*T4 zutabea*); sekziotik igarotako ibilgailu astunen kopuru metatua zerbitzuan jarri zenetik edo M&R lana amaitu zenetik datu-biltzearen momenturaino (*T5 zutabea*). 2000 baino aurreko datuak behar izanez gero, 8.4.1 atalean azaldu den gisa, %2ko hazkuntza tasa urte bakoitzean suposatu zen bai trafiko totalerako bai trafiko astunerako.

- *Prezipitazio datuak*. 5.4.5.6 atalean azaldu zen aurreko euri-zaparradek eragina izan dezakete labainketarekiko erresistentzian (Bird and Scott, 1936, Henry 1981, 2000; Hill and Henry, 1981). BFAk errepidetik hurbilago dagoen estazio meteorologikoko datu-biltzearen aurreko 15 eguneko prezipitazioaren datua sartu duenez BKSn, datu horiek, **Rainfall data**, mm-tan, sartu dira ere (*S3 zutabea*) marruskaduraren iragarpenean eragina izan ahal duen faktore bezala.

- *Polished Stone Value*. Beste egileek sartu dute agregakinen Azeleratutako Leuntze Faktorea (*Polished Stone Value*) eragina duen faktore bezala. 9.2 atalean azaldu den gisa, proiektu bakoitzean erabilitako agregakinaren PSV zehatza ez dago eskuragarri. Hala eta guztiz ere, proiektu bakoitzerako eskatzen den minimoa jakiten da, errepidea zerbitzuan jartzen den urtearen (Edo M&R lana amaitzen den urtearen) trafiko kategoriaren arabera. Hori dela eta, trazatu berria edo M&R lana amaitu zen urtearen H.AADT (*S4 zutabea*), aurreko datuari dagokion trafiko kategoria Ministerio de Fomento (2003b)-ren arabera (*S5 zutabea*) eta azkenik beharrezko PSV minimo (*S6 zutabea*) (9.7

taula). Eskatutako PSV-ren balioa trafiko kategorien araberakoa da, errodadura geruzan erabilitako materialaren araberako eta zerbitzuan jarri zen urtean zeuden arauen araberakoa. 9.8 taulak erakusten du eskatutako PSV minimo errodadura geruzaren material bakoitzerako (hormigoi bituminosoa, nahaste etenak, nahaste drainatzaileak edo kare-esneak), trafiko kategoria bakoitzerako eta denbora-tarte bakoitzean indarrean zeuden arauetarako.

9.7 taula. BI-631 errepidearen tarte batzuen adibidea tartearen identifikazioaren datuekin (ez osorik), labainketarekiko erresistentziaren, prezipitazioaren eta PSV-ren datuekin.

A	**C**	**D**	**S1**	**S2**	**S3**	**S4**	**S5**	**S6**
Errepidearen izendatzea	**Hasierako PK-a**	**Amaierako PK-a**	**SCRIM koefizientea**	**Egitura (mm)**	**Rain15 (mm)**	**H.AADT zerbitzuan jarri zen urtean**	**Trafiko kategoria zerbitzuan jarri zen urtean**	**Eskatutako PSV (Azeleratutako Leuntze Koefizientea)**
BI-631	23+0334	23+0353	38	0,42	34,3	249	T2	50
BI-631	23+0354	23+0373	33	0,39	34,3	249	T2	50
BI-631	23+0374	23+0393	38	0,45	34,3	249	T2	50
BI-631	23+0394	23+0413	38	0,45	34,3	249	T2	50

- *Egiturazko datuak:*IRI datuak analizatzeko azaldutako irizpideak errepikatzen dira. 8.4.2 atalean komentatutako datuak sartzen dira Trazatu Berria bezala sailkatutako tarteetarako (8.7 taula *I – Z zutabeak*) eta “Mantentzea eta Errehabilitazioa” bezala sailkatutako tarteetarako, 8.4.3 atalean azaldutako datuak erabiltzen dira marruskaduraren analisirako (8.8 taularen *I – AM zutabeak*).

Hasieran, bi erreiko errepide konbentzionalak baino ez ziren aztertu, 9.2 atalean aurkeztutako lanaren antzera. Trafikorako errei batzuk daudenean, leuntze akzio ezberdina egon ahal denez bide-zoruan, lehenengo pauso bezala errepide konbentzionaletan leuntze-jarduera aztertzea nahiago izan zen, hau da galtzada bakarreko errepideak eta bi noranzkoak. Hala, aztertutako errei bakoitzetik datuaren trafikoa pasatu da. Geroago, bi erreiko errepide konbentzionaletan leuntze-efektuak aztertu eta gero, ikerketa zabaldu ahal da galtzada bananduko errepideetara.

Kontserbazio Eremu 4ko errepideak galtzada bananduko errepideak dira, Bilbo metropolitarraren inguruan dauden autobide, autobia eta errei anitzeko errepideak. Horren ondorioz, Kontserbazio Eremu 1, 2 eta 3ko bi erreiko errepideak baino ez dira aztertuko analisiaren lehenengo fase honetan. Kontserbazio Eremu 1, 2 eta 3ko galtzada bananduko errepide-tarteak baztertzen dira hasierako analisi honetarako.

9.8 taula. Eskatutako PSV minimoa errodadura geruza bituminosoetan Espainian hainbat denbora tartetan.

Materialak	Araua	PSV testaren estandarra	Periodoa indarrean	Trafiko astunen kategoriak	Eskatutako PSV minimoa	Eskatutako SCRIM koef. minimoa
Hormigoi bituminosoak (AC) eta nahaste drainatzaileak (PA)	Art. 542. (MOP, 1976b)	NLT 174/72, NLT 175/73	1976 – 1989	Trafiko handiko errep	0,45	-
				Beste errepideak	0,40	
	Art 542. OC 299/1989 (MOPU, 1989b)	NLT 174/72	1989 - 2001	T0 eta T1	0,50	0,65
				T2	0,45	
				T3 eta T4	0,40	
	Art. 542. OC 5/2001 (MFOM, 2001)	NLT 174/72	2001 – 2004	T00	0,55	Nahaste drainatzaile, 60 Hormigoi bituminosoa, 65
				T0, T1	0,50	
				T2	0,45	
				T3, T4, bazterbideak	0,40	
	Art. 542. FOM/891/2004 MFOM, 2004a)	Une 146136, Annex D	2004 – 2008	T00	0,55	Nahaste drainatzaile, 60 Besteak, 65
				T0 eta T1	0,55	
				T2	0,45	
				T3 , T4, bazterbideak	0,40	
Hormigoi bituminosoak (AC)	Art. 542. OC 24/2008 MFOM, 2008)	UNE-EN-1097-8	2008 – 2014	T00 eta T0	56	65
				T1 – T31	50	
				T32, T4, bazterbideak	44	
	Art. 542 FOM/2523/2014 (MFOM, 2015)	UNE-EN-1097-8	2014 - orain	T00 eta T0	56	65
				T1 – T31	50	
				T32, T4, bazterbideak	44	
Nahaste etenak (BBTM A and B)	Art. 543. (MOP, 1976b)	NLT 174/72, NLT 175/73	1976 – 1989	Trafiko handiko errep	0,45	-
				Beste errepideak	0,40	
	Art 543. OC 299/1989 (MOPU, 1989b)	NLT 174/72	1989 - 2001	T0, T1	0,50	0,65
				T2	0,45	
				T3 eta T4	0,40	
	Art. 543. OC 5/2001 (MFOM, 2001)	NLT 174/72	2001 – 2004	T00	0,55	BBTM A, 65 BBTM B, 60
				T0, T1 eta T2	0,50	
				T3, T4, bazterbideak	0,45	
	Art. 543. FOM/891/2004 (MFOM, 2004a)	UNE 146136, Annex D	2004 – 2008	T00	0,55	BBTM A, 65 BBTM B, 60
				T0, T1 eta T2	0,50	
				T3 , T4, bazterbideak	0,45	
Nahaste etenak (BBTM A and B) eta nahaste drainatzaileak (PA)	Art. 543. OC 24/2008 (MFOM, 2008)	UNE-EN-1097-8	2008 – 2014	T00 eta T0	56	BBTM A, 65 BBTM B and PA, 60
				T1 – T31	50	
				T32, T4, bazterbideak	44	
	Art. 543 FOM/2523/2014 (MFOM, 2015)	UNE-EN-1097-8	2014 - orain	T00 eta T0	56	BBTM A, 65 BBTM B, PA, 60
				T1 – T31	50	
				T32, T4, bazterbideak	44	
Kare-esneak	Art. 540. (MOP, 1976b)	NLT 174/72, NLT 175/73	1976 – 1988	Edozein errep	0,40	-
	Art 540. OC 297/1988 (MOPU, 1989b)	NLT 174/7, NLT 175/75	1988 - 2001	T0, T1 and T2	0,50	LB1, LB2, 0,65 LB3 0,60, LB4, 0,55
				T3, T4, bazterbidea	0,45	
	Art. 540. OC 5/2001 (MFOM, 2001)	NLT 174	2001 – 2004	T0, T1 and T2	0,50	LB1, LB2, 65 LB3, 60, LB4, 55
				T3, T4, bazterbidea	0,45	
	Art. 540. FOM/891/2004 (MFOM, 2004a)	UNE 146136, Annex D	2004 – 2011	T0, T1 eta T2	0,50	LB1, LB2, 65 LB3, 60; LB4, 55
				T3 , T4	0,45	
				T3 eta T4-ren bazterbidea	0,40	
	Art. 540. OC 29/2011 (MFOM, 2011b)	UNE-EN-1097-8	2011 – 2014	T00	56	MICROF 11, 8: 65 MICROF 5: 60
				T1 – T31	50	
				T32, T4, bazterbidea	44	
	Art. 540 FOM/2523/2014 (MFOM, 2015)	UNE-EN-1097-8	2014 - orain	T00	56	MICROF 11, 8: 65 MICROF 5: 60
				T1 – T31	50	
				T32, T4, bazterbidea	44	

Oharra: MOPU = Ministerio de Obras Públicas, MFOM = Ministerio de Fomento

Horrez gain, IRI analisia gertatu zen bezala, hasierako saiakeran 20 m-ko tarte indibidual guztien marruskaduraren datuak batera aztertu nahi izan zen, IRI datuetako 100 m-ko tarteekin saiatu zen bezala. Berriro ere, balio azaltzaile bereko tartearen barruan bariantza handia nabaritu zen. Adibidez, trafiko bolumenean, adinean eta bide-zoru sekzioan ezaugarri bereak dituen 500 m-ko tarte baterako, 20 m-ko 25 tarte eskuragarri daude. Eta datu horiek aztertzean, marruskadura balioetarako aldakortasun handia sumatu zen, ezinezkoa eginez bide-zoru ezaguneko tarte guztien azpi-tarteen korrelazioa sortu. Horren ondorioz, berriro ere, batez besteko SCRIM koefizientea eta egitura kalkulatu zen antzeko ezaugarriak (bide-zoru egituran, bide-zoruaren adinean eta trafiko bolumenetan) dituen tarte homogeneoaren 20 m-ko tarteetako balioetatik. Mota honetako analisia, ezaugarri bereko tartearen batez bestekoa kalkulatzea logikoa da literaturan proposatutako modeloetan iragartzen delako batez bestekoa aldagai azaltzaileen bidez eta ez dagoen aldakuntzaren heina. Bariantzaren analisia, IRI analisian azaldu zen bezala (8.4.4 atalean) probabilitatezko modeloetan burutzen da. Horren ondorioz, modelo deterministako batek, tesi honetan garatzen direnaren antzekoak, eragina daukaten aldagaietatik batez besteko balioa iragarri behar du. Aldakortasunaren analisia osatzeko, batez bestekoa kalkulatu den tarte bakoitzean, desbideratze tipikoa (9.2 ekuazioa) kalkulatzen da. Beraz, posiblea da batez bestekoaren inguruan dagoen bariantza jakitea.

$$s = \sqrt{\sigma^2} = \sqrt{\frac{\sum_{i=1}^{N} (x_i - \mu)^2}{N}} \qquad [9.2]$$

Non x_i 100 m-ko tarte bakoitzaren SCRIM koefizientaren datua da, μ SCRIM koefizientearen batez bestekoa da datu guztietarako eta N da datu guztien kopurua kontsideratutako tarte homogeneoan.

Berriro ere, populazioko desbideratze estandarra aukeratu zen eta ez lagineko desbideratze estandarra (*N*-1izendatzailean) datuek ezaugarri bereko tartearen populazio totala adierazten dutelako. Ez da populazio batetik ateratako lagina baizik eta populazio osoa.

Bide-zoru sekzioan, adinean eta trafiko bolumenean ezaugarri bereak dituzten tarteen batez besteko balioak kalkulatu direnean, bi erreiko 127 tarte eskuragarri zeuden Kontserbazio Eremu 1, 2 et 3tik, 2011ko udan neurtutako datuak barne (9.9 taula). Gainera, bide-zoruak guztiz ezagunak ziren 11 galtzada banandutako tarteak ere eskuragarri zeuden.

2011ko udan neurtutako datuak konparatu ziren 2016an errepide-tarte berean neurtutakoekin. 9.10 taulak erakusten ditu tarte bereko eta ezaugarri bereko tarteak bi urteetan.

9.9 taula Kontsideratutako aldagaietan balio ezberdinak dituzten tarteak, SCRIM koefizientearen analisirako aukeratuak Kontserbazio Eremu 1, 2 eta 3tik

Eremu 1				Eremu 2				Eremu 3			
Road	**2e 2016**	**2g 2016**	**2011**	**Road**	**2e 2016**	**2g 2016**	**2011**	**Road**	**2e 2016**	**2g 2016**	**2011**
BI-631	4			BI-623	5	6		BI-624			
BI-634	2			BI-633	12			BI-625	9	3	
BI-635	4			BI-638				BI-630			
BI-735	1		1	BI-732	1			BI-712	1		
BI-737	1		1	BI-2224	5			BI-745			
BI-2101				BI-2301	2			BI-2521			
BI-2120	3			BI-2405	4			BI-2522			
BI-2121	3			BI-2543	8			BI-2617			
BI-2122	2		2	BI-2632				BI-2625	1		
Bi-2153	1			BI-2636				BI-2701	4		1
Bi-2235	5			N-240	15			BI-2757	3		3
Bi-2237	1			N-636				BI-2794	2		1
Bi-2238	5							N-639	5		4
BI-2704	2		2								
BI-2713	1										
BI-2731		2									
Guztira	35	2	6	**Guztira**	52	6	0	**Guztira**	25	3	9
							Bi erreiko tarteak 2016 GUZTIRA	112			
							Galtzada banandutako tarteak 2016 GUZTIRA	11			
							Bi erreiko tarteak 2011 GUZTIRA	15			

9.10 taulan. 2011ko udan eta 2016ko udan neurtutako SCRIM koefizienteen balioen konparaketa errepide-tarte berean

Errepidea	**Hasierako PK-a**	**Amaierako PK-a**	**AADT 2011n**	**H.AADT 2011n**	**SCRIM Koef 2011n**	**AADT 2016an**	**H.AADT 2016an**	**SCRIM Koef 2016an**
BI-2122	24+0500	24+0600	12333	309	63,5789	12286	147	36,2000
BI-2122	27+0000	27+0090	8099	162	60,9333	9598	134	45,4000
BI-2701	25+0900	26+0200	4667	164	66,1636	4493	171	53,4375
BI-2704	8+0500	9+0300	12888	303	72,1596	12052	229	48,2105
BI-2704	23+0400	23+0700	5214	126	86,5566	5273	119	44,7143
BI-2757	0+0000	1+0120	3411	197	64,4468	3718	279	49,8214
BI-2757	2+0820	4+0700	1302	27	72,8235	2243	56	49,3511
BI-2757	4+0700	6+0840	1302	27	60,4000	2243	56	62,3714
BI-2794	23+0850	24+0780	3931	443	58,7500	4175	376	55,2093
BI-735	9+0960	10+0300	15897	636	85,5676	15385	423	46,7778
BI-737	13+0990	14+0150	11355	540	72,8000	12046	301	44,8000
N-639	21+0360	22+0790	4407	177	63,4545	4392	209	61,5135
N-639	22+0790	23+0760	4407	177	63,5789	4392	209	53,4894
N-639	23+0760	23+0860	4407	177	60,9333	4392	209	54,1667
N-639	23+0860	24+0170	4407	177	66,1636	4392	209	44,2000

Ikusten den gisa, 5. kapituluan azaldutako urte-sasoiko ideiak kontuan hartuz, 2011 eta 2016ko balioen arteko bariantza handia ikusten da, 2011ko datuak udan neurtu zirela suposatuz. Haien aldakortasun heina 9..3 taulan adierazitakoaren antzekoa da, eta baita beste tarteetan 2011ko neguko eta 2016ko udako datuen artean dagoen heinaren antzekoa ere. Hori dela eta, Bide-zoruak Kudeatzeko Sisteman 2011ko udan datuak neurtu zirela erregistratuta egon, 2011ko udako datuak baztertzen dira gezurrezkoak direlako 2016ko udako datuekin konparatuz dagoen diferentzia handiagatik. Honek pentsarazten du 2011ko neguan hartu zirela 2011ko beste marruskadura datuekin batera.

Horren ondorioz, 112 behaketa daude, bide-zorua guztiz ezaguneko tarteak, labainketarekiko erresistentzia iragartzeko modeloa garatzeko. 112 datu horietatik, 49 tarte Trazatu Berria bezala sailkatzen dira eta beste 63 tarteak Mantentzea eta Errehabilitazioa (M&R) bezala sailkatzen dira.

Modeloan sartutako aldagaiak honako hauek dira:

- Iragarritako aldagaia, edo mendeko aldagaia, SCRIM koefizientea da, bere balio minimoa adieraziz, udan lortua. ***Mean Summer SCRIM Coefficient*** (*MSSC*) deitzen zaio Transportation Research Laboratory-ko terminologia jarraituz (5. kapitulua).
- Aldagai azaltzaileen, edo aldagai independenteen artean, batzuk kuantitatiboak dira eta beste batzuk kualitatiboak. Aldagai kuantitatiboak honako hauek dira:
    - *Eguneko Batez Besteko Intentsitatea*, ***AADT***, datu-biltzearen urtekoa.
    - *Ibilgailu astunen Eguneko Batez Besteko Intentsitatea,* ***H.AADT****,* proiektuko erreian, datu-biltzearen urtekoa.
    - *Adina,* ***Age,*** datu-biltzearen urtearen eta trazatu berria edo mantentze edo errehabilitazio lana amaitu zen urtearen arteko diferentzia.
    - *Adina erreala,* ***R.Age****,* datu-biltzearen urtearen eta trazatu berria edo mantentze edo errehabilitazio lana amaitu zen urtearen arteko diferentzia, urteak formatu dezimalean azalduta. Adibidez, datu-biltzearen kanpaina ekainaren azken egunetan gauzatu zenez, 2016,5 bezala sartu da. Beraz, bide-zoruaren adina zehatzagoa jakitea posiblea da.
    - *Ibilgailu guztiak,* ***TotVeh****,* sekziotik igaro diren ibilgailu totalen kopuru metatua, tartea eraiki zenetik (trazatu berrien tarteen kasuan) edo konponduta izan zenetik datu-biltze kanpainara arte, mila ibilgailutan.
    - *Ibilgailu astun guztiak,* ***TotH.Veh****,* “Ibilgailu guztiak”-en antzeko baina ibilgailu astunak tarteko proiektuko erreian, mila ibilgailu astunetan.
    - *Beharrezko Polished Stone Value*, $\boldsymbol{PSV_{req}}$, agregakinen eskatutako PSV minimo errodadura geruzan, geruza errodaduraren materialaren, tartearen trafiko astuneko kategoriaren (Ministerio de Fomento, 2003b) eta tartea zerbitzuan jarri zen urtearen arabera (9.8 taula).
    - *Prezipitazioaren datua*, ***Rain15***, prezipitazioaren datua, mm-tan, datu-biltzearen aurreko 15 egunetan neurtutako euri kopurua errepidetik hurbilen dagoen estazio meteorologikoan.

- *Geruza bituminosoen lodiera totala*, ***TotBit***, bide-zoruaren sekzioaren geruza bituminosoen lodiera totala, cm-tan.

Aldagai kualitatiboak honako hauek dira:

- *Bide-zoru mota*, ***PaveType***, bi aukeren artean berezituz, bide-zoru malguak (1) eta bide-zoru erdi-zurrunak (2). Esan beharra dago, bide-sekzioari buruzko informazio gehiago eskuragarri dagoela baina, hasteko bide-zoruaren motaren eragina analizatu nahi izan da.
- *Errodadura geruzaren izendatzea*, ***SurfDen***, eta *errodadura geruza mota*, ***SurfType***. Aldagai hauek berezitzen dira errodadura geruzan egon ahal diren materialen izendatzearen artean, eta antzeko ezaugarrien arabera bilduta badaude, 9.11 taulan erakusten den bezala.

9.11 taula. Errodadura geruza mota posibleak eta haien errodadura geruza taldeak

Errodadura geruzaren izendatzea (SurfDen)	Errodadura geruza mota (SurfType)
AC 16 surf S (1)	Hormigoi bituminosoa (AC) (1)
AC 22 surf S (2)	Hormigoi bituminosoa (AC) (1)
AC 16 surf D (3)	Hormigoi bituminosoa (AC) (1)
AC 22 surf D (4)	Hormigoi bituminosoa (AC) (1)
BBTM 11A (5)	Nahaste etenak eta drainatzaileak (BBTM&PA) (2)
BBTM 11B (6)	Nahaste etenak eta drainatzaileak (BBTM&PA) (2))
PA 11 (7)	Nahaste etenak eta drainatzaileak (BBTM&PA) (2))
LB2 (8)	Kare-esnea (3)

Errodadura geruzaren izendatzeak (*SurfDen*), 9.11 taularen ezkerreko zutabea, eskuineko zutabean erakusten den bezala bildu ziren zeren eta, azterlan batzuetan oso datu gutxi zeuden errodadura geruzaren izendatzearen kasu batzuetan eta ahiago izan da antzeko ezaugarrien arabera biltzea. Adibidez, nahiz eta nahaste etenak eta drainatzaileak ezberdinak izan, ezaugarri batzuk partekatzen dituzten, ura drainatzeko ahalmena daukate eta Espainiako arauen artikulu berdinean definituta daude (9.8 taula). Berdina gertatze da hormigoi bituminosozko geruzekin, gradazio ezberdinak dituzte, erdi-trinkoa (S) edo trinkoa (D), baina ezaugarri batzuk partekatzen dituzte. Ondorioz, bildu ziren datu bat edo bi datu dituzten taldeak ekiditeko.

9.3.1.1. Datu-analisia Trazatu Berria tarteetan

49 datu daude guztiz ezaguna den bide-zoru sekziorekin. 19 datu bide-zoru malgukoak dira eta 30 erdi-zurrunekoak. Errodadura geruzari dagokionez, 27 datuetan hormigoi bituminosoa dago (AC) (batez ere AC 16 surf S) eta 20 datuek nahaste etenak edo drainatzaileak dituzte. Errodadura geruzaren eragina *SurfType*-ren mailak (edo kategoriak) (9.11 taula) erabiliz aztertzea erabaki da.

Lehenengo eta behin, esplorazio-analisia gauzatzen da. Aldagai bakoitzeko datu estatistiko nagusiak kalkulatzen dira. 9.12 taulak aldagai kuantitatiboen estatistika nagusiak erakusten ditu. Banaketa normala Shapiro-Wilk estatistikoa eta KolgomorovSmirnov estatistikoen bitartez egiaztatzen da (9.13 taula). Ikusten den bezala, MSSC da banaketa normala daukan aldagai bakarra Shapiro-Wilk estatistikoa eta KolgomorovSmirnov estatistikoek egiaztatuta.

9.12 taula. Aldagai kuantitatiboen esplorazio-analisia Trazatu Berria tarteetan

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Batez bestekoa		50,39	8123,4	463,8	8,9	8,75	19535,99	1289,17	47,18	31,06	16,88
Errore estandarra		1,09	1019,1	63,14	0,99	0,99	2785,77	200,07	0,59	1,92	0,81
%95eko KT-ak	**Beheko**	48,2	6074,3	336,82	6,9	6,77	13934,8	886,92	46	27,2	15,24
	Goiko	52,58	10172,5	590,73	10,9	10,74	25137,15	1691,43	48,37	34,91	18,51
Bariantza		58,11	50891283	195360	48,47	47,72	380264605	1961249	17,07	179,76	32,44
Desbideratze tip.		7,62	7133,8	442	6,962	6,9	19500,4	1400,45	4,13	13,41	5,696
Minimoa		27,33	669	23	1	0,8	1453,5	43,01	40	2,1	8
Maximoa		64,62	31299	1408	29	28,6	98892	4447,04	50	45,8	33
Anplitudea		37,29	30630	1385	28	27,8	97438,5	4404,03	10	43,7	25
Kuartileko heina		9,4	7767	804	9	8,66	20228,3	2213,96	6	4,3	8
Simetria		-0,36	1,62	0,75	1,18	1,21	2,18	1,02	-0,96	-1,48	0,69
Kurtosia		0,69	2,53	-0,96	0,91	0,91	5,85	-0,22	-0,84	0,52	0,25

9.13 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat Trazatu Berria tarteetan

Aldagaiak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)
MSSC	0,089	49	0,200*	0,971	49	0,270
AADT	0,383	49	< 0,001	0,675	49	< 0,001
H.AADT	0,192	49	< 0,001	0,827	49	< 0,001
Age	0,242	49	< 0,001	0,816	49	< 0,001
R.Age	0,405	49	< 0,001	0,655	49	< 0,001
TotVeh	0,22	49	< 0,001	0,87	49	< 0,001
TotH.Veh	0,212	49	< 0,001	0,867	49	< 0,001
PSV	0,177	49	0,001	0,78	49	< 0,001
Rain 15	0,239	49	< 0,001	0,809	49	< 0,001
TotBit	0,14	49	0,018	0,936	49	0,01

[a] Lilliefors Esangura zuzenketa

* Hau da benetako esanguraren beheko muga.

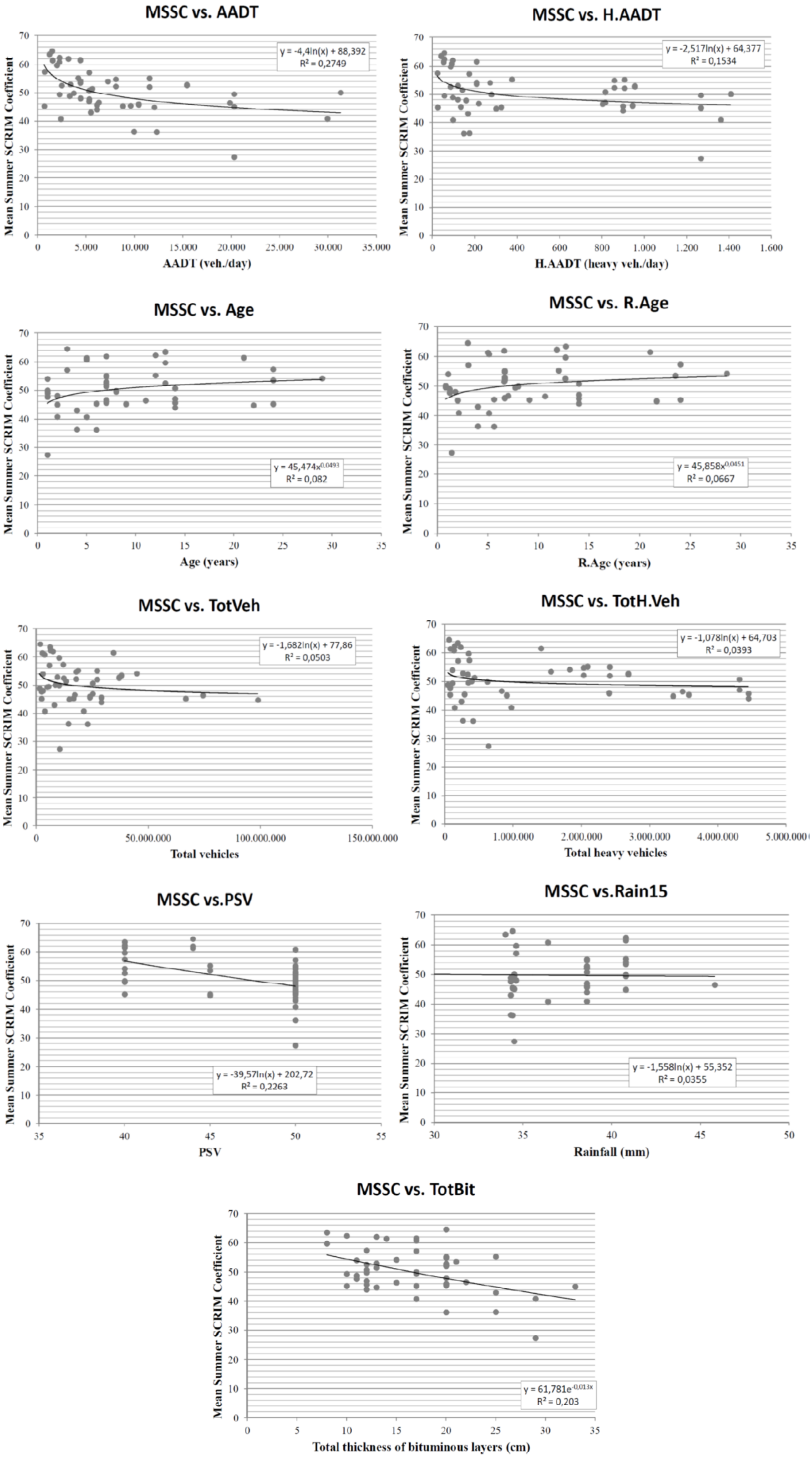
Mendeko aldagai eta aldagai independenteen arteko korrelazioa Pearson-en *R* korrelazio-koefizientearen bidez gauzatzen da, korrelazioaren esangura adieraziz (9.14 taula). Esplorazio-analisia denez, mendeko aldagaiaren eta aldagai independente bakoitzaren arteko korrelazio-koefizientea adierazten da, eta ez aldagai independentearen artean.

9.14 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) Trazatu Berria tarteetan.

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
AADT	-0,464	0,001
H.AADT	-0,36	0,011
Age	0,218	0,132
R.Age	0,212	0,144
TotVeh	-0,183	0,209
TotH.Veh	-0,169	0,247
PSV	-0,479	< 0,001
Rain 15	-0,15	0,279
TotBit	-0,429	0,002

9.3 irudiak mendeko eta aldagai independenteen arteko korrelazioa grafikoki. 9.14 taulatik ikus daiteke *MSSC*-rekiko korrelazio onenak dituzten aldagai independenteak PSV_{req}, *AADT*, *TotBit* eta *H.AADT*. Aldagai hauek esanguratsuak dira korrelazioan (p-balioa < 0,05). 9.3 irudiak erakusten du grafikoki mendeko aldagaiaren eta kontsideratutako aldagai independente bakoitzaren arteko korrelazioa eta datuak hoberen doitzen dituen kurba. *AADT* eta *H.AADT* funtzio logaritmiko bat korrelazioak hobetzen ditu. Aldagai hauetarako funtzio kubiko batek sortzen du R^2 hobeago bat, baina kurbaren itxura ez da logikoa aurreko azterlanen arabera, eta beraz baztertu da. Datuak hoberen doitzen dituen funtzioa da baina ez da literaturan aurkitutako patroia (Szatkowski and Hosking, 1972; NZTA, 2013a). PSV_{req}, nahiz eta hobekuntza txikiak lortu, erlazio lineala mantendu da. *Age*, *R.Ag*e, *TotVeh* eta *TotH.Veh*, edozein motatako funtzioak determinazio-koefiziente baxuak erakusten ditu, beti 0,12 baino baxuagoak (R^2 < 0,12), aldagai hauek ez daukatela eragin handirik marruskaduran adieraziz. Horren ondorioz, sasoiko aldakuntzak (5.33 eta 5.34 irudiak) eta labainketarekiko erresistentziaren portaera denboran zehar (5.31 irudia) egiaztatzen dira. Prezipitazioaren datuak, *Rain15*, ere ez dauka eraginik marruskaduran.

Mendeko aldagaia (*MSSC*) eta aldagai independente kualitatiboaren *PaveType* and *SurfType*) arteko korrelazioa ere egiaztatu da. Lehenengo eta behin,datuen esplorazio-analisia gauzatu da aldagai independenteen mailen arabera sailkatuz (9.15 taula). Mailen bidez egindako taldeen banaketa normala ere egiaztatu da (9.16 taula).



9.3 irudia. Mendeko aldagai eta aldagai independenteen arteko grafikoak eta datuak hoberen doitzen duen kurba Trazatu Berria tarteetan.

9.15 taula. Aldagai kualitatiboen esplorazio-analisia Trazatu Berria tarteetan, mailaz bilduta mendeko aldagaiarekiko (MSSC)

Aldagai kualitatiboak		**PaveType**		**SurfType**	
Mailak		**Malgua (1)**	**Erdi-zurruna (2)**	**AC (1)**	**BBTM&PA (2)**
N		19	30	27	20
Batez bestekoa		49,227	51,124	51,413	49,131
Errore estandarra		2,266	1,068	1,851	0,835
%95eko Konfiantza-tarteak	**Beheko**	44,467	48,940	47,609	47,394
	Goiko	53,987	53,307	55,217	50,868
Bariantza		97,542	34,194	92,460	15,342
Desbideratze tipikoa		9,876	5,848	9,616	3,917
Minimoa		27,333	36,300	27,333	40,833
Maximoa		64,625	63,500	64,625	55,102
Anplitudea		37,292	27,200	37,292	14,268
Kuartileko heina		14,389	7,223	15,583	6,674
Simetria		-0,272	0,157	-0,620	-0,172
Kurtosia		-0,309	0,849	-0,090	-0,816

9.16 taula. Normaltasun testak aldagai independente kualitatiboentzat Trazatu Berria tarteetan mailaz bilduta mendeko aldagaiarekiko (MSSC)

Aldagai independente kualitatiboak	**Aldagaiaren mailak**	**Kolmogorov-Smirnov**[a]			**Shapiro-Wilk**		
		Estatistikoa	**Askatasun-graduak**	**Esangura (p-balioa)**	**Estatistikoa**	**Askatasun-graduak**	**Esangura (p-balioa)**
PaveType	Malgua	0,131	19	0,200*	0,953	19	0,452
	Erdi-zurruna	0,115	30	0,200*	0,957	30	0,261
SurfType	Hormigoi bituminosoa	0,102	27	0,200*	0,948	27	0,191
	BBTM & PA	0,133	22	0,200*	0,958	22	0,441

[a] Lilliefors esangura zuzenketa

* Hau da benetako esanguraren beheko muga.

Horren ostean, Levene testa gauzatu da faktoreek (aldagai kualitatiboek) definitzen dituzten mailak bariantza homogeneoko populazioetatik datozen hipotesia egiaztatzeko. Levene estatistikoarekin erlazionatutako esangurak bariantzaren homogeneotasuna egiaztatzen ahalbidetzen du: esangura 0,05 baino txikiagoa bada, bariantzaren homogeneotasunaren hipotesia baztertzen da. Test hau *t* testa batekin osatzen da faktoreen mailen arteko batez bestekoak konparatzeko, batez bestekoen arteko diferentziaren bidez. Aldagai kualitatibo bakoitzean bi maila daudenez, (bide-zoru malguak eta erdi-zurrunak *PaveType* aldagaian eta AC eta BBTM&PA *SurfType* aldagaian,, *t* testa aplikatzen da. Bi maila baino gehiago badaude, ANOVA bat (*Analysis of Variance*) (Bariantzaren analisia) gauzatzen da.

Bide-zoru malgu eta erdi-zurrunerako bariantza ez da homogeneoa eta batez bestekoak berdinak dira (p-balioa > 0,05) (9.17 taula). Haien arteko diferentzia balio positiboetatik balio negatiboetara pasatu ahal dela ikusten da ere, %95eko konfiantza-mailaz, 9.17 taulan. Hori dela eta, aldagai honek ez daukala eraginik marruskaduran esan daiteke, Errodadura geruza motarako, antzeko ondorioak lortzen dira: Levene estatistikoak bi populazioen bariantza ez dela homogeneoa adierazten du, baina ez dago batez bestekoen arteko diferentziarik, %95eko konfiantza mailaz (9.18 taula).

9.17 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako PaveType-ren mailetarako MSSC-rekiko Trazatu Berria tarteetan

Levene estatistikoa			**t testa batez bestekoen berdintasunerako**						
F	**Esang.**		**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Goiko**
8,909	0,004	Bariantza berdinak suposatuta	-0,846	47	0,402	-1,896	2,241	-6,407	2,613
		Bariantza berdinak ez suposatuta	-0,757	26,082	0,456	-1,897	2,504	-7,045	3,251

9.18 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako SurfType-ren mailetarako MSSC-rekiko Trazatu Berria tarteetan

Levene estatistikoa			**t testa batez bestekoen berdintasunerako**						
F	**Esang.**		**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Goiko**
13,721	0,001	Bariantza berdinak suposatuta	1,043	47	0,302	2,282	2,187	-2,119	6,682
		Bariantza berdinak ez suposatuta	1,124	35,828	0,268	2,282	2,030	-1,836	6,400

Aldagai batzuk eraldatu ziren, 9.3 irudiaren ondorioen arabera. Espezifikoki, natural logaritmoak aplikatu ziren *AADT* eta *H.AADT* aldagaietan. Erregresio lineal anizkoitzeko modeloak garatzen saiatu ziren, esangura globala (horretarako F estatistikoaren p-balioak 0,05 baino txikiagoa izan behar du) eta sartutako aldagai guztiak esanguratsuak dira indibidualki (*t* testaren baten bitartez) egiaztatzeko. Horren helburua da zein aldagai kuantitatibo sartzen diren *MSSC* iragartzeko modeloan. Baldintza horiekin garatutako modelorik onena Ln*AADT* (*AADT*-ren logaritmo naturala) baino ez zuen erabiltzen eta bere determinazio-koefizientea zen eta bere doitutako determinazio-koefizientea zen, modeloa *MSSC*-ren aldakuntzaren %25a azaltzeko gai zela adieraziz.

Orduan, Erregresio Lineal Orokor Anizkoitzeko (GLM, *General Linear Muliple*) modelo batzuk, 7.5.5 atalean definituak saiatu ziren, aldagai kualitatibo eta kuantitatiboak erabiliz, eta horien bidez, ikus daiteke aldagai kualitatiboak esanguratsuak diren ala ez. Adibidez, GLM bat saiatu zen, Ln*H.AADT*, PSV_{req} eta Ln*AADT* aldagai kuantitatibo bezala (lehenengo biak Szatkowski and Hosking (1972)–ren modeloan agertzen direlako eta azkena erregresio lineal anitzeko modeloan sartu zen aldagai bakarra zelako) eta *PaveType* eta *SurfType* as aldagai kualitatibo bezala. 9.19 taula erakusten ditu bi aldagai kualitatiboek sortutako batez bestekoak eta desbideratze tipikoak.

9.19 taula. Batez bestekoa eta desbideratze tipikoa bi aldagai kualitatiboaren mailetan mendeko aldagaiarekiko (MSSC) Trazatu berria tarteetan

SurfType	PaveType	Batez bestekoa	Desbideratze tipikoa	N
Hormigoi bituminosoa (AC) (1)	Malgua (1)	49,955	10,807	15
	Erdi-zurruna (2)	53,235	7,961	12
	Guztira	51,413	9,616	27
Nahaste etenak eta drainatzaileak (BBTM&PA) (2)	Malgua (1)	46,497	5,261	4
	Erdi-zurruna (2)	49,716	3,475	18
	Guztira	49,131	3,917	22
Guztira	Malgua (1)	49,227	9,876	19
	Erdi-zurruna (2)	51,124	5,848	30
	Guztira	50,388	7,623	49

Proposatutako modeloak 9.3 ekuazioaren adierazpena dauka.

$$MSSC = Intercept + LnAADT + LnH.AADT + PSV_{req} + SurfType +$$

$$+ PaveType + SurfType * PaveType \qquad [9.3]$$

Modeloaren Subjektu-arteko Efektuen testa eta parametroen estimazioak 9.20 taula eta 9.21 taulan erakusten dira, hurrenez hurren. Modeloaren determinazio koefizientea 0,378 da (R^2 = 0,378) eta doitutako R^2 0,289 da. Analisi osoa ez da erakusten, behin-behineko modeloa delako, aldagai esanguratsuak eta ez esanguratsuak identifikatzeko erabilita.

9.20 taula. 9.3 ekuazioaren modeloaren Subjektu-arteko Efektuen testa Trazatu Berria tarteetan

Jatorria	Karratuen batuketa III mota	Askatasun -graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	1054,51	6	175,75	4,255	0,002	0,378	25,531	0,961
Konstantea	2432,38	1	2432,38	58,89	< 0,001	0,584	58,89	1
LnAADT	87,23	1	87,23	2,112	0,154	0,048	2,112	0,295
PSVreq	126,97	1	126,97	3,074	0,087	0,068	3,074	0,403
LnH.AADT	0,59	1	0,59	0,014	0,905	< 0,001	0,014	0,052
SurfType	56,62	1	56,62	1,371	0,248	0,032	1,371	0,208
PaveType	41,59	1	41,59	1,007	0,321	0,023	1,007	0,165
SurfType * PaveType	9,87	1	9,87	0,239	0,628	0,006	0,239	0,077
Errorea	1734,74	42	41,30					
Total	127199,61	49						
Total zuzendua	2789,25	48						

9.21 taula. 9.3 ekuazioaren modeloaren parametroen estimazioak Trazatu Berria tarteetan

Parametroak	B	Errore estandarra	t	Esang	%95 konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Goiko	Beheko			
Konstantea	115,871	15,352	7,548	< 0,001	84,889	146,853	0,576	7,548	1,000
LnAADT	-3,602	2,479	-1,453	0,154	-8,605	1,400	0,048	1,453	0,295
PSVreq	-0,631	0,360	-1,753	0,087	-1,358	0,095	0,068	1,753	0,403
LnH.AADT	-0,246	2,057	-0,120	0,905	-4,398	3,906	< 0,000	0,120	0,052
[SurfType=1]	-4,766	3,839	-1,242	0,221	-12,513	2,981	0,035	1,242	0,228
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.
[PaveType=1]	-3,412	3,751	-0,910	0,368	-10,983	4,158	0,019	0,910	0,144
[PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=1] * [PaveType=1]	2,380	4,869	0,489	0,628	-7,447	12,206	0,006	0,489	0,077
[SurfType=1] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako.

Ikusten den bezala, Ln*H.AADT* ez da esanguratsua, F estatistikoa eta bere esangura indibidualerako burututako *t* testaren arabera. *PaveType* ere ez da esanguratsua bi estatistikoen arabera. Horren ondorioz, bi aldagai hauek modelotik baztertu behar dira. Ln*AADT*, PSV_{req} eta *SurfType* aldagaiak dituen modelo batek (9.4 ekuazioa) 9.22 taulan eta 9.23 taulan erakutsitako emaitzak sortzen ditu.

$$MSSC = Intercept + LnAADT + PSV_{req} + SurfType \qquad [9.4]$$

9.22 taula. 9.4 ekuazioaren modeloaren Subjektu-arteko Efektuen testa Trazatu Berria tarteetan

Jatorria	Karratuen batuketa III mota	Askatasun -graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	1011,033	3	337,011	8,528	< 0,001	0,362	25,585	0,990
Konstantea	2814,599	1	2814,599	71,227	< 0,001	0,613	71,227	1,000
LnAADT	308,031	1	308,031	7,795	0,008	0,148	7,795	0,780
PSVreq	164,534	1	164,534	4,164	0,047	0,085	4,164	0,515
SurfType	159,316	1	159,316	4,032	0,051	0,082	4,032	0,502
Errorea	1778,217	45	39,516					
Total	127199,61	49						
Total zuzendua	2789,249	48						

Modeloaren determinazio koefizientea 0,362 (R^2 = 0,362) eta doitutako R^2 0,320 da. Determinazio-koefiziente txarrago bat dauka, baina doitutako R^2 hobeagoa da, aldagai gutxiago erabiltzen dituelako, eta gainera modeloan sartutako aldagai guztiak esanguratsuak dira. Beste aukerak saiatu dira hainbat aldagai konbinatuz, korrelazio hobeagoak dituztenen artean, eta R^2 hobeagoak lortzen dira baina aldagai guztiak ez dira esanguratsuak. Egiaztatu da *PaveType* aldagaia ez dela marruskaduran eragina duen faktorea, mailen batez besteen arteko diferentziagatik eta Erregresio Lineal Orokor Anizkoitzeko modeloetan esangura txikia erakutsi duelako. Beste aldetik, aldagaiaren mailek batez besteko berdina daukate baina esangura handia erakutsi du GLM-tan, batez ere, *PaveType* eta *LnH.AADT* ez daukan modeloan.

9.23 taula. 9.4 ekuazioaren modeloaren parametroen estimazioak Trazatu Berria tarteetan

Parametroak	B	Errore estandarra	t	Esang	%95eko konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Goiko			
Konstantea	116,527	14,27	8,166	< 0,000	87,785	145,269	0,597	8,166	1,000
LnAADT	-3,897	1,396	-2,792	0,008	-6,708	-1,086	0,148	2,792	0,780
PSV	-0,635	0,311	-2,041	0,047	-1,261	-0,008	0,085	2,041	0,515
[SurfType=1]	-4,61	2,296	-2,008	0,051	-9,234	0,014	0,082	2,008	0,502
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

9.3.1.1.1. Trazatu Berria tarteen analisia 2 urte edo gehiagoko adina errealeko datuekin

Kokkalis (1998)-ek erakutsi zuen bide-zoruko marruskaduraren oreka fasea edo periodo egonkorra,2 urte eta gero lortzen zela. Bide-zoruko errodadura geruzak aztertzean, Navarro *et al.* (2011)-ek frogatu zuen 5.31 irudian deskribatutako faseak Gipuzkoan (Bizkaiaren klima berdineko Euskal Autonomia Erkidegoko beste probintzia) gertatzen direla eta 2 urte behar dira oreka fasera heltzeko. Horren ondorioz, oreka fasera heldu ez diren tarteak baztertzea erabaki zen aurreko analisia ez eraldatzeko. Hori dela eta, 2 urte baino gutxiagoko tarteak baztertu ziren, guztira 7 tarte. Beraz, 2 urte edo gehiagoko adina errealeko datuak ($R.Age \geq 2$) aztertu dira berriro. Gauzatutako analisia aurreko atalarenaren antzeko da. 9.24 taulak erakusten du aldagai kuantitatiboen esplorazio-analisia 42 datuetarako. Berriro ere, mendeko aldagaia (*MSSC*) da banaketa normal duen aldagai kuantitatibo bakarra (9.25 taula).

9.24 taula. Aldagai kuantitatiboen esplorazio-analisia 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Batez bestekoa		51,04	7283,74	431,81	10,19	10,01	21980,1	1464,2	46,71	31,26	16,93
Errore estandarra		1,13	943,98	62,68	1,03	1,03	3088,6	221,96	0,66	2,12	0,87
%95eko KT-ak	**Beheko**	48,75	5377,33	305,23	8,11	7,93	15742,5	1016,0	45,38	26,98	15,18
	Goiko	53,33	9190,15	558,39	12,27	12,09	28217,6	1912,5	48,05	35,54	18,68
Bariantza		53,91	37426356	164993	44,7	44.41	400656120	2069107	18,4	188,3	31,6
Desbideratze tip.		7,34	6117,71	406,19	6,69	6,66	20016,4	1438	4,29	13,72	5,62
Minimoa		36,2	669	23	2	2	1760	60,11	40	2,1	8
Maximoa		64,62	29916	1361	29	28,6	98892	4447,04	50	45,8	33
Anplitudea		28,43	29247	1338	27	26,6	97132	4386,93	10	43,7	25
Kuartileko heina		10,33	7978	775	7	7,4	19780	2175	7	4,4	8
Simetria		0,04	1,69	0,71	1,24	1,25	2,07	0,80	-0,72	-1,43	0,64
Kurtosia		-0,66	3,60	-1,02	0,86	0,81	5,23	-0,63	-1,28	0,35	0,36

9.25 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Aldagaiak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)
MSSC	0,116	42	0,175	0,969	42	0,296
AADT	0,182	42	0,001	0,848	42	< 0,001
H.AADT	0,248	42	< 0,001	0,814	42	< 0,001
Age	0,231	42	< 0,001	0,853	42	< 0,001
R.Age	0,222	42	< 0,001	0,849	42	< 0,001
TotVeh	0,171	42	0,003	0,799	42	< 0,001
TotH.Veh	0,215	42	< 0,001	0,844	42	< 0,001
PSV	0,373	42	< 0,001	0,694	42	< 0,001
Rain 15	0,365	42	< 0,001	0,674	42	< 0,001
TotBit	0,139	42	0,042	0,939	42	0,027

[a] Lilliefors esangura zuzenketa

Mendeko aldagai eta aldagai independenteen arteko korrelazioei dagokienez (9.26 taula), erlazioek Pearson-en *R* korrelazio-koefiziente hobeagoak erakusten dituzte, 9.14 taularekin konparatuz.

Mendeko aldagaiaren (MSSC) eta aldagai independente bakoitzaren arteko grafikoen datuak hoberen doitzen dituen kurbak aztertzen, 9.27 taulan erakutsitako ekuazioak aukeratu dira korrelazioak hobetzeko. Kurbarik onena ez da beti aukeratu. Normalean ekuazio koadratikoak edo kubikoak hoberen doitzen dituzte datuak baina ez horiek ez dira literaturan aurkitutako patroiak. Beste batzuetan, korrelazio lineala eta beste korrelazioen arteko determinazio-koefizientearen hobekuntza oso baxua bada ($\Delta R^2 < 0{,}05$), modelo lineala mantendu da, aldagai independenteak ez direla eraldatu adieraziz.

9.26 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
AADT	-0,493	0,001
H.AADT	-0,322	0,038
Age	0,152	0,336
R.Age	0,149	0,345
TotVeh	-0,274	0,079
TotH.Veh	-0,257	0,100
PSV	-0,491	0,001
Rain 15	-0,138	0,385
TotBit	-0,362	0,018

9.27 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duten ekuazioak 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Aldagai independenteak	Ekuazio-mota	Modeloaren laburpena						Parametroen estimazioak		
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
AADT	Logaritmikoa	0,284	15,828	1	40	< 0,001	88,501	-4,382	-	-
H.AADT	Logaritmikoa	0,141	6,591	1	40	0,014	63,84	-2,33	-	-
Age	Lineala	0,023	0,950	1	40	0,336	49,337	0,167	-	-
R.Age	Lineala	0,022	0,914	1	40	0,345	49,392	0,165	-	-
TotVeh	Logaritmikoa	0,126	5,776	1	40	0,021	77,916	-2,794	-	-
TotH.Veh	Alderantzizkoa	0,105	4,716	1	40	0,036	49,169	609,596	-	-
PSV	Lineala	0,241	12,706	1	40	0,001	90,295	-0,84	-	-
Rain15	Lineala	0,019	0,771	1	40	0,385	53,342	-0,074	-	-
TotBit	Lineala	0,131	6,044	1	40	0,018	59,048	-0,473	-	-

Aldagai independente kualitatiboen esplorazio-analisia gauzatu zen (9.28 taula).

9.28 taula. Aldagai kualitatiboen esplorazio-analisia 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan, mailaz bilduta mendeko aldagaiarekiko (MSSC)

		PaveType		SurfType	
		Malgua (1)	Erdi-zurruna (2)	AC (1)	BBTM&PA (2)
N		17	25	25	17
Batez bestekoa		50,588	51,349	52,513	48,876
Errore estandarra		2,138	1,266	1,723	1,040
%95eko Konfiantza-tarteak	**Beheko**	46,055	48,737	48,956	46,671
	Goiko	55,120	53,961	56,070	51,081
Bariantza		77,712	40,038	74,254	18,389
Desbideratze tipikoa		8,815	6,328	8,617	4,288
Minimoa		36,200	36,300	36,200	40,833
Maximoa		64,625	63,500	64,625	55,102
Anplitudea		28,425	27,200	28,425	14,268
Kuartileko heina		15,078	8,259	15,852	6,971
Simetria		0,120	0,042	-0,367	-0,076
Kurtosia		-1,391	0,412	-0,950	-1,200

PaveType aldagaiaren bi mailak eta *SurfType* aldagaiaren bi mailek banaketa normala da daukate (9.29 taula). Faktore bakoitzaren araberako mailak, ez daukate bariantza homogeneorik, baina aldagai bakoitzeko mailek batez besteko berdinak dauzkate (9.30 taula eta 9.31 taula, hurrenez hurren). Hala eta guztiz ere, bide-zoruaren mota (malgua edo erdi-zurruna) batez bestekoaren berdintasunerako antzeko esangura mantendu arren, errodadura geruzaren moten kasuan, hormigoi bituminoso eta nahaste etenak eta drainatzaileen arteaz berezitzen dituena, batez bestekoaren diferentziaren esangura murriztearen joera erakutsi du datu “gazteak” kendu direnean. Horrek adierazten du errodadura geruza mota eragina duen faktorea izan daitekeela, bi mailen arteko ezberdintasuna handiagoa denez.

9.29 taula. Normaltasun testak aldagai independente kualitatiboentzat 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan, mailaz bilduta mendeko aldagaiarekiko (MSSC)

Aldagai independente kualitatiboak	Aldagaiaren mailak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
		Estatistikoa	Askatasun-graduak	Esangura (p-balioa)	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)
PaveType	Malgua (1)	0,204	17	0,059	0,923	17	0,168
	Erdi-zurruna (2)	0,123	25	0,200*	0,958	25	0,384
SurfType	AC (1)	0,118	25	0,200*	0,94	25	0,150
	BBTM & PA (2)	0,203	17	0,061	0,92	17	0,147

[a] Lilliefors esangura zuzenketa

* Hau da benetako esanguraren beheko muga.

9.30 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako PaveType-ren mailetarako MSSC-rekiko 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Levene estatistikoa			**t testa batez bestekoen berdintasunerako**						
F	**Esang..**		**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Goiko**
6,749	0,013	Bariantza berdinak suposatuta	-0,326	40	0,746	-0,761	2,334	-5,478	3,955
		Bariantza berdinak ez suposatuta	-0,306	26,968	0,762	-0,761	2,485	-5,860	4,337

9.31 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako SurfType-ren mailetarako MSSC-rekiko 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan.

Levene estatistiko			**t testa batez bestekoen berdintasunerako**						
F	**Esang.**		**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Goiko**
8,720	0,005	Bariantza berdinak suposatuta	1,606	40	0,116	3,637	2,265	-0,941	8,215
		Bariantza berdinak ez suposatuta	1,807	37,25	0,079	3,637	2,013	-0,441	7,715

Erregresio lineal anizkoitzeko modelo baten saiakerak, Pausoz Pauso Hautaketa eta Aurrera pausoz pauso funtzioen bidez, berriro ere erakusten du modeloan sartzen den aldagai bakarra Ln*AADT* dela, korrelazio handiena duen aldagaia, eta ez dela posible beste aldagaiak sartzea koefizienteen esangura beharrezko izanik. Modeloaren estatistikoak 9.32 taulan erakusten dira.

Berriro Erregresio Lineal Orokor Anizkoitzeko (GLM) modeloak saiatu ziren aldagai kualitatiboak (*PaveType* and *SurfType*) eta Pearson-en koefizientearen araberako *MSSC*-rekin hoberen erlazionatzen diren aldagai independente kuantitatiboak (eta eraldatuak) sartuz. Lehenengo saiakera 9.3 ekuazioarekin burutu zen. Lortutako balioak 9.33 eta 9.34 tauletan erakusten dira. Determinazio koefizientearen balioa (R^2) 0,354 da eta doitutako R^2-rena 0,243.

9.32 taula. Proposatutako erregresio lineal anizkoitzaren modeloaren bariantzaren analisia (ANOVA) 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Bariantzaren analisia						
Iturria	**Askatasun-graduak**	**Karratuen batuketa**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**
Modeloa	1	626,632	626,632	15,828	< 0,001	1,641
Errorea	40	1583,556	39,589			
Total zuzenduta	41	2210,188				
Estimazioaren errore estandarra	**Aldakuntza-koefizientea**	**R**	**R^2**	**R^2_{adj}**	**Kolinealtasunaren estatistikoak**	
					Tolerantzia	**VIF**
6,292	15,828	0,532	0,284	0,266	1,000	1,000
Parametroen estimazioak						
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioa**	**p-balioa**	**%95eko konfiantza-tarteak**	
Konstantea	88,501	9,465	9,350	< 0,001	69,370	107,631
Ln*AADT*	-4,382	1,101	-3,979	< 0,001	-6,608	-2,156

9.33 taula. 9.3 ekuazioaren modeloaren Subjektu-arteko Efektuen testa 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Jatorria	**Karratuen batuketa III mota**	**Askatasun-graduak**	**Batez besteko karratua**	**F**	**Esang.**	**Eta karratu partziala**	**Ez zentralitatearen parametroa**	**Ikusitako botera**
Modelo zuzendua	781,381	6	130,23	3,19	0,013	0,354	19,141	0,871
Konstantea	2037,321	1	2037,321	49,906	< 0,001	0,588	49,906	1
LnAADT	104,558	1	104,558	2,561	0,119	0,068	2,561	0,344
PSVreq	79,283	1	79,283	1,942	0,172	0,053	1,942	0,273
LnH.AADT	7,805	1	7,805	0,191	0,665	0,005	0,191	0,071
SurfType	2,115	1	2,115	0,052	0,821	0,001	0,052	0,056
PaveType	12,375	1	12,375	0,303	0,585	0,009	0,303	0,083
SurfType * PaveType	5,688	1	5,688	0,139	0,711	0,004	0,139	0,065
Errorea	1428,806	35	40,823					
Total	111628,3	42						
Total zuzendua	2210,188	41						

9.34 taula. 9.3 ekuazioaren modeloaren parametroen estimazioak 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Parametroak	B	Errore estandarra	t	Esang	%95 konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Goiko			
Konstantea	108,63	16,116	6,741	< 0,001	75,914	141,347	0,565	6,741	1,000
LnAADT	-4,489	2,805	-1,6	0,119	-10,184	1,205	0,068	1,6	0,344
PSVreq	-0,52	0,373	-1,394	0,172	-1,278	0,238	0,053	1,394	0,273
LnH.AADT	1,19	2,722	0,437	0,665	-4,336	6,716	0,005	0,437	0,071
[SurfType=1]	-1,877	5,024	-0,374	0,711	-12,076	8,322	0,004	0,374	0,065
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.
[PaveType=1]	-2,3	4,175	-0,551	0,585	-10,777	6,176	0,009	0,551	0,084
[PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=1] * [PaveType=1]	1,959	5,249	0,373	0,711	-8,697	12,615	0,004	0,373	0,065
[SurfType=1] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

PaveType, *SurfType* eta Ln*H.AADT* aldagaiak ez dira esanguratsuak. Saiakera batzuen ostean, aldagai hauek Erregresio Lineal Orokor Anizkoitzeko modelo batean hainbat eratan konbinatuz, lortutako R^2 onena ez da 0,35 baino altuagoa eta aldagai guztiak ez dira estatistikoki esanguratsuak. Adibidez, 9.4 ekuazioaren modeloa erakusten da (9.35 taula eta 9.36 taula).

9.35 taula. 9.4 ekuazioaren modeloaren Subjektu-arteko Efektuen testa 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Jatorria	Karratuen batuketa III mota	Askatasun-graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	743,708	3	247,903	6,424	0,001	0,336	19,271	0,952
Konstantea	2115,631	1	2115,631	54,821	< 0,001	0,591	54,821	1,000
LnAADT	205,648	1	205,648	5,329	0,027	0,123	5,329	0,614
PSVreq	96,711	1	96,711	2,506	0,122	0,062	2,506	0,339
SurfType	51,129	1	51,129	1,325	0,257	0,034	1,325	0,202
Errorea	1466,479	38	38,592					
Totala	111628,3	42						
Total zuzendua	2210,188	41						

9.36 taula. 9.4 ekuazioaren modeloaren parametroen estimazioak 2 urte edo gehiagoko adina errealeko Trazatu Berria tarteetan

Parametroak	B	Errore estandarra	t	Esang	%95eko konfiantza-tartea		Eta karrat partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Goiko			
Konstantea	108,271	15,286	7,083	< 0,001	77,325	139,216	0,569	7,083	1,000
LnAADT	-3,682	1,595	-2,308	0,027	-6,91	-0,453	0,123	2,308	0,614
PSVreq	-0,514	0,325	-1,583	0,122	-1,171	0,143	0,062	1,583	0,339
[SurfType=1]	-2,949	2,562	-1,151	0,257	-8,137	2,238	0,034	1,151	0,202
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

Hainbat konbinazioren azterketaren ondoren, ondorioztatu ahal da Ln*AADT* ia beti aldagai esanguratsua dela, PSV_{req} %90eko konfiantza-mailarekin benetako aldagaia dela (bere koefizientearen balioa ez da zero) eta *SurfType* sartu ahal da bere esangura baxua gora behera. (p-balioa = 0,257).

9.3.1.2. Mantentzea eta Errehabilitatzio (M&R) tarteen datuen analisia

Mantentzea eta Errehabilitatzio (M&R) tarteak analizatzeko 63 tarte daude. Horietako 3 ez dute 2 urte eta beraz, ez direla heldu leuntze-portaeraren fase egonkorrera kontsideratzen da. Hori dela eta, analisitik baztertzen dira, Kokkalis (1998)-en irizpidea eta Navarro *et al*. (2011)-en emaitzak jarraituz, aurreko atalean Trazatu Berriak tarteetarako erabaki den bezala Antzeko esplorazio-analisia gauzatu da 2 urte edo gehiagoko adina errealeko 60 Mantentzea eta Errehabilitatzio tarteetan (9.37 taula). Normaltasuna testak (9.38 taula) MSSC aldagaia banaketa normala duen aldagai bakarra dela erakusten du.

9.37 taula. Aldagai kuantitatiboen esplorazio-analisia Mantentzea eta Errehabilitatzio tarteetan

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Batez bestekoa		54,80	6978,65	277,28	5,83	5,85	13413,0	589,0	47,70	21,36	19,52
Errore estandarra		1,08	637,04	26,73	0,37	0,37	1659,5	84,6	0,49	2,00	0,71
%95eko KT-ak	**Beheko**	52,64	5703,9	223,80	5,09	5,10	10092,48	419,63	46,72	17,36	18,11
	Goiko	56,96	8253,4	330,76	6,58	6,59	16733,59	758,35	48,68	25,35	20,93
Bariantza		69,71	24349274	42857	8,24	8,30	165227148	429814	14,25	239,15	29,81
Desbideratze tip.		8,35	4934,50	207,02	2,87	2,88	12854	656	3,78	15,46	5,46
Minimoa		34,27	665	23	2	2	1818	14	40	2,1	12
Maximoa		73,32	19050	956	12	12,1	70547	3647	50	45,8	37
Anplitudea		39,05	18385	933	10	10,1	68729	3633	10	43,7	25
Kuartileko heina		12,858	5281,0	273,0	5	4,97	13258	633,97	5	29,7	10
Simetria		-0,127	0,620	1,305	0,44	0,443	2,181	2,640	-1,261	-0,036	0,318
Kurtosia		-0,204	-0,193	2,301	-1,05	-1,06	6,506	8,845	-0,044	-1,717	0,252

9.38 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat Mantentzea eta Errehabilitazio tarteetan

Aldagaiak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Estatistikoa	**Askatasun-graduak**	**Esangura (p-balioa)**	**Estatistikoa**	**Askatasun-graduak**	**Esangura (p-balioa)**
MSSC	0,078	60	0,200*	0,984	60	0,61
AADT	0,24	60	< 0,001	0,84	60	< 0,001
H.AADT	0,126	60	0,019	0,93	60	0,002
Age	0,127	60	0,018	0,889	60	< 0,001
R.Age	0,429	60	< 0,001	0,622	60	< 0,001
TotVeh	0,238	60	< 0,001	0,897	60	< 0,001
TotH.Veh	0,195	60	< 0,001	0,906	60	< 0,001
PSV	0,117	60	0,04	0,918	60	0,001
Rain 15	0,184	60	< 0,001	0,79	60	< 0,001
TotBit	0,202	60	< 0,001	0,722	60	< 0,001

[a] Lilliefors esangura zuzenketa

* Hau da benetako esanguraren beheko muga.

9.39 taulak erakusten ditu mendeko aldagai eta aldagai independenteen arteko korrelazioak, Pearson-en R korrelazio-koefizienteekin.

9.39 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) Mantentzea eta Errehabilitazio tarteetan

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
AADT	-0,443	<0,001
H.AADT	-0,436	<0,001
Age	0,023	0,862
R.Age	0,021	0,874
TotVeh	-0,436	< 0,001
TotH.Veh	-0,426	0,001
PSV	-0,500	<0,001
Rain 15	0,109	0,407
TotBit	-0,059	0,654

Aurreko analisian bezala, *AADT* and *H.AADT* korrelazio ona daukate MSSC-rekiko. Beste aldetik, *Rain15*, *Age* eta *R.Age* aldagaiak korrelazio baxuak erakusteaz gain, korrelazioa ez da esanguratsua. Aitzitik, *TotVeh* eta *TotH.Veh* korrelazio onak dauzkate mendeko aldagaiarekin. Aurreko analisian burutu zen bezala, *MSSC* aldagaia eta aldagai independente bakoitzaren arteko datuak hoberen doitzen dituzten kurbak kalkulatzen dira (9.40 taula) eta ekuazio horiek kontuan hartuta, eraldaketa batzuk proposatzen dira. *AADT*, *H.AADT*, *TotVeh* eta *TotH.Veh* aldagaietarako eraldaketa logaritmikoa gauzatu zen. PSV_{req}, *Age* eta *R.Age* aldagaietarako, erlazio lineal batetik beste batera aldatzeak ez du determinazio-koefizientean (R^2) 0,05-ko hobekuntzarik, era ondorioz, ez dira eraldatzen. Azkenik, *Rain15* ekuazio koadratiko baten bidez eraldatzen da.

9.40 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duten ekuazioak Mantentzea eta Errehabilitazio tarteetan

Aldagai independente	Ekuazio-mota	Modeloaren laburpena						Parametroen estimazioak		
		R^2	F	Askatasun -gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
AADT	Logaritmikoa	0,318	27,067	1	58	<0,001	93,268	-4,542	-	-
H.AADT	Logaritmikoa	0,273	21,824	1	58	<0,001	79,914	-4,745	-	-
Age	Lineala	0,001	0,03	1	58	0,862	54,411	0,067	-	-
R.Age	Lineala	0	0,025	1	58	0,874	54,445	0,061	-	-
TotVeh	Logaritmikoa	0,327	28,243	1	58	<0,001	131,715	-4,813	-	-
TotH.Veh	Logaritmikoa	0,291	23,857	1	58	<0,001	108,421	-4,197	-	-
PSV	Lineala	0,25	19,319	1	58	<0,001	107,541	-1,106	-	-
Rain15	Koadratikoa	0,107	3,404	2	57	0,040	49,549	0,863	0,019	-
TotBit	Lineal	0,003	0,203	1	58	0,654	56,561	-0,09	-	-

Trazatu Berria tarteetan burututako antzeko analisia egin zen aldagai independente kualitatiboekin Mantentzea eta Errehabilitazio tarteetarako (9.1 taula). Kasu honetan, kare-esneko errodadura geruzan duten tarteak daude, beraz *SurfType* aldagaian beste maila bat agertzen da. Faktore bakoitzean sortzen diren mailek mendeko aldagaiarekiko, banaketa normalak dituzte (9.42 taula).

9.41 taula. taula. Aldagai kualitatiboen esplorazio-analisia Mantentzea eta Errehabilitazio tarteetan, mailaz bilduta mendeko aldagaiarekiko (MSSC)

		PaveType		SurfType		
		Malgua (1)	**Erdi-zurruna (2)**	**AC (1)**	**BBTM& PA (2)**	**Kare-esnea (3)**
N		41	19	17	8	35
Batez bestekoa		56,057	52,086	56,547	47,094	55,712
Errore estandarra		1,363	1,587	2,834	1,155	1,044
%95eko Konfiantza-tarteak	**Beheko**	53,302	48,751	50,539	44,363	53,590
	Goiko	58,811	55,421	62,555	49,825	57,834
Bariantza		76,163	47,879	136,541	10,670	38,165
Desbideratze tipikoa		8,727	6,919	11,685	3,267	6,178
Minimoa		36,000	34,273	34,273	43,576	43,600
Maximoa		73,318	60,674	73,318	53,400	65,980
Anplitudea		37,318	26,401	39,046	9,824	22,380
Kuartileko heina		14,325	9,800	16,353	4,852	8,567
Simetria		-0,131	-0,953	-0,489	0,913	-0,410
Kurtosia		-0,595	0,855	-0,491	0,827	-0,699

9.42 taula. Normaltasun testak aldagai independente kualitatiboentzat Mantentzea eta Errehabilitazio tarteetan, mailaz bilduta mendeko aldagaiarekiko (MSSC)

Aldagai independente kualitatiboak	Aldagaiaren mailak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
		Estatistikoa	Askatasun-graduak	Esangura (p-balioa)	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)
PaveType	Malgua (1)	0,108	41	0,200*	0,975	41	0,505
	Erdi-zurruna (2)	0,171	19	0,147	0,916	19	0,096
SurfType	AC (1)	0,139	17	0,200*	0,95	17	0,459
	BBTM & PA (2)	0,164	8	0,200*	0,916	8	0,401
	Kare-esnea (3)	0,124	35	0,194	0,956	35	0,168

[a] Lilliefors esangura zuzenketa

* Hau da benetako esanguraren beheko muga.

Levene estatistikoak *PaveType* aldagaiaren mailekiko erakusten du bariantza homogeneoa daukatela eta ez dagoela batez bestekoen arteko diferentzia esanguratsurik (9.43 taula).

9.43 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako PaveType-ren mailetarako MSSC-rekiko Mantentzea eta Errehabilitazio tarteetan

Levene estatistikoa			t testa batez bestekoen berdintasunerako						
F	Esang..		t	Askatasun-graduak	Esangura (bi aldekoa)	Batez besteko diferentzia	Errore estandarra	%95eko konfiantza-tarteak diferentziarako	
								Beheko	Goiko
2,724	0,104	Bariantza berdinak suposatuta	1,743	58	0,087	3,970	2,278	-0,590	8,531
		Bariantza berdinak ez suposatuta	1,898	43,647	0,064	3,970	2,092	-0,247	8,188

Aitzitik, *SurfType*, aldagairako, hiru maila daudenez (AC, BBTM&PA, eta Kare-esnea) bariantzaren analisi bat (ANOVA) gauzatu zen. Levene estatistikoa (9.44 taula) hipotesi nulua errefusatzen dela ($p < 0,05$) eta beraz, mailek ez daukate bariantza homogeneorik. Batez bestekoen berdintasunaren test sendoek erakusten dute mailen artean ez dagoela batez bestekoaren berdintasunik (9.45 taula), eta ondorio berdina ateratzen da ANOVA testetik (9.46 taula). Gainera, *SurfType*-ren mailetarako mendeko aldagaiarekiko estatistiko batzuen bitartez burututako konparazio anizkoitzak erakusten du batez bestekoen diferentzia dagoela BBTM&PA beste bi mailen artean (AC eta Kare-esnea), baina ez dagoela AC eta Kare-esnearen artean (9.47 taula). Kasu honetan, Tamhane’s T2 and Dunnett’s T3 estatistikoak erabili dira mailek bariantza homogeneorik ez zutelako.

9.44 taula. Bariantzaren homogeneotasunaren testa SurfType-ren mailetarako MSSC-rekiko Mantentzea eta Errehabilitazio tarteetan

Levene estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
8,833	2	57	< 0,001

9.45 taula. Batez bestekoen berdintasunaren test sendoak SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) Mantentzea eta Errehabilitazio tarteetan

Testa	Estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
Welch	16,160	2	24,371	< 0,001
Brown-Forsythe	4,520	2	24,477	0,021

9.46 taula. Bariantzaren analisia (ANOVA) SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) Mantentzea eta Errehabilitazio tarteetan

Aldagaia (SurfType)	Karratuen batuketa	Askatasun-graduak	Batez besteko karratua	F	Esangura
Mailen artean	556,014	2	278,007	4,455	0,016
Mailaren barruan	3556,976	57	62,403		
Guztira	4112,989	59			

9.47 taula. Konparazio anizkoitza SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) Mantentzea eta Errehabilitazio tarteetan

Estatistikoa	SurfType (I)	SurfType (J)	Batez besteko diferentzia (I-J)	Errore estad.	Esang.	95%95eko Konfiantza-tarteak	
						Beheko	Goikoa
Tamhane's T2	AC (1)	BBTM&PA (2)	9,453	3,060	0,017	1,498	17,408
		Kare-esnea (3)	0,835	3,020	0,990	-7,016	8,686
	BBTM&PA (2)	AC (1)	-9,453	3,060	0,017	-17,408	-1,498
		Kare-esnea (3)	-8,618	1,557	0,000	-12,667	-4,568
	Slurry (3)	AC (1)	-0,835	3,020	0,990	-8,686	7,016
		BBTM&PA (2)	8,618	1,557	0,000	4,568	12,667
Dunnett's T3	AC (1)	BBTM&PA (2)	9,453	3,060	0,017	1,529	17,376
		Kare-esnea (3)	0,835	3,020	0,989	-6,985	8,655
	BBTM&PA (2)	AC (1)	-9,453	3,060	0,017	-17,376	-1,529
		Slurry (3)	-8,618	1,557	0,000	-12,651	-4,584
	Slurry (3)	AC (1)	-0,835	3,020	0,989	-8,655	6,985
		BBTM&PA (2)	8,618	1,557	0,000	4,584	12,651

Modeloak garatzeko, erregresio lineal anizkoitza gauzatu zen mendeko aldagaiarekin hoberen erlazionatzen diren mendeko aldagaiak ikusteko. Aldagai independenteak eraldatu ziren aurreko emaitzen arabera (9.40

taula). IBM SPSS v.24 programaren Pausoz Pauso Hautaketa eta Aurrera pausoz pauso funtzioak erabiliz, modeloan sartzen zen aldagai bakarra Ln*AADT* zen (9.48 taula).

9.48 taula. Proposatutako erregresio lineal anizkoitzaren modeloaren bariantzaren analisia (ANOVA) Mantentzea eta Errehabilitazio tarteetan

Bariantzaren analisia						
Iturria	**Askatasun-graduak**	**Karratuen batuketa**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**
Modeloa	1	1308,686	1308,686	27,067	< 0,001	1,037
Errorea	58	2804,303	48,35			
Total zuzenduta	59	4112,989				
Estimazioaren errore estandarra	**Aldakuntza-koefizientea**	**R**	**R^2**	**R^2_{adj}**	**Kolinealtasunaren estatistikoak**	
					Tolerantzia	**VIF**
6,953		0,564	0,318	0,306	1,000	1,000
Parametroen estimazioak						
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioa**	**p-balioa**	**%95eko konfiantza-tarteak**	
Konstantea	93,268	7,448	12,522	< 0,001	78,358	108,178
LnAADT	-4,542	0,873	-5,203	< 0,001	-6,290	-2,795

Orduan, Erregresio Lineal Orokor Anizkoitzeko modelo batzuk probatu ziren, aldagai kualitatiboak erabiliz. Berriro ere, 9.3 ekuazioan erakutsitako aldagaiak erabiliz analizatu zen (9.49 taula eta 9.50 taula).

9.49 taula. 9.3 ekuazioaren modeloaren Subjektu-arteko Efektuen testa Mantentzea eta Errehabilitazio tarteetan

Jatorria	**Karratuen batuketa III mota**	**Askatasun-graduak**	**Batez besteko karratua**	**F**	**Esang.**	**Eta karratu partziala**	**Ez zentralitatearen parametroa**	**Ikusitako botera**
Modelo zuzendua	2267,057	7	323,865	9,123	< 0,001	0,551	63,863	1,000
Konstantea	2063,186	1	2063,186	58,12	< 0,001	0,528	58,12	1,00
LnAADT	91,143	1	91,143	2,567	0,115	0,047	2,567	0,35
LnH.AADT	12,683	1	12,683	0,357	0,553	0,007	0,357	0,09
PSVreq	63,133	1	63,133	1,778	0,188	0,033	1,778	0,258
SurfType	939,544	2	469,772	13,234	< 0,001	0,337	26,467	0,996
PaveType	171,325	1	171,325	4,826	0,033	0,085	4,826	0,578
SurfType * PaveType	58,89	1	58,89	1,659	0,203	0,031	1,659	0,244
Errorea	1845,932	52	35,499					
Total	184291,279	60						
Total zuzendua	4112,989	59						

9.50 taula. 9.3 ekuazioaren modeloaren parametroen estimazioak Mantentzea eta Errehabilitazio tarteetan

Parametroak	**B**	**Errore estandarra**	**t**	**Esang**	**%95 konfiantza-tartea**		**Eta karratu partziala**	**Ez zentralitatearen parametroa**	**Ikusitako botera**
					Beheko	**Goiko**			
Konstantea	121,270	15,42	7,861	<0,001	90,314	152,226	0,543	7,861	1,000
LnAADT	-4,281	2,672	-1,602	0,115	-9,643	1,080	0,047	1,602	0,350
PSVreq	1,338	2,238	,598	0,553	-3,153	5,828	0,007	0,598	0,090
LnH.AADT	-0,746	0,559	-1,334	0,188	-1,867	0,376	0,033	1,334	0,258
[SurfType=1]	-13,403	3,815	-3,513	0,001	-21,058	-5,747	0,192	3,513	0,932
[SurfType=2]	-8,963	2,626	-3,413	0,001	-14,233	-3,693	0,183	3,413	0,918
[SurfType=3]	0[a]	.	.	.	.	.	.	.	.
[PaveType=1]	2,816	2,445	1,152	0,255	-2,090	7,721	0,025	1,152	0,204
[PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=1] * [PaveType=1]	5,732	4,450	1,288	0,203	-3,198	14,662	0,031	1,288	0,244
[SurfType=1] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=3] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=3] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

9.49 taulan ikusten den bezala, LnH.AADT da esangura baxuagoa daukan aldagaia. Modelotik baztertuta, PSV_{req} aldagaia da esangura txikiagoko aldagaia (p-balioa = 0,204).

Analisitik baztertuta, Erregresio Lineal Orokor Anizkoitzeko modelo bat analizatu zen Ln*AADT* aldagai kuantitatiboa, eta *SurfType* eta *PaveType* aldagai kualitatiboak erabiliz, *PaveType* eta *PaveType***SurfType* faktoreen konbinaketa ez ziren esanguratsuak (p-balioa = 0,070 eta p-balioa = 0,160, hurrenez hurren). Analisitik *PaveType* kenduta, 9.5 ekuazioko aldagaiak dituen modeloa frogatu zen. Modeloaren aldagai guztiak estatistikoki esanguratsua ziren (9.51 eta 9.52 taulak) eta modeloaren determinazio-koefizientea (R^2) 0,503 zen eta doitutako R^2 0,476 zen.

$$MSSC = Intercept + LnAADT + SurfType \tag{9.5}$$

9.51 taula. 9.5 ekuazioaren modeloaren Subjektu-arteko Efektuen testa Mantentzea eta Errehabilitazio tarteetan

Jatorria	Karratuen batuketa III mota	Askatasun -graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	2069,161	3	689,72	18,898	<0,001	0,503	56,694	1,000
Konstantea	5807,797	1	5807,797	159,131	<0,001	0,74	159,131	1,000
LnAADT	1513,148	1	1513,148	41,46	<0,001	0,425	41,46	1,000
SurfType	760,475	2	380,237	10,418	<0,001	0,271	20,837	0,984
Errorea	2043,828	56	36,497					
Totala	184291,279	60						
Total zuzendua	4112,989	59						

9.52 taula. 9.5 ekuazioaren modeloaren parametroen estimazioak Mantentzea eta Errehabilitazio tarteetan

Parametroak	B	Errore estandarra	t	Esang	%95eko konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Goiko			
Intercept	113,185	8,984	12,598	<0,001	95,187	131,182	0,739	12,598	1
LnAADT	-6,488	1,008	-6,439	<0,001	-8,507	-4,47	0,425	6,439	1
[SurfType=1]	-8,644	2,315	-3,735	<0,001	-13,281	-4,008	0,199	3,735	0,956
[SurfType=2]	-7,409	2,375	-3,12	0,003	-12,166	-2,651	0,148	3,12	0,866
[SurfType=3]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

9.3.1.3. Bide-zoruaren egitura ezaguneko Trazatu Berriak eta Mantentzea eta Errehabilitazioa tarteen analisiaren ondorioak.

Trazatu Berria eta Mantentze eta Errehabilitazioa taldeen arabera bide-zoruaren egitura osoa ezaguneko tarteetarako modelo batzuk garatu ondoren, labainketarekiko erresistentzian, *MSSC*-ren bidez neurtuta, eragina handiagoa daukaten faktoreak honako hauek dira: Eguneko Batez Besteko Intentsitatea (*ADDT*) (batez ere, bere logaritmo naturala kalkulatzekotan), ibilgailu astunen Eguneko Batez Besteko Trafikoa (*H.AADT*) eta agregakinen eskatutako Azeleratutako Leuntze Koefizientea (PSV_{req}). Azken biak literaturan faktore nagusiak bezala identifikatu ziren. Frogatu da AADT aldagai nagusi bihurtzen dela eta besteak ez direla esanguratsuak berarekin jartzean. Beste aldagai kuantitatiboek, adina (*Age)*, adina erreala(*R.Age*), geruza bituminosoen lodiera totala (*TotBit*), eta datu-biltzea baino aurreko 15 egunetan neurtutako euriaren datua (*Rain15*) beti korrelazio baxuak dituzte mendeko aldagaiarekiko eta ez dira modeloetan sartzen. Sekzioa zeharkatu duten ibilgailu totalen kopuru metatuak (*TotVeh*) eta sekzioa zeharkatu duten ibilgailu astunen kopuru metatuak (*TotH.Veh*) nahiko korrelazio ona erakusten dute (0,38 < R < 0,20).

Aldagai kualitatiboei dagokienez, nahiz eta batzuetan, kasuen arabera (Trazatu Berria edo Mantentzea eta Errehabilitazioa tarteak), *PaveTyp*e-ren mailen bariantza homogeneoa zen edo ez, ez zegoen batez bestekoen arteko diferentziarik, marruskaduran eragina ez duela adieraziz. Gainera, GLM batean sartzean, ez zen estatistikoki esanguratsua, eta ondorioz, beti baztertzen zen. Aitzitik, *SurfType* aldagaiaren mailek bariantza ez homogeneoa eta batez bestekoen diferentziarik eza erakutsi zituen Trazatu Berria tartetan baina Mantentzea eta Errehabilitazioa tarteetan batez bestekoen diferentzia zegoen. Horrez gain, GLM-tan sartzean esangura handia zuela ikusi zen eta modeloaren iragarpena hobetuz.

Eragina daukaten aldagaiak ia berdinak direnez kasu guztietan, eta kasu bietan daude datu kopuru txikia ekiditeko, analisi orokor bat gauzatzen da bide-zoruaren egitura osoa ezaguneko tarteekin, haien sailkapena kontuan hartu gabe. Hala eta guztiz ere, “Trazatu Berria” eta “Mantentzea eta Errehabilitazioa (M&R)” mailak ezartzen dira aldagai independente kualitatibo batean, gauzatutako Lan mota (*WorkType*), bere eragina aztertu ahal izateko.

9.3.1.4. Bide-zoruaren sekzio osoa ezaguneko tarteen analisi globala

Bide-zoruaren sekzio osoa ezaguneko tarteen analisi globala datu guztiekin batera burutzen da. 2016. urteko 112 datuetatik, bi urte baino gutxiagoko adina zeukatenak baztertu ziren, lehenago aipatutako ideiak jarraituz (Kokkalis, 1998, Navarro *et al.*, 2011). Horren ondorioz 102 datu eskuragarri daude (42 + 60).

Esplorazio-analisia 9.53 taulan aurkezten da. Aldagai kuantitatiboen normaltasunaren testak *MSSC* aldagaia dela banaketa normala duena bakarra adierazten du.

9.53 taula. Aldagai kuantitatiboen esplorazio-analisia bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Batez bestekoa		53,25	7104,27	340,91	7,63	7,56	16940,6	949,4	47,29	25,43	18,45
Errore estandarra		0,80	537,24	30,97	0,52	0,52	1648,3	112,0	0,40	1,53	0,56
%95eko KT-ak	**Beheko**	51,66	6038,54	279,47	6,60	13670,9	13670,9	727,3	46,51	22,39	17,34
	Goiko	54,85	8170,01	402,35	8,66	20210,4	20210,4	1171,4	48,08	28,48	19,56
Bariantza		66,06	29439479	97854	27,62	27,13	277114620	1278385	16,03	240,12	31,89
Desbideratze tip.		8,13	5425,8	312,8	5,26	5,21	16647	1131	4,00	15,50	5,65
Minimoa		34,27	665	23	2	2	1760	14	40	2,1	8
Maximoa		73,32	29916	1361	29	28,6	98892	4447	50	45,8	37
Anplitudea		39,05	29251	1338	27	26,6	97132	4433	10	43,7	29
Kuartileko heina		13,65	6694	345	5	5,18	16898	1018	5	33,7	9
Simetria		0,011	1,239	1,288	1,873	1,874	2,318	1,691	-1,004	-0,503	0,395
Kurtosia		-0,384	2,263	0,835	4,146	4,105	7,136	2,063	-0,705	-1,506	0,056

9.54 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagaiak	**Kolmogorov-Smirnov[a]**			**Shapiro-Wilk**		
	Estatistikoa	**Askatasun-graduak**	**Esangura (p-balioa)**	**Estatistikoa**	**Askatasun-graduak**	**Esangura (p-balioa)**
MSSC	0,069	102	0,200*	0,985	102	0,286
AADT	0,258	102	<0,001	0,803	102	<0,001
H.AADT	0,118	102	0,001	0,905	102	<0,001
Age	0,197	102	<0,001	0,833	102	<0,001
R.Age	0,407	102	<0,001	0,655	102	<0,001
TotVeh	0,207	102	<0,001	0,811	102	<0,001
TotH.Veh	0,198	102	<0,001	0,814	102	<0,001
PSV	0,127	102	<0,001	0,949	102	0,001
Rain 15	0,181	102	<0,001	0,776	102	<0,001
TotBit	0,237	102	<0,001	0,749	102	<0,001

[a] Lilliefors Esangura zuzenketa

* Hau da beheko muga benetako esangurarako

Mendeko aldagai eta aldagai independenteen arteko korrelazioak 9.55 taulan erakusten dira eta mendeko aldagaia eta aldagai independenteen arteko erlazioak hoberen doitzen duten kurbak 9.56 taulan azaltzen dira.

9.55 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	**Korrelazioa MSSC-ekin (R)**	**Korrelazioren esangura (bi aldekoa)**
AADT	-0,452	<0,001
H.AADT	-0,382	<0,001
Age	-0,015	0,88
R.Age	-0,013	0,893
TotVeh	-0,381	<0,001
TotH.Veh	-0,356	<0,001
PSV	-0,448	<0,001
Rain 15	-0,052	0,606
TotBit	-0,115	0,251

Aldagai kualitatiboen mailak (*PaveType*, *SurfType* and *WorkType*) mendeko aldagaiarekiko ere aztertzen dira (9.57)

9.56 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duten ekuazioak bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Ekuazio-mota	Modeloaren laburpena						Parametroen estimazioak		
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
AADT	Logaritmikoa	0,300	42,767	1	100	<0,001	92	-4,558	-	-
H.AADT	Logaritmikoa	0,210	26,554	1	100	<0,001	72,557	-3,592	-	-
Age	Lineala	< 0,001	0,023	1	100	0,880	53,431	-0,023	-	-
R.Age	Lineala	< 0,001	0,018	1	100	0,893	53,411	-0,021	-	-
TotVeh	Logaritmikoa	0,274	37,787	1	100	<0,001	122,137	-4,251	-	-
TotH.Veh	Logaritmikoa	0,212	26,865	1	100	<0,001	92,738	-3,02	-	-
PSV	Lineala	0,201	25,088	1	100	<0,001	96,247	-,909	-	-
Rain 15	Lineala	0,003	0,268	1	100	0,606	53,942	-,027	-	-
TotBit	Lineala	0,013	1,333	1	100	0,251	56,298	-,165	-	-

Levene estatistikoak erakusten du *PaveType*-ren mailek ez daukat bariantza homogeneoa eta *t* testak azaltzen du ez dagoela batez bestekoen arteko diferentziarik (9.58 taula)

Levene estatistikoa *WorkType*-erako adierazten du hipotesi nulua onartzen dela (p-balioa > 0,05) eta bariantzak homogeneoak dira eta *t* testak adierazten du badagoela batez bestekoen arteko diferentzia %95eko konfiantza-maila (9.59 taula).

9.57 taula. Aldagai kualitatiboen esplorazio-analisia mailaz bilduta mendeko aldagaiarekiko (MSSC) bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		PaveType		SurfType			WorkType	
		Malgua	Erdi-zurruna	AC	BBTM&PA	Slurry	Trazatu Berria	M&R
N		58	44	42	25	35	42	60
Batez bestekoa		54,45	51,67	54,15	48,31	55,71	51,04	54,80
Errore estandarra		1,19	0,98	1,55	0,80	1,04	1,13	1,08
%95eko Konfiantza-tarteak	**Beheko**	52,08	49,68	51,02	46,65	53,59	48,75	52,64
	Goiko	56,83	53,65	57,27	49,96	57,83	53,33	56,96
Bariantza		81,57	42,53	100,77	16,09	38,17	53,91	69,71
Desbideratze tipikoa		9,03	6,52	10,04	4,01	6,18	7,34	8,35
Minimoa		36,00	34,27	34,27	40,83	43,60	36,20	34,27
Maximoa		73,32	63,50	73,32	55,10	65,98	64,63	73,32
Anplitudea		37,32	29,23	39,05	14,27	22,38	28,43	39,05
Kuartileko heina		15,34	8,92	16,82	7,00	8,57	10,33	12,86
Simetria		-0,07	-0,41	-0,23	0,21	-0,41	0,04	-0,13
Kurtosia		-0,82	0,30	-0,63	-1,09	-0,70	-0,66	-0,20

9.58 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako PaveType-ren mailetarako MSSC-rekiko bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa		**t testa batez bestekoen berdintasunerako**							
F	**Esang..**	**Batez besteko diferentzia**	**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Beheko**
8,875	0,004	Bariantza berdinak suposatuta	1,732	100	0,086	2,786	1,609	-0,406	5,979
		Bariantza berdinak ez suposatuta	1,809	99,791	0,073	2,786	1,540	-0,270	5,842

9.59 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako WorkType-ren mailetarako MSSC-rekiko bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa		**t testa batez bestekoen berdintasunerako**							
F	**Esang..**	**Batez besteko diferentzia**	**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Goiko**
0,717	0,399	Bariantza berdinak suposatuta	-2,349	100	0,021	-3,758	1,600	-6,932	-0,584
		Bariantza berdinak ez suposatuta	-2,403	94,827	0,018	-3,758	1,564	-6,863	-0,654

Levene estatistikoa *SurfType*-ren mailetarako erakusten du hipotesi nulua errefusatzen dela (p-balioa < 0,05) (9.60 taula) eta bariantzak ez direla homogeneoak. Batez bestekoen berdintasunaren test sendoek (9.61 taula) eta ANOVAk (9.62 taula) adierazten dute ez dagoela batez bestekoen berdintasuna (diferentzia dagoela). Horrez gain, konparazio anizkoitza *SurfType*-ren mailetarako, Tamhane's T2 eta Dunnett's T3 estatistikoen bitartez, erakusten du diferentzia agertzen dela AC-ren eta BBTM&PA-ren artean eta BBTM&PA-ren eta Kare-esnearen artean, baina ez AC eta Kare-esnearen artean (9.63 taula).

9.60 taula. Bariantzaren homogeneotasunaren testa SurfType-ren mailetarako MSSC-rekiko bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa	**Askatasun-graduak 1**	**Askatasun graduak 2**	**Esangura**
10,982	2	99	< 0,001

9.61 taula. Batez bestekoen berdintasunaren test sendoak SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Testa	Estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
Welch	17,342	2	65,934	< 0,001
Brown-Forsythe	8,881	2	84,391	< 0,001

9.62 taula. Bariantzaren analisia (ANOVA) SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagaia (SurfType)	Karratuen batuketa	Askatasun-graduak	Batez besteko karratua	F	Esangura
Mailen artean	856,900	2	428,450	7,294	0,001
Mailaren barruan	5815,240	99	58,740		
Guztira	6672,140	101			

Aldagai kuantitatiboekin (eta haien transformazioekin) erregresio lineal anizkoitzeko modeloa saiatu zen MSSC-rekin hoberen erlazionatzen direnak ikusteko. IMB SPSS v24 programaren Aurrera Pausoz Pauso eta Pausoz Pauso Hautaketa funtzioak erabili ziren eta modeloan sartzen zen aldagai bakarra Ln*AADT* zen, esangura globalarekin eta esangura indibidualarekin %95eko konfiantza-mailaz (9.64 taula).

9.63 taula. Konparazio anizkoitza SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Estatistikoa	SurfType (I)	SurfType (J)	Batez besteko diferentzia (I-J)	Errore estad.	Esang.	%95eko Konfiantza-tarteak	
						Beheko	**Goikoa**
HSD Tukey	AC	BBTM&PA	5,840	1,744	0,004	1,553	10,127
		Kare-esnea	-1,566	1,868	0,789	-6,136	3,005
	BBTM&PA	AC	-5,840	1,744	0,004	-10,127	-1,553
		Kare-esnea	-7,406	1,317	<0,001	-10,644	-4,167
	Kare-esnea	AC	1,566	1,868	0,789	-3,005	6,136
		BBTM&PA	7,406	1,317	0,000	4,167	10,644
Bonferroni	AC	BBTM&PA	5,840	1,744	0,004	1,557	10,123
		Kare-esnea	-1,566	1,868	0,786	-6,132	3,001
	BBTM&PA	AC	-5,840	1,744	0,004	-10,123	-1,557
		Kare-esnea	-7,406	1,317	<0,001	-10,641	-4,171
	Kare-esnea	AC	1,566	1,868	0,786	-3,001	6,132
		BBTM&PA	7,406	1,317	<0,001	4,171	10,641

9.64 taula. Proposatutako erregresio lineal anizkoitzaren modeloaren bariantzaren analisia (ANOVA) bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

			Bariantzaren analisia			
Iturria	**Karratuen batuketa**	**Askatasun-graduak**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**
Modeloa	1998,686	1	1998,686	42,767	<0,001	1,265
Errorea	4673,453	100	46,735			
Total zuzenduta	6672,140	101				
Estimazioaren errore estandarra	**Aldakuntza-koefizientea**	**R**	**R^2**	**R^2_{adj}**	**Kolinealtasunaren estatistikoak**	
					Tolerantzia	**Tolerantzia**
6,836		0,547	0,300	0,293	1,000	1,000
			Parametroen estimazioak			
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioa**	**p-balioa**	**%95eko konfiantza-tarteak**	
Konstantea	92,000	5,964	15,427	<0,001	80,168	103,831
LnAADT	-4,558	,697	-6,540	<0,001	-5,940	-3,175

Erregresio Lineal Orokor Anizkoitzeko (GLM) modeloak aztertu ziren. Lehenengo eta behi,. 9.6 ekuazioaren itxura duen modeloa saiatu zen aldagai kualitatiboen esangura aztertzeko. Modeloaren determinazio-koefizientea (R^2) 0,508 zen eta doitutako R^2-a 0,448. (9.65 taula).

$$MSSC = Intercept + LnAADT + LnH.AADT + PSV_{req} + SurfType + PaveType +$$
$$+ WorkType + SurfType * WorkType + SurfType * PaveType + WorkType * PaveType +$$
$$+ SurfType * WorkType * PaveType \qquad [9.6]$$

Ikusten den bezala, *WorkType* eta *PaveType* aldagaiek (eta haien konbinazioek) esangura baxua daukate. *WorkType* aldagaiak baztertu zen, marruskadura eskuragarrian eragina duen faktore bat ez dela erakutsi ondoren. Beste GLM bat *WorkType* aldagairik gabe aztertu zen, 9.3 ekuazioaren modeloa erabiliz. Lortutako determinazio-koefizientea 0,489 zen (doitutako R^2 0,446) (9.66 taula).

PaveType eta Ln*H.AADT* aldagaiak oraindik ez dira esanguratsuak. *PaveType* baztertu da ez daukalako eraginik labainketarekiko erresistentzian. *PaveType* aldagairik gabeko GLM berri batek erakusten du Ln*H.AADT* aldagaiak ez duela esangurarik. Horren ondorioz, modelotik kentzen da. 9.7 ekuazioan azaldutako modeloa garatzen da (9.67 eta 9.68 taulak), lortutako determinazio-koefizientea (R^2) 0,466 izanik (doitutako R^2 = 0,444) Aldagai guztiak esanguratsuak dira %90eko konfiantza-mailaz.

$$MSSC = Intercept + LnAADT + PSV_{req} + SurfType \qquad [9.7]$$

9.65. taula. 9.6 ekuazioaren modeloaren Subjektu-arteko Efektuen testa bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Jatorria	**Karratuen batuketa III mota**	**Askatasun -graduak**	**Batez besteko karratua**	**F**	**Esang.**	**Eta karratu partziala**	**Ez zentralitatearen parametroa**	**Ikusitako botera**
Modelo zuzendua	3391,539	11	308,322	8,458	<0,001	0,508	93,043	1,000
Konstantea	4590,841	1	4590,841	125,94	<0,001	0,583	125,945	1,000
LnAADT	219,266	1	219,266	6,015	0,016	0,063	6,015	0,68
LnH.AADT	17,923	1	17,923	0,492	0,485	0,005	0,492	0,107
PSVreq	144,623	1	144,623	3,968	0,049	0,042	3,968	0,504
SurfType	890,11	2	445,055	12,21	<0,001	0,213	24,419	0,995
WorkType	29,284	1	29,284	0,803	0,372	0,009	0,803	0,144
PaveType	42,722	1	42,722	1,172	0,282	0,013	1,172	0,188
SurfType * WorkType	5,912	1	5,912	0,162	0,688	0,002	0,162	0,068
SurfType * PaveType	76,323	2	38,161	1,047	0,355	0,023	2,094	0,228
WorkType * PaveType	124,477	1	124,477	3,415	0,068	0,037	3,415	0,448
SurfType * WorkType * PaveType	0	0	.	.	.	0	0	.
Errorea	3280,601	90	36,451					
Total	295919,58	102						
Total zuzendua	6672,14	101						

9.66. taula. 9.3 ekuazioaren modeloaren Subjektu-arteko Efektuen testa bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Jatorria	**Karratuen batuketa III mota**	**Askatasun -graduak**	**Batez besteko karratua**	**F**	**Esang.**	**Eta karratu partziala**	**Ez zentralitatearen parametroa**	**Ikusitako botera**
Modelo zuzendua	3265,985	8	408,248	11,14	<0,001	0,489	89,173	1,000
Konstantea	5593,539	1	5593,539	152,72	<0,001	0,622	152,723	1,000
LnAADT	249,217	1	249,217	6,80	0,011	0,068	6,804	0,733
LnH.AADT	15,467	1	15,467	0,42	0,517	0,005	0,422	0,099
PSVreq	189,044	1	189,044	5,16	0,025	0,053	5,162	0,614
SurfType	1147,412	2	573,706	15,66	<0,001	0,252	31,328	0,999
PaveType	13,539	1	13,539	0,37	0,545	0,004	0,37	0,092
SurfType * PaveType	75,207	2	37,604	1,03	0,362	0,022	2,053	0,224
Errorea	3406,154	93	36,625					
Total	295919,58	102						
Total zuzendua	6672,14	101						

9.67 taula. 9.7 ekuazioaren modeloaren Subjektu-arteko Efektuen testa bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Jatorria	Karratuen batuketa III mota	Askatasun-graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	3112,197	4	778,049	21,2	<0,001	0,466	84,8	1
Konstantea	5560,814	1	5560,814	151,519	<0,001	0,61	151,519	1
LnAADT	750,149	1	750,149	20,44	<0,001	0,174	20,44	0,994
PSVreq	116,165	1	116,165	3,165	0,078	0,032	3,165	0,421
SurfType	1109,204	2	554,602	15,112	<0,001	0,238	30,223	0,999
Errorea	3559,942	97	36,7					
Total	295919,58	102						
Total zuzendua	6672,14	101						

9.68 taula. 9.7 ekuazioaren modeloaren parametroen estimazioak bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Parametroak	B	Errore estandarra	t	Esang	%95 konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Beheko			
Konstantea	119,797	9,647	12,418	<0,001	100,65	138,943	0,614	12,418	1
LnAADT	-4,661	1,031	-4,521	<0,001	-6,707	-2,615	0,174	4,521	0,994
PSVreq	-0,464	0,261	-1,779	0,078	-0,982	0,054	0,032	1,779	0,421
[SurfType=1]	-8,643	1,692	-5,109	<0,001	-12	-5,285	0,212	5,109	0,999
[SurfType=2]	-5,737	1,602	-3,581	0,001	-8,917	-2,558	0,117	3,581	0,944
[SurfType=3]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

Erregresio Lineal Orokor Anizkoitzeko modelo alternatibo bat aztertu zen, Ln*H.AADT*, PSV_{req} eta *SurfType* aldagaiak erabiliz, Szatkowski and Hosking (1972)-ek proposatutako ideiak jarraituz. Modelo horretan aldagai guztiak esanguratsuak ziren (9.69 taula), baina determinazio-koefiziente txikiago bat lortzen zen (R^2 = 0,387; doitutako R^2 = 0,362).

9.69 taula. LnH.AADT, PSVreq eta SurfType erabiltzen dituen Subjektu-arteko Efektuen testa bide-zoruaren egitura ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Jatorria	Karratuen batuketa III mota	Askatasun-graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	2581,54	4	645,385	15,304	<0,001	0,387	61,216	1
Konstantea	5144,756	1	5144,756	121,997	<0,001	0,557	121,997	1
LnH.AADT	219,491	1	219,491	5,205	0,025	0,051	5,205	0,617
PSVreq	571,996	1	571,996	13,564	<0,001	0,123	13,564	0,954
SurfType	996,078	2	498,039	11,81	<0,001	0,196	23,62	0,993
Errorea	4090,6	97	42,171					
Total	295919,58	102						
Total zuzendua	6672,14	101						

9.3.1.5. Bide-zoruaren egitura osoa ezaguneko tarteen analisitik jasotako ondorioak

Bide-zoruaren egitura osoa ezaguneko tarteen analisi globala burutu eta gero ondorioztatu ahal da bide-zoru osoa ezagutzea ez dela beharrezkoa. Frogatu da *PaveType*-k, bide-zoru malguak eta bide-zoru erdi-zurruna berezitzen dituen aldagaiak, ez daukala eraginik marruskaduraren iragarpenean. Horrez gain, *TotBit* aldagaia, geruza bituminosoen lodiera osoa adierazten duen aldagaia, korrelazio txikiak erakutsi ditu mendeko aldagaiarekiko (*MSSC*) eta ez da modeloetan sartu %95eko konfiantza-mailaz. Gainera, *WorkType* aldagaiaren mailen analisiak (Trazatu Berria eta Mantentzea eta Errehabilitazioa) erakutsi du ez dela eraginik duen faktorea, eta datuak batera aztertu zirenean, esangura baxuko aldagai kualitatiboa dela adierazi du.

Antzeko ondorioak lortu ziren marruskadura iragartzeko gauzatutako aurreko lanetan (Pérez-Acebo *et al.*, 2017a). Hala eta guztiz ere, frogatu nahi zen labainketarekiko erresistentziaren modeloetarako ez zela beharrezkoa bide-zoruaren egitura osoa ezagutzea. Ideia hau oso ezaguna da literaturan. Aldi askotan frogatu da bide-zoruen egiturazko ezaugarriak ez dutela eraginik edo ez daukatela erlaziorik gainazaleko ezaugarriekin (Pérez-Acebo *et al.*, 2018b, Flora, 2009; Prang *et al.*, 2012).

Beste aldagaiak, (*AADT*, *H.AADT*, PSV_{req}, *TotVeh*, *TotH.Veh*, *Age*, *R.Age* and *SurfType*, jakin ahal dira errodadura geruzaren datuak eskuragarri badira. Horren ondorioz, Bizkaiko Foru Aldundiko Bide-zoruak Kudeatzeko Sistemaren Errodadura Geruza artxiboa erabili ahal da errodadura geruzaren ezaugarriak baino ez dira behar. Trafiko datuak ere eskuragarri dira. Hori dela eta, Bizkaiko Foru Aldundiak kudeatzen dituen errepide-sareko errepide guztien luzera osoan aztertu ahal dira errodadura geruza iragartzeko. Analisia 9.4 atalean gauzatzen da.

9.4. Labainketarekiko erresistentzia iragartzeko modeloak errodadura geruza ezaguneko tarteetan

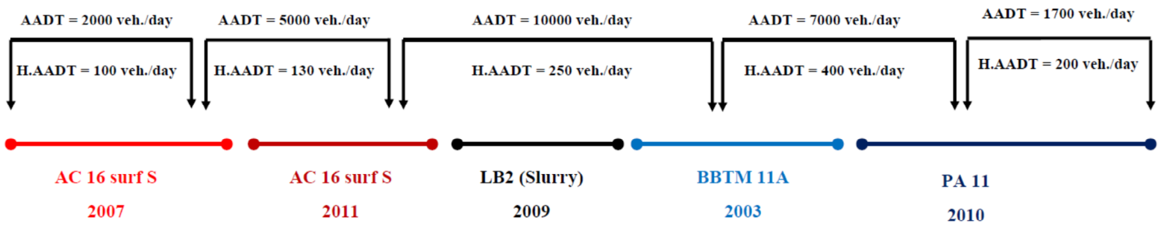
Aurreko atalean ondorioztatu zenez, labainketarekiko erresistentzia modelizatzeko, errodadura geruzaren eta trafikoaren datuak baino ez dira behar. Beraz, bide-zoruaren sekzio osoa ezaguneko tarteez gain, errepide bakoitzeko luzeraren beste tarteak ere modeloaren garapenean ere sartu ahal dira. Helburu honekin, errepide guztien Errodadura Geruza artxiboak lortzen dira BKStik eta luzera osoa tarteetan zatitzen da errodadura geruzaren arabera eta noiz gauzatu zen, hau da, tartean errodadura geruza eraldatzen duen azken proiektuaren arabera. Beraz, zatiketa errepidean gauzatutako proiektuen arabera gauzatzen da. Elkarren alboan dauden bi tarteetan errodadurako material bera zabaldu ere, momentu ezberdinetan burutu baziren, errodadura ezberdinak bezala adierazten dira.

Errepide bakoitzaren zatiketaren detaileko analisia II. eranskineko Errepide artxiboetan erakusten da. Artxibo horien azken atalean, "*Skid resistance of surface layers in 2016*" titulupean errepidearen zatiketa osoa erakusten da. Adierazten dira: hasierako eta amaierako Puntu Kilometrikoak, jarduera horren proiektua, lanen data eta errodadura geruzaren materiala eta bere lodiera, 9.70 taulan erakusten den bezala.

9.70 taula. II. eranskinaren Errepidea Artxiboaren "Skid resistance of surface layers in 2016" atalaren adibidea BI-631 errepideko datuekin (Eremu 1).

Skid resistance of surface layers in 2016
19+1040 – 19+1480: PROJ-1334 (01/07/2011) LB2. 20+0000 – 21+0600: PROJ-1335 (01/07/2015) LB2. 21+0600 – 22+0190: PROJ-1337 (01/07/2011) LB2. 23+0000 – 23+0320: PROJ-280 (01/06/2012) AC 16 surf S 6 cm. 23+0680 – 23+0800: PROJ-280 (01/06/2012) AC 16 surf S 6 cm.

Gainazaleko ezaugarrien arabera errepidea zatitu denean, trafikoaren datuak beste zatiketa egiten du, aurrekoarekin bat ez egitea posiblea izanik. Bi zatiketa hauek ezaugarri bereko tarteak sortzen ditu errodadura geruza eta trafikoari dagokienez, luzera aldakorrarekin (eta horren ondorioz, daturen kopurua (9.4 irudia).



9.4 irudia. Errepidearen luzeraren zatiketaren adibidea errodadura geruza eta trafikoko datuen arabera labainketarekiko erresistentzia modelizatzeko.

Berriro ere, ez da posible datu guztiak sartu (20 m-tan behin).aldakortasun handia dagoela aldagai independenteetan balio bereko behaketetan. Hori dela eta, IRI-en analisian eta aurreko labainketarekiko analisian, tarte bakoitzeko batez besteko balioa eta desbideratzen estandarra kalkulatzen dira. Bide-zoruaren egituraren datuak ere adierazitako irizpideen arabera ere kalkulatzen da.

20 m-ko tarte bakoitzean sartutako informazioa honako hau da (9.71 taula):

- *Tartearen identifikazioa*, IRI-en analisirako erabilitako zutabe berdinak (*A – G zutabeak*) (8.4.1 atalean) eta bide-zoruaren egitura osoa ezaguneko tarteen labainketarekiko erresistentziaren analisirako.

- *Labainketarekiko erresistentziaren datuak:* **SCRIM Koefizientea** (*S1 zutabea*) kontsideratutako 20 m-ko tarterako, 6.6.2 atalean azaldu den gisa, 0tik 100rako eskalan adierazita; eta *egitura* (*textura*) (*S2 zutabea*), mm-tan, Batez besteko Profilaren Sakonera (*Mean Profile Depth*, MPD).

- *Trafiko datuak*. IRI datuetarako eta labainketarekiko erresistentziaren analisirako informazio berdina. *T1 – T5 zutabeak* erabiltzen dira. *T1, T2 eta T3 zutabeak* 2016ko datuak dituzte, datu-biltzearen urtea, eta *T4 eta T5 zutabeek* adierazten dute ireki denetik edo azken M&R lana gauzatu denetik igarotako ibilgailu totalaren kopuru metatua eta ibilgailu astunen kopuru metatua, hurrenez hurren-

- *Euri data:* errepidetik hurbilago dagoen estazio meteorologikoko datu-biltzearen aurreko 15 eguneko prezipitazioaren datua, ***Rain15***, mm-tan, S3 zutabean sartzen da.

- *Polished Stone Value* (Azeleratutako Leuntze Koefizientea), 9.3.1 atalean deskribatutako antzeko informazioa sartzen da: trazatu berria edo M&R lana amaitu zen urtearen H.AADT (*S4 zutabea*), aurreko datuari dagokion trafiko kategoria Ministerio de Fomento (2003b)-ren arabera (*S5 zutabea*) eta azkenik beharrezko PSV minimo (*S6 zutabea*) (9.8 taula).

- *Egiturazko datuak*: Bide-zoruari buruzko informazio gutxi sartu ahal da Errodadura Geruza artxibotik, ez baitauka Bide-zoru Egitura artxiboak ematen duen bezainbeste informazio. Sartutako datuak honako hauek dira: **azken lanaren data**, tartea zerbitzuan jarri zen edo azken M&R lanen data izan daitekeena (*I zutabea*); **azken lanaren urtea** (*J zutabea*); azken lanaren urtea, zenbakiz (K zutabea), **adina** (*age*) (*L zutabea*), F zutabeari J zutabea kenduz (*L zutabea = F zutabea – J zutabea*), **adin erreala** (*real age*) (*M zutabea*), G zutabeari K zutabea kenduz (*M zutabea = G zutabea – K zutabea*) eta **errodadura geruzaren materiala** (*N zutabea*), errodadura geruzan zabaldutako materiala adieraziz.

9.71 taula. Tarte bakoitzeko datuak errodadura geruza ezaguneko labainketarekiko erresistentzia modelizatzeko, BI-631 errepidearen datuekin.

A	B	C	D	E	F	G	S1	S2
Errepidearen izendatzea	**Galtzada**	**Hasierako PK-a**	**Amaierako PK-a**	**Datu-biltzearen data zehatza**	**Datu-biltzearen urtea**	**Datu-biltzearen urtea, zenbakitan**	**SCRIM koefizientea**	**Egitura (mm)**
BI-631	Bi-631 bakarra	19+0990	19+1009	30/06/2016	2016	2016,5	49	0,62
BI-631	Bi-631 bakarra	19+1010	19+1029	30/06/2016	2016	2016,5	54	0,88

T1	T2	T3	T4	T5	S3	S4	S5	S6
AADT 2016an	**H.ADDT 2016an**	**Trafiko kategoria 2016an**	**Ibilgailu totalak**	**Ibilgailu astun totalak**	**Rain15 (mm)**	**H.AADT zerbitzuan jarri zen urtean**	**Trafiko kategoria zerbitzuan jarri zen urtean**	**Eskatutako PSV (Azeleratutako Leuntze Koefizientea)**
11.840	189	T31	20.220.635	432.605	34,3	337	T2	50
11.840	189	T31	20.220.635	432.605	34,3	337	T2	50

I	J	K	L	M	N
Azken lanaren data	**Azken lanaren urtea**	**Azken lanaren urtea, zenbakiz**	**Age**	**Real Age**	**Errodadura geruzaren materiala**
01/07/2011	2011	2011,5	5	5	LB2
01/07/2011	2011	2011,5	5	5	LB2

Metodologia hau erabiliaz, eta aldagaietan balio berak dituzten tarteen batez bestekoak kalkulatuz (eta desbideratze tipikoa 9.2 ekuazioa), tarte gehiago daude errepide bakoitzetik. 9.72 taulak erakusten du errepide bakoitzean kontsideratutako tartea, bi erreiko errepide konbentzionaletan edo galtzada banandutako errepideak diren.

9.4.1. Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko tarteetarako labainketarekiko erresistentziaren portaera modeloak

Hasieran, bi erreiko errepide konbentzionalak baino ez ziren aztertu. Hori dela eta, Eremu 1, 2 eta 3 bakarrik kontuan hartu ziren, Eremu 4tan sartzen dira Bilboko inguruetan dauden autobideak eta autobiak (bi erreiko tarte batzuk gora behera)

Eremu 1, 2 eta 3etan 112 eta 321 datu daude labainketarekiko erresistentziaren analisirako bi erreiko errepideetan. Aurretik adierazitako irizpideak jarraituz 2 urte baino gutxiagoko adina erreala duten tarteak baztertzen dira (*R.Age* < 2 urte) (44 tarte) oreka fasera heldu ez direlako. Datu hauek kenduta, 389 datu daude modelorako.

9.72 taula. Eremu 1, 2 eta 3ko errepide bakoitzeko kontsideratutako tarteak haien errodadura geruza eta trafiko bolumenen arabera labainketarekiko erresistentzia modelorako (bi erreiko errepide konbentzionalekoak eta galtzada bananduko errepidetakoak)

Eremu 1			**Eremu 2**			**Eremu 3**		
Errepidea	**2e 2016**	**2g 2016**	**Errepidea**	**2e 2016**	**2g 2016**	**Errepidea**	**2e 2016**	**2g 2016**
BI-631	9		BI-623	9	2	BI-624	2	
BI-634	7		BI-633	15		BI-625	20	5
BI-635	15		BI-638	2		BI-630	12	
BI-735	5		BI-732	4		BI-712	4	
BI-737	7		BI-2224	5		BI-745	3	
BI-2101	2		BI-2301	2		BI-2521	4	
BI-2120	11		BI-2405	4		BI-2522	6	
BI-2121	26		BI-2543	6		BI-2617	0	
BI-2122	7		BI-2632	1		BI-2625	2	
Bi-2153	5		BI-2636	2		BI-2701	27	
Bi-2235	9		N-240	38		BI-2757	0	
Bi-2237	6		N-636	0		BI-2794	1	
Bi-2238	13					N-639	5	
BI-2704	11							
BI-2713	10							
BI-2731	4							
Guztira	147		**Guztira**	88	2	**Guztira**	86	5
Errepide konbentzionalak (Eremu 1, 2, 3), guztira					321			
Galtzada banandutako errepideak (Eremu 1, 2, 3) guztira					7			
GUZTIRA					328			

9.73 erakusten du aldagai kuantitatiboen esplorazio-analisia erakusten du. Aldagai bat ere ez du banaketa normal (9.74 taula).

Mendeko aldagaiaren eta aldagai independente kuantitatiboen arteko korrelazioak ikusgai daude 9.75 taulan. Mendeko aldagai eta aldagai independente bakoitzaren arteko datuak hoberen doitzen dituzten kurbak 9.76 taulan erakusten dira.

Aldagai independente kualitatiborako, *SurfType*, aurreko ataletan burututako antzeko analisia gauzatu da (9.77 taula). Ikus daiteke bariantzak ez dira homogeneoak (9.78 taula) eta esan daiteke ez dagoela batez bestekoen arteko berdintasunik (9.79 taula eta 9.80 taula) %95eko konfiantza-maila.

Post hoc konparazioek erakusten du ez dagoela batez besteko berdintasunik Kare-esnea eta BBTM&PA-ren artean eta Kare-esnea eta AC-ren artean, baina ez dagoela diferentziarik AC eta BBTM&PA-ren artean (9.81 taula).

9.73 taula. Aldagai kuantitatiboen esplorazio-analisia Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15
Batez bestekoa		53,25	7261,70	318,29	7,50	7,47	19482,19	1058,89	47,46	27,08
Errore estandarra		0,46	297,30	15,95	0,25	0,25	1147,80	81,02	0,18	0,73
%95eko KT-ak	**Beheko**	52,35	6677,19	286,92	7,00	6,98	17225,51	899,60	47,11	25,64
	Goiko	54,15	7846,22	349,65	8,00	7,97	21738,87	1218,18	47,80	28,52
Bariantza		82,334	34381643	99004	24,843	24,948	512483095	2553390	11,88	208,27
Desbideratze tip.		9,074	5864	315	4,984	4,995	22638	1598	3,45	14,43
Minimoa		33	654	10	2	2	936,04	13,96	40	2,1
Maximoa		85,875	29916	1361	29	28,6	165782,51	8022,56	50	45,8
Anplitudea		52,875	29262	1351	27	26,6	164846,47	8008,6	10	43,7
Kuartileko heina		13,351	7267	327	6	5,7	19439,81	747,16	5	25,4
Simetria		0,312	1,42	1,53	1,566	1,585	2,85	2,457	-0,956	-0,75
Kurtosia		-0,038	2,408	1,542	3,004	3,01	10,765	5,863	-0,414	-1,093

9.74 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagaiak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Estatistikoa	**Askatasun-graduak**	**Esangura (p-balioa)**	**Estatistikoa**	**Askatasun-graduak**	**Esangura (p-balioa)**
MSSC	0,048	389	0,034	0,991	389	0,019
AADT	0,145	389	<0,001	0,868	389	<0,001
H.AADT	0,211	389	<0,001	0,792	389	<0,001
Age	0,135	389	<0,001	0,855	389	<0,001
R.Age	0,137	389	<0,001	0,853	389	<0,001
TotVeh	0,206	389	<0,001	0,702	389	<0,001
TotH.Veh	0,293	389	<0,001	0,629	389	<0,001
PSV	0,379	389	<0,001	0,708	389	<0,001
Rain 15	0,293	389	<0,001	0,8	389	<0,001

[a] Lilliefors Esangura zuzenketa

9.75 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
AADT	-0,541	<0,001
H.AADT	-0,459	<0,001
Age	-0,093	0,067
R.Age	-0,086	0,089
TotVeh	-0,407	<0,001
TotH.Veh	-0,34	<0,001
PSV	-0,305	<0,001
Rain 15	-0,078	0,123

9.76 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duten ekuazioak Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Ekuazio-mota	Modeloaren laburpena					Parametroen estimazioak			
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
AADT	Logaritmikoa	0,342	200,865	1	387	<0,001	103,837	-5,922	-	-
H.AADT	Logaritmikoa	0,274	145,709	1	387	<0,001	78,736	-4,804	-	-
Age	Lineala	0,009	3,372	1	387	0,067	54,519	-0,169	-	-
R.Age	Lineala	0,007	2,91	1	387	0,089	54,423	-0,157	-	-
TotVeh	Logaritmikoa	0,303	168,18	1	387	<0,001	97,735	-4,755	-	-
TotH.Veh	Logaritmikoa	0,252	130,093	1	387	<0,001	75,083	-3,562	-	-
PSV	Lineala	0,093	39,677	1	387	<0,001	91,342	-0,803	-	-
Rain 15	Esponentziala	0,009	3,465	1	387	0,063	54,096	-0,001	-	-

9.77 taula. SurfType aldagai kualitatiboen esplorazio-analisia mailaz bilduta mendeko aldagaiarekiko (MSSC) Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		SurfType		
		AC (1)	**BBTM& PA (2)**	**Kare-esnea (3)**
N		143	59	187
Batez bestekoa		50,348	48,407	56,997
Errore estandarra		0,749	0,753	0,621
Batez bestekoa	**Beheko**	48,868	46,900	55,772
	Goikoa	51,828	49,914	58,222
Bariantza		80,166	33,431	72,097
Desbideratze tipikoa		8,954	5,782	8,491
Minimoa		33	37,744	38
Maximoa		73,318	63,495	85,875
Anplitudea		40,318	25,751	47,875
Kuartileko heina		12,635	8,131	12,006
Simetria		0,192	0,775	0,347
Kurtosia		-0,446	0,197	0,142

9.78 taula. Bariantzaren homogeneotasunaren testa SurfType-ren mailetarako MSSC-rekiko Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
6,156	2	386	0,002

9.79 taula. Batez bestekoen berdintasunaren test sendoak SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Testa	Estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
Welch	44,948	2	189,160	< 0,001
Brown-Forsythe	44,749	2	343,551	< 0,001

9.80 taula. Bariantzaren analisia (ANOVA) SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagaia (SurfType)	Karratuen batuketa	Askatasun-graduak	Batez besteko karratua	F	Esangura
Taldeen artean	5213,067	2	2606,534	37,636	< 0,001
Taldeen barruan	26732,712	386	69,256		
Total	31945,780	388			

9.81 taula. Konparazio anizkoitza SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Estatistikoa	SurfType (I)	SurfType (J)	Batez besteko diferentzia (I-J)	Errore estad.	Esang.	%95eko Konfiantza-tarteak	
						Beheko	Goikoa
Tamhane's T2	AC	BBTM&PA	1,941	1,062	0,194	-0,620	4,503
		Kare-esnea	-6,649	0,973	<0,001	-8,984	-4,313
	BBTM&PA	AC	-1,941	1,062	0,194	-4,503	0,620
		Kare-esnea	-8,590	0,976	<0,001	-10,947	-6,232
	Kare-esnea	AC	6,649	0,973	<0,001	4,313	8,984
		BBTM&PA	8,590	0,976	<0,001	6,232	10,947
Dunnett's T3	AC	BBTM&PA	1,941	1,062	0,193	-0,619	4,502
		Kare-esnea	-6,649	0,973	<0,001	-8,984	-4,313
	BBTM&PA	AC	-1,941	1,062	0,193	-4,502	0,619
		Kare-esnea	-8,590	0,976	<0,001	-10,946	-6,233
	Kare-esnea	AC	6,649	0,973	<0,001	4,313	8,984
		BBTM&PA	8,590	0,976	<0,001	6,233	10,946

Erregresio lineal anizkoitzeko modelo batzuk frogatu ziren. Determinazio-koefizienterik onena daukan modeloak Ln*AADT* eta Ln*TotVeh* erabiltzen ditu aldagai bezala (9.82 taula). Hala ere, auto-korrelazio arazoak ikus daitezke: DW estatistikoa = 1,137 (9.82 taula) eta Condition Index > 30 (9.83 taula).

9.82 taula. taula. Proposatutako erregresio lineal anizkoitzaren modeloaren bariantzaren analisia (ANOVA) Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Analysis of variance						
Iturria	**Karratuen batuketa**	**Batez besteko karratua**	**F balioa**	**p-balioa**	**Durbin-Watson**	**Batez besteko karratua**
Modeloa	11619,972	2	5809,986	110,335	<0,001	1,137
Errorea	20325,808	386	52,658			
Total zuzenduta	31945,780	388				
Estimazioaren errore estandarra	**Aldakuntza-koefizientea**	**R**	**R^2**	**R^2_{adj}**	**Kolinealtasunaren estatistikoak**	
					Tolerantzia	**Tolerantzia**
	7,257	0,603	0,364	0,360	0,386	2,593
Parametroen estimazioak						
Aldagaia	**Parametroaren estimazioa**	**Errore estandarra**	**t balioa**	**p-balioa**	**%95eko konfiantza-tarteak**	
Intercept	106,944	3,633	29,433	<0,001	99,800	114,088
LnAADT	-4,023	0,662	-6,074	<0,001	-5,325	-2,721
LNTotVeh	-2,066	0,565	-3,658	<0,001	-3,177	-0,956

9.83 taula. Kolinealtasunaren diagnosia

Modeloa	Dimentsioa	Balio propioa	Condition Index
1	1	2,991	1,000
	2	0,007	20,703
	3	0,002	34,664

Aitzitik, LnAADT eta PSV_{req} aldagaiak dituen erregresio lineal anizkoitzeko modeloak berdina dauka (0,354) eta Durbin Watson estatistikoa 1,136 da eta antzeko balioak condition index indizean.

Hainbat Erregresio Lineal Orokor Anizkoitzeko modelo aztertu ziren. Aurreko esperientziarekin eta mendeko aldagaiaren eta aldagai independenteen arteko korrelazioak kontuan hartuta, aldagai independenteen hainbat konbinazio frogatu dira, aldagaia koaldagaiak edo faktore (aldagai kualitatiboa) bezala sartuz. Adibidez, PSV_{req}, bere balioak mailak bezala kontsideratu ahal dira. 9.84 taulak erakusten ditu aztertutako ekuazio batzuk, determinazio-koefizientea R^2, doitutako R^2, eta agertu diren arazoak edo oharrak. Behin-behineko analisi hau modelo bakoitzeko Subjektu-arteko Efektuen testetatik dator, aldagai bakoitzaren esangura adierazten duena F testaren bidez. Horrez gain, modeloaren parametroen estimazioetatik koefiziente bakoitzeko esangura ere jakin daiteke. Hauek dira egiaztatu behar diren lehenengo datuak, 7. kapituluan azaldu zen bezala. 9.84 taulan, modelo bakoitzari buruzko oharrak azaltzen dira azken zutabean. Modelo guztiek konstantea daukate eta faktore bezala kontsideratzen diren aldagaiak (f) adierazlea daukate.

SurfType aldagaiaz gain, errodadura geruzaren materiala Hormigoi Bituminosoan (AC), nahaste etenak eta drainatzaileak (BBTM&PA) eta LB2 (Kare-esnea) taldeetan zatitzen duena, beste aldagai bat erabili da *SurfDen*, 9.11 taulan erakutsi zen bezala. Aldagai hau gehiago berezitzen ditu talde hauen artean eta aukerak (mailak) hauek dira: AC 16 surf S, AC 22 surf S, BBTM 11A, PA 11 and LB2. Errodadura geruzetan bakarrik material hauek aurkitu ahal dira errepide konbentzionaletan Eremu 1, 2 eta 3an.

9.84 taulan ikusten den bezala, aldagai guztiak esanguratsuak izanik (p-balioa < 0,05) modelorik onenak Ln*AADT* aldagai kuantitatiboa (koaldagaia) eta *SurfType* edo *SurfDen* faktore finko bezala. Ln*AADT* aldagaia modeloan sartzen zen aldagai bakarra izanda eta *SurfType* eta *SurfDen* erabilitako faktore bakarrak, modelo independenteak sortzen saiatu zen errodadura mota bakoitzerako.

ala eta guztiz ere, lortutako determinazio-koefizienteak txikiagoak ziren eta *SurfType* aldagaiaren mailen araberako modeloak ez zuen determinazio-koefizientean hobekuntzarik sortzen (AC-rako: R^2 = 0,392; BBTM&PA-rako, R^2 =0,283 eta Kare-esnerako, R^2 =0,44).

9.84 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak Eremu 1, 2 eta 3ko errepide konbentzionaletako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
MSSC = Int + LnAADT + LnH.AADT + PSV + SurfType(f)	0,491	0,484	Esangura baxua, LnH.AADT (p=0,798),PSV (p=0,955)
MSSC = Int + LnAADT + LnH.AADT + PSV(f) + SurfType(f) + PSV*SurfType	0,497	0,482	Esangura baxua, LnH.AADT (p=0,592), PSV (p=0,748), PSV*SurfType (p=0,454)
MSSC = Int + LnAADT + PSV + SurfType(f)	0,490	0,485	Esangura baxua, PSV (p=0,987)
MSSC = Int + LnAADT + PSV(f) + SurfType(f)+ PSV*SurfType	0,496	0,483	Esangura baxua, PSV (p=0,768), PSV*SurfType (p=0,480)
MSSC = Int + LnAADT + PSV + SurfType(f)	0,490	0,485	Esangura baxua, PSV (p=0.,987)
MSSC = Int + LnAADT + SurfType(f)	0,490	0,486	Aldagai guztiak esanguratsuak (p-balioa<0,001)
MSSC = Int + LnH.AADT + PSV + SurfType(f)	0,407	0,401	Aldagai guztiak esanguratsuak (p-balioa <0,02)
MSSC = Int + LnAADT + LnH.AADT + SurfType(f)	0,491	0,485	Esangura baxua, LnH.AADT (p=0,803)
MSSC = Int + LnAADT + LnH.AADT + PSV + SurfDen(f)	0,491	0,482	Esangura baxua, LnH.AADT (p=0,735) PSV (p=0,994)
MSSC = Int + LnAADT + LnH.AADT + SurfDen(f)	0,491	0,483	Esangura baxua, LnH.AADT (p=0,731)
MSSC = Int + LnAADT + SurfDen(f)	0,491	0,484	Aldagai guztiak esanguratsuak (p-balioa <0,001)

Ikerketaren hurrengo pausuan, labainketarekiko erresistentziaren modeloa garatzeko galtzada banandutako errepideetako tarte eskuragarriak ere sartzea erabaki zen. IRI analisian, 2 errei edo gehiago galtzada batean noranzko batean egoteak trafiko astuna handiagoa duen erreian eragina izan dezake zeren eta hain kargatuta ez dagoen beste erreiak deformazioren parte bat murriztu behar (IRI-en bidez neurtu ahal dena. Aitzitik bi erreiko errepide konbentzional batean, bi erreiak trafiko astun berdina daukatenez, portaera ezberdina izango litzateke IRI aztertzean. Hala eta guztiz ere, errei gehiago egoteak ez du eraginik izango trafiko astun handiagoko erreiaren gaineko leuntze efektuan. Horren ondorioz, Eremu 1, 2 eta 3ko galtzada banandutako errepideetako tarteak ere sartu dira analisian. Korrelazioetan eta modeloetan hobekuntzak ikusten badira, Eremu 4ko tarteak, batez ere autobideak eta autobideak dituen eremua, ere sartuko dira,

9.4.2. Eremu 1, 2 eta 3ko errepide guztietako errodadura geruza ezaguneko tarteetarako labainketarekiko erresistentziaren portaera modeloak

9.9 taulan eta 9.72 taulan, 451 tarte daude (123+328) errodadura geruza ezaguneko hainbat proiektutatik eta hainbat trafiko bolumenetatik sortutakoak. Berriro ere, aurreko irizpideak jarraituz, bi urte baino adina errealeko tarteak baztertzen dira, Kokkalis (1998)-ek gomendatzen duen bezala. Gainera, tarte batzuetan mikro-fresaketa esperimentala gauzatu zuten 2015 eta 2016an marruskadura hobetzeko, eta ondorioz, tarte hauek kendu behar dira.456 datuetatik, 46 tartek ez daukat 2 urte eta beraz, baztertzen dira. Eremu 1, 2 eta 3ko errepideetan analizatzeko dagoen tarteen kopurua 405 da.

9.85 taulak erakusten du mendeko aldagaiaren eta aldagai independente kuantitatibo bakoitzaren arteko Pearson-en R korrelazio-koefizientea. 9.86 taulak adierazten ditu mendeko aldagaiaren eta aldagai independente bakoitzaren arteko datuak hobeto doitzen dituzten kurbak. Taula hauetako balioak 9.75 eta 9.76 tauletako datuekin konparatuz gero hobekuntza ikus daiteke erlazionatutako aldagai gehienentzat

9.85 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) Eremu 1, 2 eta 3ko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
AADT	-0,545	<0,001
H.AADT	-0,459	<0,001
Age	-0,104	0,037
R.Age	-0,097	0,05
TotVeh	-0,4	<0,001
TotH.Veh	-0,355	<0,001
PSV	-0,31	<0,001
Rain 15	-0,084	0,091

9.86 taula. Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duten ekuazioak Eremu 1, 2 eta 3ko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Aldagai independenteak	Ekuazio-mota	Modeloaren laburpena					Parametroen estimazioak			
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
AADT	Logaritmikoa	0,344	211,074	1	403	< 0,001	103,73	-5,907	-	-
H.AADT	Logaritmikoa	0,276	153,706	1	403	< 0,001	78,95	-4,862	-	-
Age	Lineala	,011	4,372	1	403	,037	54,488	-0,189	-	-
R.Age	Lineala	,009	3,865	1	403	,050	54,397	-0,177	-	-
TotVeh	Logaritmikoa	0,310	181,189	1	403	< 0,001	129,95	-4,717	-	-
TotH.Veh	Logaritmikoa	0,261	142,582	1	403	< 0,001	100,05	-3,595	-	-
PSV	Lineala	0,096	42,937	1	403	< 0,001	92,18	-0,822	-	-
Rain15	Lineala	,007	2,877	1	403	,091	54,532	-0,054	-	-

Erregresio Lineal Orokor Anizkoitzeko (GLM) modelo batzuk 9.4.2 ataleko irizpideak jarraituz frogatu ziren. 9.87 taulak erakusten itu aztertutako modelo batzuk eta haiei buruzko komentarioak. Taula burututako analisiaren laburpena da.

9.87 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak Eremu 1, 2 eta 3ko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Proposatutako modeloak	R^2	R^2_{adj}	Komentarioak eta oharrak
MSSC = Int + LnAADT + PSV(f) + SurfType(f)+ + PSV*SurfType	0,499	0,486	Esangura baxua, PSV (p=0,767), PSV*SurfType (p=0,451)
MSSC = Int + LnH.AADT + PSV(f) + SurfType(f)+ + PSV*SurfType	0,413	0,398	Esangura baxua, PSV (p=0,575), PSV*SurfType (p=0,884)
MSSC = Int + LnAADT + SurfType(f)	0,493	0,489	Aldagai guztiak esanguratsuak (p-balioa <0,001)
MSSC = Int + LnH.AADT + PSV + SurfType(f)	0,410	0,404	Aldagai guztiak esanguratsuak (p-balioa <0,02)

Ikusten den bezala, hobekuntza txikia ikusten da bi erreiko errepideak baino aztertzen ez zirenean lortzen ziren balioekin (9.84 taula). Hobekuntza txiki hori geratzen da analisian sartutako galtzada banandutako tarte berrien kopuru txikiagatik. Lehen 389 datu zeuden eta puntu berriak sartzean 405, hau da galtzada bananduko 16 tarte berri. Horren ondorioz, Eremu 4ko tarteak ere sartzea erabili da. Eremu horretan batez ere galtzada banandutako tarteak daude.

9.4.3. Errepide guztietako errodadura geruza ezaguneko tarteetarako labainketarekiko erresistentziaren portaera modeloak (Eremu 1, 2, 3 eta 4)

Aurreko atalean ondorioztatu zenez, Eremu 1, 2 eta 3ko galtzada banandutako tarteen datuak labainketarekiko erresistentziaren modeloan sartzeak, errorea ez sartzeaz gain, emaitzak ere hobetu zituen haiek sartu gabeko modeloekin konparatuta. Horren ondorioz, edozein motako errepideak (bi erreikoa edo galtzada banandutako) barne sartzen dituen marruskadura iragartzeko modeloa garatzea erabaki zen. Hori dela eta, Eremu 4ko errepideetako tarteak, Bilboko Eremu Metropolitarraren errepide nagusiak barne hartzen dituena, hau da Bilboko eraztun metropolitarraren autobiak eta errei anitzeko errepideak, sartzen dira modelizazioan. Eremu 4ko errepideen Errodadura Geruza artxiboaren antzeko analisia egiten da, errodadura geruza eraldatu dituzten proiektuak aztertuz.

Errepidearen zatiketa osoa, galtzadaz bananduta (gorantz, beherantz edo bakarra bi erreiko errepide konbentzionaleko tarteetan) erakusten dira II. eranskineko Eremu 4ko Errepide Artxiboetan, "*Skid resistance of surface layers in 2016*" atalean. 9.88 taulak erakusten du errepide bakoitzean kontsideratutako tarteak (galtzada bakarreko edo banandutako galtzadetan berezita), 9.4 irudian azaldutako irizpideen arabera.

9.88 taula. Eremu 4ko errepide bakoitzeko kontsideratutako tarteak, haien errodadura geruza eta trafiko bolumenen arabera labainketarekiko erresistentzia modelorako (bi erreiko errepidekoak eta galtzada bananduko errepidetakoak)

	Eremua 4	
Errepideak	**Bi erreiko errepideak**	**Galtzada banandutako errepideak**
A-8	-	117
BI-604	-	24
BI-628	6	16
BI-631	-	37
BI-636	17	32
BI-637	1	36
BI-644	-	10
BI-647	4	6
BI-738	-	-
N-633	-	13
N-634	64	32
N-637	-	58
N-644	-	4
Guztira	**92**	**385**
GUZTIRA	**477**	

Eremu 4ko tarte berriak gehitzen dira Eremuko 1, 2 eta 3ko 451 datuei. 928 datu totaletatik, 114 tartek ez dute adina errealeko 2 urte. Haiek baztertu ondoren, aurretik aipatutako irizpideak jarraituz, guztira 814 behaketa analizatzen ziren. Data hauek Bizkaiko Foru Aldundiak kudeatzen dituen errepide osoko tarteak irudikatzen dituzte proiektu ezberdin batetik datorren errodadura geruza (adina ezberdina suposatzen duena) eta trafiko bolumen ezberdin batekin (bai trafiko totalean, *AADT*, bai astunen trafikoan, *H.AADT*).

Aldagaien esplorazio-analisia 9.89 taulan erakusten da. Normaltasun testek adierazten dute aldagaietako batek ere ez dauka banaketa normal (9.90 taula).

Mendeko aldagaiaren eta aldagai independente kuantitatiboen arteko korrelazioak 9.91 taulan erakusten dira. Aurreko analisietan bezala, korrelaziorik onenak *AADT*, *H.AADT* eta PSV_{req} aldagaiekin lortzen dira, nahiz eta *TotVeh* eta *TotHVeh* aldagaiekin ere korrelazio onak aurki daitezkeen. *Rain15*, *Age* eta *R.Age* aldagaiek korrelazio oso baxua daukate. Beste analisietan bezala, aldagaien eraldatzeak aztertu dira. Lehenengo eta behin, mendeko aldagaiaren aldagai independente bakoitzaren arteko datuak hoberen doitzen duten kurbak kalkulatu dira (9.92 taula).

Orduan, AADT eta H.AADT aldagaiek lortu zituztela korrelaziorik onenak logaritmo naturalaren bidez, eraldatutako aldagai hauek ere aztertu ziren berriro, datuak hoberen doitzen duten kurbarik onenak lortzeko. Eraldatutako aldagaien eta *MSSC*-ren arteko korrelazioak ere aztertu ziren (9.93 taula).

9.89 taula. Aldagai kuantitatiboen esplorazio-analisia errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15
Batez bestekoa		49,51	27109,59	809,94	8,53	8,54	88494	3252,04	49,09	28,32
Errore estandarra		0,28	1147,03	33,28	0,21	0,21	5018	188,7	0,13	0,50
%95eko KT-ak	**Beheko**	48,95	24858,10	744,62	8,12	8,13	78655,5	2882,04	48,84	27,34
	Goiko	50,06	29361,08	875,26	8,93	8,94	98343,2	3622,82	49,34	29,29
Bariantza		65,99	1070960304	901324	34,924	34,92	20491922500	28983978	12,93	201,4
Desbideratze tip.		8,12	32725,5	949,4	5,91	5,91	143153	5386,67	3,60	14,2
Minimoa		31	654	10	2	2	93604	13,96	40	2,1
Maximoa		85,88	139210	4872	36	36,5	1143417	48379	56	46,9
Anplitudea		54,88	138556	4862	34	34,5	1142481	48365	16	44,8
Kuartileko heina		9,81	28196	770	7	7	101331	3393,6	0	25,4
Simetria		0,802	1,728	1,871	1,461	1,474	3,143	3,514	-0,527	-0,732
Kurtosia		0,842	2,118	3,163	2,565	2,614	11,680	15,84	0,866	-0,95

9.90 taula. Normaltasun testak mendeko aldagai eta aldagai independente kuantitatiboentzat errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagaiak	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)	Estatistikoa	Askatasun-graduak	Esangura (p-balioa)
MSSC	0,080	814	< 0,001	0,962	814	< 0,001
AADT	0,268	814	< 0,001	0,835	814	< 0,001
H.AADT	0,239	814	< 0,001	0,743	814	< 0,001
Age	0,216	814	< 0,001	0,750	814	< 0,001
R.Age	0,364	814	< 0,001	0,776	814	< 0,001
TotVeh	0,135	814	< 0,001	0,871	814	< 0,001
TotH.Veh	0,134	814	< 0,001	0,871	814	< 0,001
PSV	0,270	814	< 0,001	0,602	814	< 0,001
Rain 15	0,274	814	< 0,001	0,588	814	< 0,001
TotBit	0,239	814	< 0,001	0,742	814	< 0,001

[a] Lilliefors Esangura zuzenketa

9.91 taula. Korrelazioa mendeko aldagai eta aldagai independenteren artean (Pearson-en R koefizientea) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
AADT	-0,343	< 0,001
H.AADT	-0,307	< 0,001
Age	-0,118	0,001
R.Age	-0,118	0,001
TotVeh	-0,263	< 0,001
TotH.Veh	-0,23	< 0,001
PSV	-0,35	< 0,001
Rain 15	-0,086	0,014

9.92 taula Aldagai independente bakoitzaren eta mendeko aldagaiaren arteko korrelazio onena ematen duten ekuazioak errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Ekuazio-mota	Modeloaren laburpena					Parametroen estimazioak			
		R^2	F	Askatasun-gradu 1	Askatasun-gradu 2	Esang.	Konstantea	b1	b2	b3
AADT	Logaritmikoa	0,330	400,031	1	812	< 0,001	84,332	-3,668	-	-
H.AADT	Logaritmikoa	0,266	294,514	1	812	< 0,001	70,766	-3,511	-	-
Age	Lineala	0,014	11,505	1	812	0,001	50,891	-0,162	-	-
R.Age	Lineala	0,014	11,560	1	812	0,001	50,896	-0,163	-	-
TotVeh	Logaritmikoa	0,310	364,041	1	812	< 0,001	102,727	-3,073	-	-
TotH.Veh	Logaritmikoa	0,266	294,656	1	812	< 0,001	88,178	-2,766	-	-
PSV	Lineala	0,122	113,339	1	812	< 0,001	88,324	-0,791	-	-
LnAADT	Alderantzizkoa	0,367	470,866	1	812	< 0,001	15,003	321,457		
LnH.AADT	Alderantzizkoa	0,300	348,229	1	812	< 0,001	30,009	113,003		

9.93 taula. Korrelazioa mendeko aldagai eta eraldatutako aldagai independenteren artean (Pearson-en R koefizientea) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagai independenteak	Korrelazioa MSSC-ekin (R)	Korrelazioren esangura (bi aldekoa)
LnAADT	-0,574	< 0,001
LnH.AADT	-0,516	< 0,001
1/LnAADT	0,606	< 0,001
1/LnH.AADT	0,548	< 0,001
(AADT)^1/2	-0,454	< 0,001
(H.AADT)^1/2	-0,407	< 0,001
LnAADT	-0,556	< 0,001
LnH.AADT	-0,516	< 0,001

SurfType aldagai independente kualitatiboa, Hormigoi Bituminosoen (AC), nahaste etena eta drainatzaileen (BBTM&PA) eta kare-esneen artean berezitzen dituena, aztertu zen (9.94 taula).

Aldagai kualitatibo berri bat sortu zen, *RoadType* izenekoa, bi erreiko errepide konbentzionalen (2L) eta galtzada banandutako errepideen artean (2C) berezitzen dituena, bereizketa honek labainketarekiko erresistentzian eragina duen egiaztatzeko.

SurfType-ren mailetarako bariantzaren analisi bat (ANOVA) gauzatu zen. Levene estatistikoak adierazten du bariantza homogeneorik ez daudela (p-balioa<0,001). Batez bestekoen berdintasunerako test sendoek eta ANOVA analisiak erakusten dute onartzen dela batez bestekoak ezberdinak direla %95eko konfiantza-mailarekin (9.96 taula eta 9.97 taula).

Horrez gain, Tamhane's T2 eta Dunnett's T3 estatistikoek adierazten dute ez dagoela edozein *SurfType*-ren bi mailaren arteko berdintasunik (p < 0,05) (9.98 taula).

9.94 taula. Aldagai kualitatiboen esplorazio-analisia mailaz bilduta mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		RoadType		**SurfType**		
		2L	**2C**	**AC (1)**	**BBTM&PA (2)**	**Kare-esnea (3)**
N		475	339	196	401	217
Batez bestekoa		52,21	45,71	49,348	45,899	56,313
Errore estandarra		0,40	0,27	0,595	0,249	0,564
%95eko Konfiantza-tarteak	**Beheko**	51,42	45,18	48,174	45,410	55,201
	Goiko	53,01	46,25	50,522	46,387	57,425
Bariantza		77,66	25,09	69,428	24,793	69,053
Desbideratze tipikoa		8,81	5,01	8,332	4,979	8,310
Minimoa		33	31	33,000	31,000	38,000
Maximoa		85,88	64,04	73,318	63,495	85,875
Anplitudea		52,88	33,04	40,318	32,495	47,875
Kuartileko heina		12,46	6,51	10,718	6,356	10,703
Simetria		0,453	0,214	0,383	0,236	0,440
Kurtosia		0,116	0,786	-0,183	0,734	0,237

9.95. taula. Bariantzaren homogeneotasunaren testa SurfType-ren mailetarako MSSC-rekiko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa	**Askatasun-graduak 1**	**Askatasun graduak 2**	**Esangura**
46,471	2	811	< 0,001

9.96 taula. Batez bestekoen berdintasunaren test sendoak SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Testa	Estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
Welch	146,218	2	355,846	< 0,001
Brown-Forsythe	131,787	2	506,852	< 0,001

9.97 taula. Bariantzaren analisia (ANOVA) SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Aldagaia (SurfType)	Karratuen batuketa	Askatasun-graduak	Batez besteko karratua	F	Esangura
Mailen artean	15278,619	2	7639,309	161,462	< 0,001
Mailaren barruan	38371,227	811	47,313		
Guztira	53649,846	813			

SurfType aldagaiaren maila bakoitzaren barruan errodadura geruzako material gehiago daudenez, *SurfDen*-en mailak (9.11 taulan erakutsitakoak); *SurfDen* ezarri zen aldagai independente kualitatibo berri bat bezala eta analizatu zen (9.99 taula).

SurfDen-erako Levene estatistikoak adierazten du mailek ez daukatela bariantza homogeneoak. Hala ere, batez bestekoaren berdintasunerako testa ez da gauzatu maila batzuek balio bakarra daukatelako.

Horren ondorioz, beharrezkoa dirudi material batzuk ia berdinak diren izendatze batzuetan batzea. 9.11 taula oinarria bezala hartuta, aldagai independente kualitatibo berri bat sortu zen, SurfDen2, antzeko errodadura geruzak batzen dituena maila bakoitzean datu kopuru minimo bat izateko (9.100 taula). Hormigoi bituminosoak (AC) agregakinen diametro maximoaren arabera zatitzea, 16 eta 22 mm, eta ez kontsideratu nahastearen gradazioa (trinkoa, D eta erdi-trinkoa, S).

SurfDen2 aldagai berriaren esplorazio-analisia 9.101 taulan aurkezten da. Levene estatistikoa erakusten du mailek ez dauzkatela bariantza homogeneoak (p-balioa< 0,01). Bai batez bestekoen berdintasunerako test sendoek bai ANOVA analisiak adierazten dute ez dagoela batez bestekoen berdintasunik (9.103 taula eta 9.104 taula).

Tamhane’s T2 estatistikoak seinalatzen du zein mailaren artean dagoen batez bestekoen berdintasuna (9.105 taula). Kasu honetan, Dunnett’s T3 estatistikoa ez da aurkeztu balio berdinak, eta beraz, ondorio berdinak, ematen zituelako.

9.98 taula. Konparazio anizkoitza SurfType-ren mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Estatistikoa	SurfType (I)	SurfType (J)	Batez besteko diferentzia (I-J)	Errore estad.	Esang.	%95eko Konfiantza-tarteak	
						Beheko	Goiko
Tamhane's T2	AC	BBTM&PA	3,449	0,645	< 0,001	1,899	4,999
		Kare-esnea	-6,965	0,820	< 0,001	-8,931	-4,999
	BBTM&PA	AC	-3,449	0,645	< 0,001	-4,999	-1,899
		Kare-esnea	-10,415	0,616	< 0,001	-11,895	-8,934
	Kare-esnea	AC	6,965	0,820	< 0,001	4,999	8,931
		BBTM&PA	10,415	0,616	< 0,001	8,934	11,895
Dunnett's T3	AC	BBTM&PA	3,449	0,645	< 0,001	1,900	4,999
		Kare-esnea	-6,965	0,820	< 0,001	-8,931	-4,999
	BBTM&PA	AC	-3,449	0,645	< 0,001	-4,999	-1,900
		Kare-esnea	-10,415	0,616	< 0,001	-11,895	-8,935
	Kare-esnea	AC	6,965	0,820	< 0,001	4,999	8,931
		BBTM&PA	10,415	0,616	< 0,001	8,935	11,895

9.99 taula. SurfDen aldagai kualitatiboen esplorazio-analisia mailaz bilduta mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		SurfDen							
		AC 16 surf S	AC 16 surf D	AC 22 surf D	AC 22 surf S	BBTM 11A	BBTM 11B	PA 11	LB2
N		174	3	1	18	312	10	79	217
Batez bestekoa		49,173	48,667		50,645	46,38	47,80	43,77	56,31
Errore estandarra		0,627	2,228		2,262	0,28	1,30	0,50	0,56
%95eko Konfiantza-tarteak	Beheko	47,936	39,080		45,873	45,82	44,86	42,78	55,20
	Goiko	50,410	58,253		55,418	46,93	50,73	44,77	57,43
Bariantza		68,336	14,893		92,098	24,96	16,79	19,73	69,05
Desbideratze tipikoa		8,267	3,859		9,597	5,00	4,10	4,44	8,31
Minimoa		33,000	45,600		33,944	31,00	41,19	33,33	38,00
Maximoa		73,318	53,000		64,625	63,49	54,67	53,75	85,88
Anplitudea		40,318	7,400		30,681	32,49	13,48	20,42	47,88
Kuartileko heina		10,486	.		13,621	6,45	6,63	5,87	10,70
Simetria		0,478	1,318		-0,307	0,26	-0,03	-0,01	0,44
Kurtosia		-0,043	.		-0,750	0,87	-0,41	-0,02	0,24

9.100 taula. Errodadura geruzarako aldagai independente kualitatiboak

Errodadura geruzaren izendatzea (SurfDen)	Errodadura geruza mota (SurfType)	Errodadura geruzaren izendatzea 2(SurfDen2)
AC 16 surf S (1)	Hormigoi bituminosoa (AC) (1)	AC 16 (1)
AC 22 surf S (2)	Hormigoi bituminosoa (AC) (1)	AC 22 (2)
AC 16 surf D (3)	Hormigoi bituminosoa (AC) (1)	AC 16 (1)
AC 22 surf D (4)	Hormigoi bituminosoa (AC) (1)	AC 22 (2)
BBTM 11A (5)	Nahaste etenak eta drainatzaileak (BBTM&PA) (2)	BBTM 11A (3)
BBTM 11B (6)	Nahaste etenak eta drainatzaileak (BBTM&PA) (2)	BBTM 11B (4)
PA 11 (7)	Nahaste etenak eta drainatzaileak (BBTM&PA) (2)	PA (5)
LB2 (8)	Kare-esnea (3)	LB2 (6)

Levene estatistikoa *RoadType* aldagaiaren mailetarako erakusten du mailek ez dauzkate bariantza homogeneoak eta *t* testak adierazten du batez bestekoen diferentzia dagoela (p-balioa<0,001) (9.106 taula).

9.101 taula. SurfDen2 aldagai kualitatiboen esplorazio-analisia mailaz bilduta mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

		SurfDen2					
		AC 15	AC 22	BBTM 11A	BBTM 11B	PA 11	LB2
N		177	19	312	10	79	217
Batez bestekoa		49,164	51,059	46,377	47,796	43,771	56,313
Errore estandarra		0,617	2,179	0,283	1,296	0,500	0,564
%95eko Konfiantza-tarteak	Beheko	47,947	45,820	45,820	44,864	42,776	55,201
	Goiko	50,382	46,930	46,933	50,727	44,766	57,425
Bariantza		67,345	90,228	24,958	16,791	19,732	69,053
Desbideratze tipikoa		8,206	9,499	4,996	4,098	4,442	8,310
Minimoa		33,000	33,944	31,000	41,191	33,333	38,000
Maximoa		73,318	64,625	63,495	54,667	53,750	85,875
Anplitudea		40,318	30,681	32,495	13,476	20,417	47,875
Kuartileko heina		10,401	13,196	6,449	6,633	5,872	10,703
Simetria		0,483	-0,408	0,264	-0,031	-0,013	0,440
Kurtosia		-0,004	-0,703	0,866	-0,410	-0,018	0,237

9.102 taula. Bariantzaren homogeneotasunaren testa SurfDen2-ren mailetarako MSSC-rekiko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
20,551	5	808	< 0,001

9.103 taula. Batez bestekoen berdintasunaren test sendoak SurfDen2-ren mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Testa	Estatistikoa	Askatasun-graduak 1	Askatasun graduak 2	Esangura
Welch	62,438	5	65,784	< 0,001
Brown-Forsythe	65,511	5	117,717	< 0,001

9.104 taula. Bariantzaren analisia (ANOVA) SurfDen2-ren mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan.

Aldagaia (SurfType)	Karratuen batuketa	Askatasun-graduak	Batez besteko karratua	F	Esangura
Mailen artean	15805,269	5	3161,054	67,490	< 0,001
Mailaren barruan	37844,576	808	46,837		
Guztira	53649,846	813			

Marruskadura modelizazioan lortutako esperientziagatik, Erregresio Lineal Orokor Anizkoitzeko modelo batzuk frogatu ziren, hainbat aldagairen (aldagai kuantitatiboak) eta faktoreren (aldagai kualitatiboak, *SurfType*, *SurfDen2* eta *RoadType*) konbinaziorekin.

9.107 taulak modelo bakoitzaren ekuazioa, lortutako R^2 eta doitutako R^2 eta modeloari buruzko ohar batzuk. Taula honek burutako analisi zabalagoaren laburpena da.

Ikusten den bezala, aldagai guztiak esanguratsuak (%95eko konfiantza-mailaz, p-balioa < 0,05) dituen korrelaziorik onenek dauzkate, 1/Ln*AADT*,eta PSV_{req} (faktore bezala); eta *SurfType* (9.8 ekuazioa) edo *SurfDen2* (9.9 ekuazioa), haien determinazio-koefizienteak, R^2, 0,488 eta 0,498 izanik, hurrenez hurren. *SurfDen* faktore bezala erabiltzean *PSV*(f) eta 1/Ln*AADT*-rekin batera, nahiz eta determinazio koefizientea hobetu, *PSV*(f) ez da esanguratsua.

$$MSSC = Intercept + 1/LnAADT + PSV_{req}(f) + SurfType(f) + PSV * SurfType \quad [9.8]$$

$$MSSC = Intercept + 1/LnAADT + PSV_{req}(f) + SurfDen2(f) + PSV * SurfDen2 \quad [9.9]$$

9.105 taula. Konparazio anizkoitza SurfDen2-ren mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Estatistikoa	**SurfType (I)**	**SurfType (J)**	**Batez besteko diferentzia (I-J)**	**Errore estad.**	**Esang.**	**%95eko Konfiantza-tarteak**	
						Beheko	**Goiko**
Tamhane's T2	AC 16	AC 22	-1,895	2,265	1,000	-9,368	5,579
		BBTM 11A	2,788	0,679	0,001	0,782	4,793
		BBTM 11B	1,369	1,435	0,999	-3,716	6,453
		PA 11	5,394	0,794	0,000	3,046	7,741
		LB2	-7,149	0,836	0,000	-9,612	-4,686
	AC 22	AC 16	1,895	2,265	1,000	-5,579	9,368
		BBTM 11A	4,682	2,197	0,512	-2,687	12,051
		BBTM 11B	3,263	2,535	0,970	-4,893	11,420
		PA 11	7,288	2,236	0,057	-0,138	14,715
		LB2	-5,254	2,251	0,365	-12,705	2,196
	BBTM 11A	AC 16	-2,788	0,679	0,001	-4,793	-0,782
		AC 22	-4,682	2,197	0,512	-12,051	2,687
		BBTM 11B	-1,419	1,326	0,996	-6,494	3,656
		PA 11	2,606	0,574	0,000	0,894	4,318
		LB2	-9,937	0,631	0,000	-11,798	-8,075
	BBTM 11B	AC 16	-1,369	1,435	0,999	-6,453	3,716
		AC 22	-3,263	2,535	0,970	-11,420	4,893
		BBTM 11A	1,419	1,326	0,996	-3,656	6,494
		PA 11	4,025	1,389	0,185	-1,040	9,090
		LB2	-8,518	1,413	0,001	-13,589	-3,446
	PA 11	AC 16	-5,394	0,794	0,000	-7,741	-3,046
		AC 22	-7,288	2,236	0,057	-14,715	0,138
		BBTM 11A	-2,606	0,574	0,000	-4,318	-0,894
		BBTM 11B	-4,025	1,389	0,185	-9,090	1,040
		LB2	-12,543	0,754	0,000	-14,770	-10,315
	LB2	AC 16	7,149	0,836	0,000	4,686	9,612
		AC 22	5,254	2,251	0,365	-2,196	12,705
		BBTM 11A	9,937	0,631	0,000	8,075	11,798
		BBTM 11B	8,518	1,413	0,001	3,446	13,589
		PA 11	12,543	0,754	0,000	10,315	14,770

9.106 taula. Levene estatistikoa bariantza homogeneorako eta t testak batez bestekoaren berdintasunerako RoadType-ren mailetarako MSSC-rekiko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa			**t testa batez besteko berdintasunerako**						
F	**Esang..**		**t**	**Askatasun-graduak**	**Esangura (bi aldekoa)**	**Batez besteko diferentzia**	**Errore estandarra**	**%95eko konfiantza-tarteak diferentziarako**	
								Beheko	**Goikoa**
100,53	< 0,001	Bariantza berdinak suposatuta	12,243	812,000	< 0,001	6,501	0,531	5,459	7,543
		Bariantza berdinak ez suposatuta	13,340	776,982	< 0,001	6,501	0,487	5,544	7,458

RoadType aldagaia faktore bezala sartzen denean 9.8 ekuazioan eta 9.9 ekuazioan, determinazio-koefizientea, R^2 eta doitutako R^2 hobeagoak lortzen dira (0,488tik 0,506 9.8 ekuazioan eta 0,498tik 0,513ra 9.9 ekuazioan), baina faktoreen konbinazioak ez dira esanguratsuak.

Faktoreen arteko konbinazioak normalean esangura baxua daukatela ikusita, faktoreen konbinazio guztiak erabiltzen ez dituzten modeloak egiaztatzea erabaki zen. Gainera, faktore eta aldagai kuantitatiboen arteko konbinazioak ere egiaztatutako dira. SPSS v.24 softwareak Erregresio Lineal Orokor Anizkoitzeko modeloak sartzen diren aldagaiak eta faktoreak eta koaldagaiak nahi diren bezala konbinatzen ahalbidetzen du.

Horrez gain, ikus daitekeenez, *RoadType* aldagai kualitatiboak benetan eragina dauka aldagai azalduaren balioetan, eta R^2 altuagoak dituzten modeloetan agertzen da. 8.4.1 atalean azaldu den bezala, *H.AADT* irudikatzen du banandutako galtzada batean dagoen errei kopurua. Hala ere, *AADT* aldagaiak *H.AADT* aldagaiak baino korrelazio hobeagoak erakutsi ditu *MSSC*-rekiko eta ez da berdina autobide edo autobia batean 50.000 ibilgailu/egun-eko Eguneko Batez Besteko Intentsitatea (*AADT*) bi errei edo hiru erreiko galtzada batean. Errei bakoitzetik pasatzen diren batez besteko ibilgailu kopuru totala ez da berdina. Hori dela eta, aldagai kualitatibo berri bat sortzea beharrezkoa dirudi galtzada bakoitzean dauden errei kopurua adierazteko. *Lanes* izena dauka eta bere mailak, 9.108 taulan erakusten dira.

Aldagai berriaren mailak aztertu dira. Levene estatistikoak erakusten du mailek ez dauzkatela bariantza homogeneoak (p-balioa<0,01) (9.109 taula). Ez dago batez bestekoen berdintasunik, test sendoek eta ANOVA analisiak erakusten duten bezala %95eko konfiantza-mailaz (9.110 taula eta 9.111 taula). Gainera, Thamhane's T2 and Dunnett's T3 estatistikoek adierazten dute "bi erreiko errepideak" ez daukala batez bestekoen berdintasunik beste bi taldeekin baina banandutako galtzadaren 2 errei eta 3 erreiren artean berdintasuna dago (9.112 taula).

9.107 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Proposatutako modeloak	R^2	R^2_{adj}	**Komentarioak eta oharrak**
MSSC = Int + LnAADT + SurfType(f)	0,423	0,421	Aldagai guztiak esanguratsuak (p-balioa < 0,001)
MSSC = Int + LnAADT + SurfType(f) + RoadType(f) + SurfType * RoadType	0,439	0,435	Esangura baxua, SurfType*RoadType (p = 0,387)
MSSC = Int + 1/LnAADT +1/LnH.AADT + SurfType(f)	0,462	0,460	Esangura baxua, 1/LnH.AADT (p = 0,81)
MSSC = Int + 1/LnAADT +1/LnH.AADT + SurfType(f) + RoadType(f) + SurfType * RoadType	0,479	0,474	Esangura ertaina, 1/LnH.AADT (p = 0,191) and SurfType * RoadType (p = 0,181
MSSC = Int + 1/LnAADT + SurfType(f)	0,462	0,460	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnAADT + SurfType(f) + RoadType(f) + SurfType * RoadType	0,478	0,474	Esangura ertaina, SurfType*RoadType (p=0,128)
MSSC = Int + 1/LnAADT + PSV(f) + SurfType(f) + PSV*SurfType	0,488	0,481	Aldagai guztiak esanguratsuak (p-balioa < 0,02
MSSC = Int + 1/LnAADT + PSV(f) + SurfType(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,506	0,495	Esangura baxua, PSV daukaten biderkadura (p≈0,12).
MSSC = Int + 1/LnAADT + 1/LnH.AADT + PSV(f) + SurfType(f) + PSV*SurfType	0,489	0,480	Esangura baxua, 1/LnH.AADT (p=0,735)
MSSC = Int + 1/LnH.AADT + PSV(f) + SurfType(f) + PSV*SurfType	0,442	0,434	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnH.AADT + PSV(f) + SurfType(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,454	0,442	Esangura baxua, RoadType (p=0,447) PSV daukaten biderkadura (p >0,138)
MSSC = Int + 1/LnAADT + 1/LnH.AADT + PSV(f) + SurfDen(f) + PSV*SurfDen	0,501	0,486	Esangura baxua, 1/LnH.AADT (p=0,880) PSV (P = 0,282)
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen(f) + PSV*SurfDen	0,502	0,488	Esangura baxua, PSV(f) (0,281)
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,515	0,495	Esangura baxua, PSV (p=0,089) eta biderkadurak (p>0,156).
MSSC = Int + 1/LnAADT + 1/LnH.AADT + PSV + SurfDen2(f)	0,469	0,463	Esangura baxua, 1/LnH.AADT (p=0,826), PSV (p=0,828)
MSSC = Int + 1/LnAADT + 1/LnH.AADT + SurfDen2(f)	0,468	0,464	Esangura baxua, 1/LnH.AADT (p=0,851)
MSSC = Int + 1/LnAADT + SurfDen2(f)	0,468	0,465	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnAADT + SurfDen2(f) + RoadType(f) + SurfDen2*RoadType	0,488	0,480	Esangura baxua, SurfDen2*RoadType (p = 0,158
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen2(f) + PSV*SurfDen2	0,498	0,486	Aldagai guztiak esanguratsuak (p-balioa < 0,03
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,513	0,495	Esangura baxua, biderkadurak (p>0,10)
MSSC = Int + LnAADT + PSV(f) + SurfDen2(f) + PSV*SurfDen2	0,476	0,463	Aldagai guztiak esanguratsuak (p-balioa < 0,01

9.108 taula. Errepide mota eta haien sailkapena RoadType eta Lanes aldagai kualitatiboen arabera

Errepide mota	***RoadType* aldagaiaren mailak**	***Lanes* aldagaiaren mailak**
Galtzada bakarra, errei bat noranzko bakoitzean	1	1
Banandutako galtzada, bi errei galtzadako noranzko bakoitzean	2	2
Banandutako galtzada, hiru edo errei gehiago galtzadako noranzko bakoitzean *	2	3

* BFAko trafikoko datuetan 3 edo 4 erreiko galtzada, berdin adierazita daude datu-basean, beraz, ezin dira berezitu

9.109 taula. Bariantzaren homogeneotasunaren testa Lanes-en maileetarako MSSC-rekiko errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Levene estatistikoa	**Askatasun-graduak 1**	**Askatasun graduak 2**	**Esangura**
58,709	2	793	< 0,001

9.110 taula. Batez bestekoen berdintasunaren test sendoak Lanes-en maileetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Testa	**Estatistikoa**	**Askatasun-graduak 1**	**Askatasun graduak 2**	**Esangura**
Welch	96,963	2	235,290	< 0,001
Brown-Forsythe	140,256	2	621,954	< 0,001

9.111 taula. Bariantzaren analisia (ANOVA) Lanes-en maileetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan

Aldagaia (SurfType)	**Karratuen batuketa**	**Askatasun-graduak**	**Batez besteko karratua**	**F**	**Esangura**
Mailen artean	8438,469	2	4219,234	79,425	< 0,001
Mailaren barruan	42126,131	793	53,122		
Guztira	50564,600	795			

Aldagai kualitatibo berriarekin, *Lanes*, eta faktoreen arteko eta faktore eta aldagai kuantitatiboen arteko konbinazio anizkoitzarekin, analisi osoa gauzatu ziren. Determinazio-koefiziente onenekin eta aldagai gehienak esanguratsuak izanik lortu diren modelo onenak 9.113 – 9.115 tauletan erakusten dira. Hor agertzen diren modeloak frogatu diren modelo guztien laburpena da. 9.113 taulak *SurfTyp*e aldagaia errodadura geruzaren materialaren faktore bezala daukaten modeloak erakusten ditu, 9.114 taulak *SurfDe*n erabiltzen du eta 9.115 taulak *SurfDen2* dauka, *9.112 taula. Konparazio anizkoitza Lanes-en mailetarako mendeko aldagaiarekiko (MSSC) errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan*

Estatistikoa	SurfType (I)	SurfType (J)	Batez besteko diferentzia (I-J)	Errore estad.	Esang.	%95eko Konfiantza-tarteak	
						Beheko	Beheko
Tamhane's T2	1	2	6,742	0,498	< 0,001	5,549	7,935
		3	6,133	0,618	< 0,001	4,644	7,621
	2	1	-6,742	0,498	< 0,001	-7,935	-5,549
		3	-0,610	0,557	0,620	-1,957	0,738
	3	1	-6,133	0,618	< 0,001	-7,621	-4,644
		2	0,610	0,557	0,620	-0,738	1,957
Dunnett's T3	1	2	6,742	0,498	< 0,001	5,549	7,935
		3	6,133	0,618	< 0,001	4,644	7,621
	2	1	-6,742	0,498	< 0,001	-7,935	-5,549
		3	-0,610	0,557	0,618	-1,957	0,737
	3	1	-6,133	0,618	< 0,001	-7,621	-4,644
		2	0,610	0,557	0,618	-0,737	1,957

9.113 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan, faktore eta koaldagaien arteko konbinazio batzuekin, eta SurfType errodadura geruzaren materiala adierazteko

Proposatutako modeloak	R^2	R^2_{adj}	Komentarioak eta oharrak
MSSC = Int + 1/LnAADT + SurfType(f) + RoadType(f)	0,475	0,420	Aldagai guztiak esanguratsuak (p-balioa < 0,001)
MSSC = Int + 1/LnAADT + SurfType(f) + RoadType(f) + RoadType*1/LnAADT	0,505	0,502	Aldagai guztiak esanguratsuak (p-balioa < 0,001)
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f)	0,493	0,490	Aldagai guztiak esanguratsuak (p-balioa < 0,001)
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + SurfType*Lanes	0,496	0,491	Esangura baxua, SurfType*Lanes (p=0,249)
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + 1/LnAADT*Lanes	0,510	0,506	Aldagai guztiak esanguratsuak (p-balioa < 0,01)
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + RoadType(f)+SurfType*RoadType	0,493	0,488	Aldagai guztiak esanguratsuak (p-balioa < 0,02)
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + RoadType(f)+SurfType*RoadType*1/LnAADT	0,516	0,508	Esangura baxua, PSV (p=0,629)
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + RoadType(f) + Lanes(f) + Lanes*1/LnAADT	0,511	0,504	Esangura baxua, PSV (p=0,827)
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + Lanes(f) + Lanes*1/LnAADT + SurfType*PSV	0,519	0,509	Esangura baxua, PSV (p=0,102)
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + Lanes*1/LnAADT + SurfType*PSV	0,519	0,509	Esangura ertaina, PSV*SurfType (p=0,104)
MSSC = Int + LnAADT + SurfType(f) + Lanes(f) + Lanes*LnAADT + SurfType*PSV	0,520	0,510	Esangura ertaina, PSV*SurfType (p=0,121)

9.114 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan, faktore eta koaldagaien arteko konbinazio batzuekin, eta SurfDen errodadura geruzaren materiala adierazteko

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
MSSC = Int + 1/LnAADT + SurfDen(f) + SurfDen*PSV	0,502	0,488	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + SurfDen*PSV	0,509	0,494	Aldagai guztiak esanguratsuak (p-balioa < 0,01
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + SurfDen*PSV + RoadTyoe*1/LnAADT	0,529	0,514	Esangura baxua,PSV*SurfDen (p=0,215)
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + RoadTyoe*1/LnAADT	0,518	0,512	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f)	0,509	0,500	Esangura ertainua,PSV (p=0,107)
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f) + SurfDen*PSV	0,520	0,504	Esangura baxua,PSV (p=0,538)
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f) + SurfDen*PSV + Lanes*1/LnAADT	0,533	0,517	Esangura baxua,PSV (p=0,339), PSV*Surf*Den (p=0,110)
MSSC = Int + 1/LnAADT + SurfDen(f) + Lanes(f) + Lanes*1/LnAADT	0,423	0,516	Aldagai guztiak esanguratsuak (p-balioa < 0,005
MSSC = Int + LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT	0,525	0,518	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT	0,520	0,518	Esangura baxua,1/LnAADT (p=0,214)
MSSC = Int + LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT + PSV(f) + PSV*SurfDen	0,534	0,518	Esangura baxua,PSV (p=0,433) and PSV*Surf*Den (p=0,170)

Mendeko aldagaiarekin korrelazio onenak dituzten aldagaian AADT eta H.AADT eta mendeko aldagaiaren arteko grafikoak ikusten eta Malahanobis eta Cooken distantziak kalkulatuz "*outlier*" posibleak identifikatu dira. Datu horiek aztertu ziren gezurrezko datuak izan daitezkeen. Analisiaren ostean, datu batzuk baztertu ziren. Baztertutako kasuak 9.116 taulan zerrendatzen dira baztertzearen arrazoia adieraziz.

Gezurrezko datuak kontsideratu diren behaketak baztertu eta gero (18), 796 datu daude analisirako. Datu horiekin, 9.113, 9.114 eta 9.115 tauletatako determinazio-koefiziente onenak dituzten modeloak berriro kalkulatu dira 9.117 taulan.

9.115. taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak errepide guztietako errodadura geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan, faktore eta koaldagaien arteko konbinazio batzuekin, eta SurfDen2 errodadura geruzaren materiala adierazteko

Proposatutako modeloak	**R^2**	**R^2_{adj}**	**Komentarioak eta oharrak**
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f)	0,494	0,487	Aldagai guztiak esanguratsuak (p-balioa < 0,03
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f) + PSV*SurfDen2	0,507	0,493	Aldagai guztiak esanguratsuak (p-balioa < 0,06
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f) + RoadType*1/LnAADT	0,515	0,507	Esangura baxua,PSV (p=0,884)
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f) + PSV*SurfDen2 + RoadType*1/LnAADT	0,526	0,513	Esangura baxua,PSV (p=0,264)
MSSC = Int + 1/LnAADT + SurfDen2(f) + RoadType(f) + PSV*SurfDen2 + RoadType*1/LnAADT	0,526	0,513	Esangura baxua,PSV*SurfDen2 (p=0,134)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV(f)	0,506	0,480	Esangura ertaina,PSV (p=0,099)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV(f) + PSV*SurfDen2	0,517	0,503	Esangura baxua,PSV (p=0,277)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2	0,517	0,503	Aldagai guztiak esanguratsuak (p-balioa < 0,03
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV(f) + Lanes*1/LnAADT	0,520	0,511	Esangura baxua,PSV (p=0,937)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2 + Lanes*1/LnAADT	0,530	0,516	Esangura baxua,PSV*SurfDen2 (p=0,209)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2 + Lanes*LnAADT	0,531	0,517	Esangura baxua,PSV*SurfDen2 (p=0,262)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2*PSV*LnAADT	0,556	0,531	Aldagai guztiak esanguratsuak (p-balioa < 0,01
MSSC = Int + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2 + PSV*SurfDen2*PSV*LnAADT	0,572	0,541	Esangura baxua, Konstantea (p=0,301)

9.117 taulan ikus daitekeenez, modeloek hobekuntza erakusten dute determinazio-koefizientea, R^2, nahiz eta kasu batzuetan, aldagai batzuen esangura (edo faktoreekiko konbinazioak) txarragoak izan. Gezurrezko datuak baztertu direnean, marruskadura eskuragarria azaltzeko modelo posibleen analisia jarraitzu zen. 9.118 taulak erakusten du burututako modeloen azterketaren laburpena. Aldagai guztiak estatistikoki esanguratsuak diren modeloak (p-balioa < 0,05) nahiago izan dira.

9.116 taula. Baztertu ziren datuak gezurrezkoak kontsideratzeagatik

Errepidea (noranzkoa)	Hasierako eta amaierako PK-ak	MSSC	AADT	H.AADT	Eraikitze-data	Errodadura	Azalpena
A-8 (desc)	131+0599 - 130+0969	54,3	70821	2498	30/05/2001	LB2	Urte asko trafiko handiarekin. Konpondu behar izan da. Kare-esnea erabiltzen duen A-8 autobideko tartea
BI-604 (asc)	6+0796 - 6+0956	34,5	14853	584	26/02/1995	PA 11	Balio oso baxua eta trafikoa ez oso altua.
N-634 (asc)	97+0679 - 97+0738	33,3	20610	773	31/08/1994	PA 11	Datu gutxi bataz bestekoa kalkulatzeko
BI-631 (asc)	6+0740 - 7+0009	34,3	29350	455	01/06/1997	PA 11	19 urte, konpondu behar izan da
BI-631 (desc)	1+0260 - 0+0941	35,59	42408	552	01/03/2007	PA 11	Balio oso baxua
N-637 desc	28+0140 - 28+0061	53,75	30777	1600	01/06/2007	PA 11	4 datu bataz bestekoa kalkulatzeko
BI-637 (asc)	17+0210 - 18+0369	37,86	35342	389	21/08/2006	PA 11	Datu gutxi bataz bestekoa kalkulatzeko
BI-637	18+0370 - 18+0669	38,06	35342	389	21/08/2006	PA 11	Batez bestekotik oso urrun
N-637 (desc)	10+0660 - 9+0671	53,1	137109	4079	01/06/1997	BBTM 11A	Balio altua trafiko bolumenerako
BI-637 (desc)	9+0157 - 8+0978	50,75	120758	2174	19/09/2013	BBTM 11A	Balio altua trafiko bolumenerako
BI-628 asc	16+0400 - 16+0479	32,50	1300	335	01/11/2005	BBTM 11A	Datu gutxi bataz bestekoa kalkulatzeko
BI-631 (asc)	1+0260 - 1+0439	31,00	42408	552	01/03/2007	BBTM 11A	Balio baxua
A-8 (asc)	108+0203 - 108+0722	61,15	56127	2385	01/06/2014	BBTM 11A	Beharbada ez du 2 urte
A-8 (asc)	1030+0890 - 131+0000	58,73	76821	2498	20/11/1987	AC 22 surf S	29 urte ondoren, kontserbatu behar izan da
BI-638	8+0880 - 8+0899	33	6532	245	01/06/2013	AC 16 surf S	Datu bat bataz bestekoa kalkulatzeko
N-639	21+0300 - 21+0359	36	4392	209	01/06/2012	AC 16 surf S	3 datu bataz bestekoa kalkulatzeko
BI-625	377+0370 - 378+0029	33,95	12510	500	01/06/2013	AC 22 surf S	Balio oso baxua
N-240	55+0188 - 55+0207	33,18	29910	1361	01/06/2009	AC 16 surf S	2 datu bataz bestekoa kalkulatzeko

9.117 taula. 9.113, 9.114 eta 9.115 tauletako Erregresio Lineal Orokor Anizkoitzeko modelo onenen berkalkulatzea 9.116 taulako gezurrezko daturik gabe

Proposatutako modeloak	R^2	R^2_{adj}	Komentarioak eta oharrak
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + 1/LnAADT*Lanes	0,523	0,518	Aldagai guztiak esanguratsuak (p-balioa < 0,004
MSSC = Int + LnAADT + SurfType(f) + Lanes(f) + Lanes*LnAADT + SurfType*PSV	0,528	0,519	Esangura baxua,PSV*SurfType (p=0,735)
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + SurfDen*PSV + RoadTyoe*1/LnAADT	0,529	0,523	Aldagai guztiak esanguratsuak (p-balioa < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f) + SurfDen*PSV + Lanes*1/LnAADT	0,539	0,522	Esangura baxua, PSV (p=0,472), PSV*Surf*Den (p=0,536)
MSSC = Int + LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT + PSV(f) + PSV*SurfDen	0,540	0,524	Esangura baxua, PSV (p=0,589) and PSV*Surf*Den (p=0,529)
MSSC = Int + 1/LnAADT + SurfDen2(f) + RoadType(f) + PSV*SurfDen2 + RoadType*1/LnAADT	0,533	0,520	Esangura baxua,PSV*SurfDen2 (p=0,602)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2 + Lanes*LnAADT	0,537	0,522	Esangura baxua, PSV*SurfDen2 (p=0,729)
MSSC = Int + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2 + PSV*SurfDen2*PSV*LnAADT	0,577	0,546	Esangura baxua, konstantea (p=0,310)

9.118 taula. Proposatutako Erregresio Lineal Orokor Anizkoitzeko modeloak geruza ezaguneko eta 2 urte edo gehiagoko adina errealeko tarteetan, faktore eta koaldagaien arteko konbinazio batzuekin, gezurrezko daturik gabe

Proposatutako modeloak	R^2	R^2_{adj}	Komentarioak eta oharrak
MSSC = Int + LnAADT + SurfType(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2*Lanes*LnAADT	0,555	0,532	Aldagai guztiak esanguratsuak (p-balioa < 0,03)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2	0,537	0,522	Esangura baxua, PSV*SurfDen2 (p=0,729)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes*LnAADT + PSV*SurfDen2*LnAADT	0,542	0,528	Esangura baxua, PSV*SurfDen2*LnAADT (p=0,117)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2*LnAADT	0,546	0,528	Esangura baxua, PSV*SurfDen2*LnAADT (p=0,139)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfType*LnAADT	0,542	0,430	Aldagai guztiak esanguratsuak (p-balioa < 0,05)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfType*LnAADT*Lanes	0,554	0,537	Aldagai guztiak esanguratsuak (p-balioa < 0,01)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + SurfType*LnAADT*Lanes	0,536	0,530	Aldagai guztiak esanguratsuak (p-balioa < 0,01)
MSSC = Int + LnAADT + SurfDen2(f) + PSV(f) + Lanes*LnAADT	0,503	0,495	Aldagai guztiak esanguratsuak (p-balioa < 0,01)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + PSV(f) + Lanes*LnAADT	0,531	0,522	Esangura baxua, PSV (p=0,881)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT	0,530	0,524	Aldagai guztiak esanguratsuak (p-balioa < 0,01)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + SurfDen2*PSV*LnAADT	0,540	0,528	Esangura baxua, SurfDen2*PSV*LnAADT (p=0,139)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + SurfDen2*PSV*LnAADT*Lanes	0,560	0,534	Aldagai guztiak esanguratsuak (p-balioa < 0,04)

9.118 taulako eta 9.113 – 9.115 tauletako emaitzak aztertuz, ondorio batzuk atera daitezke:

- Modelo onenak sortzen dituzten aldagai kuantitatiboak Ln*AADT* eta 1/Ln*AAD*T dira eta aldagai kualitatiboen artean, *PSV*, *RoadType*, *Lanes*, *SurfType* eta *SurfDen2*. *SurfDen* aldagai kualitatiboak, eragina handiagoa zeukan faktorea izango litzatekeela suposatzen zena, errodadura geruzako materialen aukera guztiak determinatzen dituenez, ez da korrelazio onena lortzen duena. Arrazoia da kasu batzuetan kasu gutxi dituela, hala nola AC 22 surf D (datu bat) eta AC 16 surf D (3 datu). Horren ondorioz, logikoa dirudi errodadura materialak batzea antzeko propietate eta ezaguarrien arabera, *SurfType* eta *SurfDen2* aldagaietan bezala.

- *H.AADT* aldagaiak *AADT* aldagaiak (edo haien eraldaketak) baino modelo txarragoak sortzen ditu. Bi aldagaiak batera sartzen badira modeloan, normalean H.AADT ez da esanguratsua.

- Faktoreen konbinazioak sartzea modelo hobeagoak sortzen ditu, faktoreak ez konbinatzekoarekin konparatuz

- PSV_{req} faktore bakarra bezala erabiltzen bada normalean ez da esanguratsua.
- *Lanes* aldagai kualitatiboak *RoadType* faktoreak baino modelo hobeagoak sortzen ditu.
- 1/Ln*AADT*Lanes* edo Ln*AADT*Lanes* konbinazioek beti emaitza hobetzen du eta 1/Ln*AADT*RoadType* edo Ln*AADT*RoadType* konbinazioek baino balio hobeago ematen dute.. Beraz, Lanes aldagaiaren garrantzia azpimarratzen da, 3 edo 4 erreien presentzia deskribatzen duena galtzada banandutako errepide batean. *Roadtype* baino faktore hobeagoa da.
- Nahiz eta Ln*AADT* edo1/Ln*AADT* modeloan sartu aldagai kuantitatibo bezala eta 1/Ln*AADT*Lanes* edo Ln*AADT*Lanes* konbinazioak sartu, *Lanes* aldagaia ere berriro sartzen bada faktore bezala, modeloa hobetzen da eta beti esanguratsua da.
- 1/Ln*AADT* erabiltzen bada aldagai kuantitatibo bezala eta *Lanes**Ln*AADT* konbinazioa sartzen bada (edo kontrako aukera: Ln*AADT* and Lanes*1/Ln*AADT*) aldagai kuantitatiboa ez da esanguratsua. Beraz, Ln*AADT* edo 1/Ln*AADT* erabili behar da bi aldagaietan ez esanguratsu ez bihurtzeko.
- PSV_{req} faktore bezala eta beste faktore batekin konbinaketak, *SurfType* edo *SurfDen2*-rekin, konbinazioa ez esanguratsu bihurtzen da. *Lanes* ere erabiltzen bada konbinazio horretan, esangura hobetzen da baina p-balioa ez da 0,05 baino txikiagoa. 3 faktoreak Ln*AADT*-rekin konbinatzen bada, modeloa hobetzen da.

Ondorio hauekin eta aztertutako modeloen zerrenda luzearen R^2 ikusita, 2 modelo proposatzea erabaki zen, Bizkaiko errepide-sarean marruskadura eskuragarria hoberen irudikatzeko modelo onenak bezala:

- Modelo labur bat, aldagai kuantitatibo batekin, faktore indibidualak eta bi aldagaien konbinazio bat, koefizienteen zerrenda luzea aplikatzeko ez izateko, 9.10 ekuazioaren itxurakoa. Lortutako a R^2 0,530 da eta doitutako R^2 = 0,524 eta aldagai guztiak esanguratsuak dira (p-balioa<0,001)

$$MSSC = Intercept + LnAADT + SurfDen2(f) + Lanes(f) + Lanes(f) * LnAADT \qquad [9.10]$$

- Modelo luze bat, 9.10 ekuazioan erabilitako aldagaiak eta 3 faktore eta aldagai kuantitatibo baten konbinazioa emaitza hobeagoa lortzeko, R^2 = 0,560 eta doitutako R^2 = 0,534, eta aldagai guztiak esanguratsuak %95eko konfiantza-mailarekin (p-balioa<0,04).

$$MSSC = Intercept + LnAADT + SurfDen2(f) + Lanes(f) + Lanes(f) * LnAADT$$
$$+ SurfDen2(f) * PSV(f) + Lanes(f) * LnAADT \qquad [9.11]$$

Nahiz eta modelo labur hobeago bat eta modelo luze hobeagoa bat lortzea indibidualki posiblea izan, nahiago zen aldagai berak erabiltzea bi ekuazioetan. Modelo luzeak modelo laburrak baino aldagai bat gehiago izatea nahiago zen. Proposatutako bi ekuazioen artean 9.11 ekuazioaren azken osagaia da, hiru faktore eta aldagai kuantitatibo baten konbinazioa dena.

9.119 taulak erakusten du 9.10 ekuazioaren modeloaren Subjektu-arteko Efektuen testa, non ikus daitekeen aldagai guztiak esanguratsuak direla (p-balioa<0,001). 9.120 taulak aurkezten ditu 9.10 ekuazioaren modeloaren parametroen estimazioak (koefizienteak). Koefiziente-kontrasteren *L* matrizeak, efektu bakoitzera erlazionatutako koefizienteak (modeloan dagoen hipotesia definitzen duten koefizienteak) lortzen ahalbidetzen dituzte, 9.5 irudian eta 9.121 taula eta 9.122 tauletan erakusten dira. 9.123 eta 9.124 taulak erakusten dituzte mendeko aldagaiaren batez besteko marjinalak faktore maila bakoitzarekiko.

9.119 taula. 9.10 ekuazioaren modeloaren Subjektu-arteko Efektuen testa, proposatutako modelo laburra

Jatorria	Karratuen batuketa III mota	Askatasun-graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	26815,861	10	2681,586	88,638	< 0,001	0,53	886,382	1,000
Konstantea	3333,343	1	3333,343	110,182	< 0,001	0,123	110,182	1,000
LnAADT	631,574	1	631,574	20,876	< 0,001	0,026	20,876	0,995
Lanes	1950,421	2	975,211	32,235	< 0,001	0,076	64,47	1,000
Lanes*LnAADT	2315,593	2	1157,797	38,27	< 0,001	0,089	76,541	1,000
SurfDen2	5284,665	5	1056,933	34,936	< 0,001	0,182	174,681	1,000
Errorea	23748,739	785	30,253					
Totala	2015919,12	796						
Totala zuzendua	50564,6	795						

9.120 taula. 9.10 ekuazioaren modeloaren parametroen estimazioak, proposatutako modelo laburra

Parametroak	B	Errore estandarra	t	Esang	%95 konfiantza-tartea		Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
					Beheko	Goikoa			
Konstantea	78,497	21,451	3,659	< 0,001	36,389	120,606	0,017	3,659	0,955
LnAADT	-2,321	1,894	-1,226	0,221	-6,038	1,396	0,002	1,226	0,232
[Lanes=1]	26,693	21,607	1,235	0,217	-15,722	69,107	0,002	1,235	0,235
[Lanes=2]	-16,663	22,030	-,756	0,450	-59,907	26,581	0,001	0,756	0,118
[Lanes=3]	0[a]	.	.	.	.	.	.	.	.
[Lanes=1] * LnAADT	-3,352	1,915	-1,750	0,081	-7,112	,408	0,004	1,750	0,416
[Lanes=2] * LnAADT	1,328	1,955	,679	0,497	-2,510	5,166	0,001	0,679	0,104
[Lanes=3] * LnAADT	0[a]	.	.	.	.	.	.	.	.
[SurfDen2=1]	-6,989	0,562	-12,433	< 0,001	-8,093	-5,886	0,165	12,433	1,000
[SurfDen2=2]	-5,047	1,391	-3,628	< 0,001	-7,778	-2,316	0,016	3,628	0,952
[SurfDen2=3]	-5,468	0,680	-8,046	< 0,001	-6,802	-4,134	0,076	8,046	1,000
[SurfDen2=4]	-4,213	1,857	-2,269	0,024	-7,859	-,568	0,007	2,269	0,620
[SurfDen2=5]	-7,467	0,959	-7,789	< 0,001	-9,349	-5,586	0,072	7,789	1,000
[SurfDen2=6]	0[a]	.	.	.	.	.	.	.	.

[a] Zeroa da parametro hau erredundantea delako

Intercept		LnAADT		Lanes			Lanes*LnAADT			SurfDen2					
Parameter	L1	Parameter	L2	Parameter	L3	L4	Parameter	L6	L7	Parameter	L9	L10	L11	L12	L13
Intercept	1	Intercept	0	Intercept	0	0	Intercept	0	0	Intercept	0	0	0	0	0
LnAADT	0	LnAADT	1	LnAADT	0	0	LnAADT	0	0	LnAADT	0	0	0	0	0
[Lanes=1]	0,333	[Lanes=1]	0	[Lanes=1]	1	0	[Lanes=1]	0	0	[Lanes=1]	0	0	0	0	0
[Lanes=2]	0,333	[Lanes=2]	0	[Lanes=2]	0	1	[Lanes=2]	0	0	[Lanes=2]	0	0	0	0	0
[Lanes=3]	0,333	[Lanes=3]	0	[Lanes=3]	-1	-1	[Lanes=3]	0	0	[Lanes=3]	0	0	0	0	0
[Lanes=1]*LnAADT	0	[Lanes=1]*LnAADT	0,333	[Lanes=1]*LnAADT	0	0	[Lanes=1]*LnAADT	1	0	[Lanes=1]*LnAADT	0	0	0	0	0
[Lanes=2]*LnAADT	0	[Lanes=2]*LnAADT	0,333	[Lanes=2]*LnAADT	0	0	[Lanes=2]*LnAADT	0	1	[Lanes=2]*LnAADT	0	0	0	0	0
[Lanes=3]*LnAADT	0	[Lanes=3]*LnAADT	0,333	[Lanes=3]*LnAADT	0	0	[Lanes=3]*LnAADT	-1	-1	[Lanes=3]*LnAADT	0	0	0	0	0
[SurfDen2=1]	0,167	[SurfDen2=1]	0	[SurfDen2=1]	0	0	[SurfDen2=1]	0	0	[SurfDen2=1]	1	0	0	0	0
[SurfDen2=2]	0,167	[SurfDen2=2]	0	[SurfDen2=2]	0	0	[SurfDen2=2]	0	0	[SurfDen2=2]	0	1	0	0	0
[SurfDen2=3]	0,167	[SurfDen2=3]	0	[SurfDen2=3]	0	0	[SurfDen2=3]	0	0	[SurfDen2=3]	0	0	1	0	0
[SurfDen2=4]	0,167	[SurfDen2=4]	0	[SurfDen2=4]	0	0	[SurfDen2=4]	0	0	[SurfDen2=4]	0	0	0	1	0
[SurfDen2=5]	0,167	[SurfDen2=5]	0	[SurfDen2=5]	0	0	[SurfDen2=5]	0	0	[SurfDen2=5]	0	0	0	0	1
[SurfDen2=6]	0,167	[SurfDen2=6]	0	[SurfDen2=6]	0	0	[SurfDen2=6]	0	0	[SurfDen2=6]	-1	-1	-1	-1	-1

9.5 irudia.9.10 ekuazioaren modeloaren L matrizea, efektu bakoitzerako erlazionatutako koefizienteak lortzeko.

9.121 taula. L`kontraste-matrizea, efektu bakoitzerako erlazionatutako koefizienteak lortzen ahalbidetzen duena

Parametroa	**Lanes**		
	Bi errei	**Banandutako galtzada 2 erreirekin**	**Banandutako galtzada 3 erreirekin**
Konstantea	1	1	1
LnAADT	9,477	9,477	9,477
[Lanes=1]	1	0	0
[Lanes=2]	0	1	0
[Lanes=3]	0	0	1
[Lanes=1] * LnAADT	9,477	0	0
[Lanes=2] * LnAADT	0	9,477	0
[Lanes=3] * LnAADT	0	0	9,477
[SurfDen2=1]	0,167	0,167	0,167
[SurfDen2=2]	0,167	0,167	0,167
[SurfDen2=3]	0,167	0,167	0,167
[SurfDen2=4]	0,167	0,167	0,167
[SurfDen2=5]	0,167	0,167	0,167
[SurfDen2=6]	0,167	0,167	0,167

9.122 taula. L`kontraste-matrizea, efektu bakoitzerako erlazionatutako koefizienteak lortzen ahalbidetzen duena

Parametroa	SurfDen2					
	AC 16	AC 22	BBTM 11A	BBTM 11B	PA 11	LB2
Konstantea	1	1	1	1	1	1
LnAADT	9,477	9,477	9,477	9,477	9,477	9,477
[Lanes=1]	0,333	0,333	0,333	0,333	0,333	0,333
[Lanes=2]	0,333	0,333	0,333	0,333	0,333	0,333
[Lanes=3]	0,333	0,333	0,333	0,333	0,333	0,333
[Lanes=1] * LnAADT	3,159	3,159	3,159	3,159	3,159	3,159
[Lanes=2] * LnAADT	3,159	3,159	3,159	3,159	3,159	3,159
[Lanes=3] * LnAADT	3,159	3,159	3,159	3,159	3,159	3,159
[SurfDen2=1]	1	0	0	0	0	0
[SurfDen2=2]	0	1	0	0	0	0
[SurfDen2=3]	0	0	1	0	0	0
[SurfDen2=4]	0	0	0	1	0	0
[SurfDen2=5]	0	0	0	0	1	0
[SurfDen2=6]	0	0	0	0	0	1

9.123 taula. MSSC mendeko aldagaiaren estimatutako batez besteko marjinalak Lanes faktorearen mailetarako

Mendeko aldagaia: MSSC				
Lanes	**Batez bestekoa**	**Errore estandarra**	**%95eko Konfiantza-tarteak**	
			Beheko muga	**Goiko muga**
Bi errei	46,565[a]	0,518	45,548	47,582
Banandutako galtzada 2 erreirekin	47,565[a]	0,660	46,269	48,861
Banandutako galtzada 3 erreirekin	51,639[a]	3,549	44,672	58,607

a. Modeloan agertzen diren aldagai kuantitatiboak honako balio hauetan ebaluatzen dira: LnAADT = 9,4774.

9.124 taula. MSSC mendeko aldagaiaren estimatutako batez besteko marjinalak SurfDen2 faktorearen mailetarako

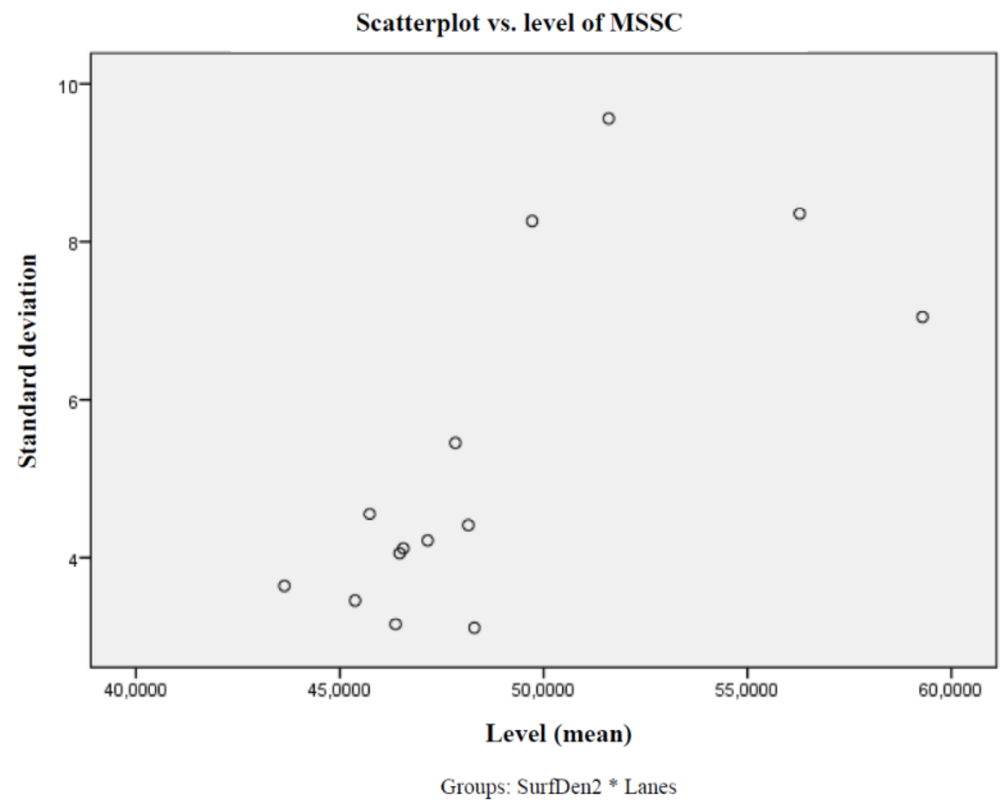
Mendeko aldagaia: MSSC				
SurfDen2	**Batez bestekoa**	**Errore estandarra**	**%95eko Konfiantza-tarteak**	
			Beheko muga	**Goiko muga**
AC 16	46,465[a]	1,314	43,884	49,045
AC 22	48,407[a]	1,790	44,894	51,920
BBTM 11A	47,986[a]	1,238	45,555	50,416
BBTM 11B	49,241[a]	2,107	45,104	53,377
PA 11	45,987[a]	1,254	43,525	48,448
LB2	53,454[a]	1,311	50,881	56,027

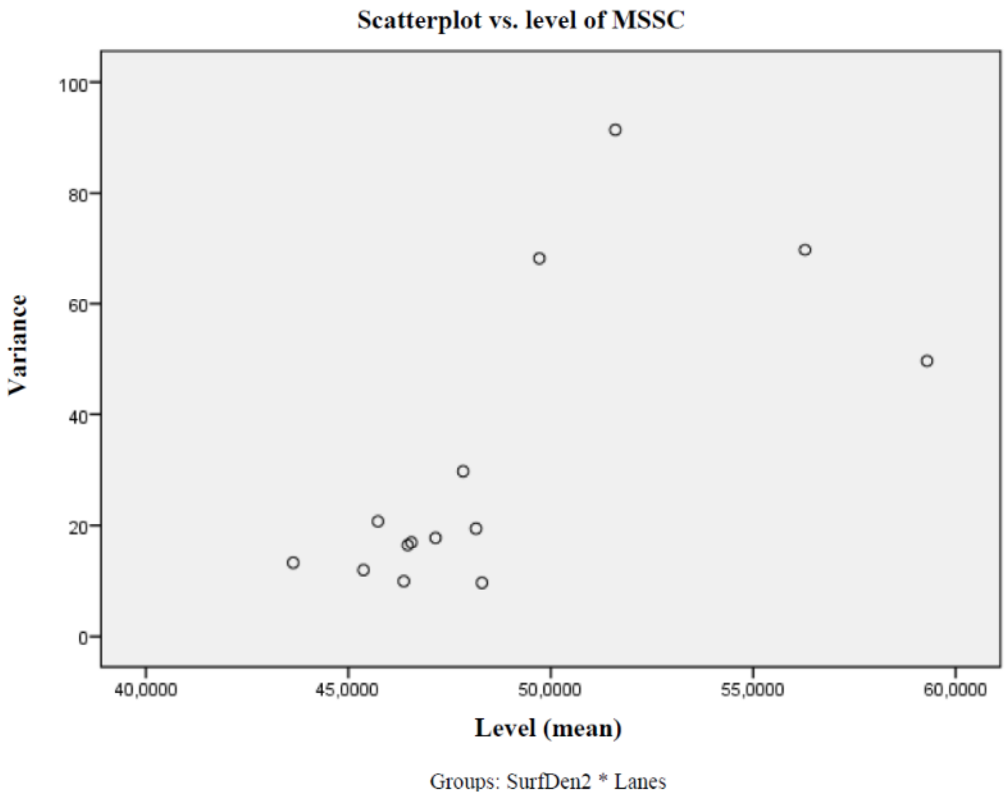
a. Modeloan agertzen diren aldagai kuantitatiboak honako balio hauetan ebaluatzen dira: LnAADT = 9,4774.

9.6 irudiak aurkezten ditu dispertsio-diagrama maila bakoitzerako eta bariantzaren homogeneotasunari buruzko informazioa grafikoa ematen du eta batez bestekoen tamaina eta bariantzaren arteko erlazio posibleak aurkitzen ahalbidetzen du. Bariantzak berdinak ez direnez, Levene testek egiaztatu zutenez, 9.6a eta 9.6b irudietako puntuak ez daude horizontalki alienatuta.

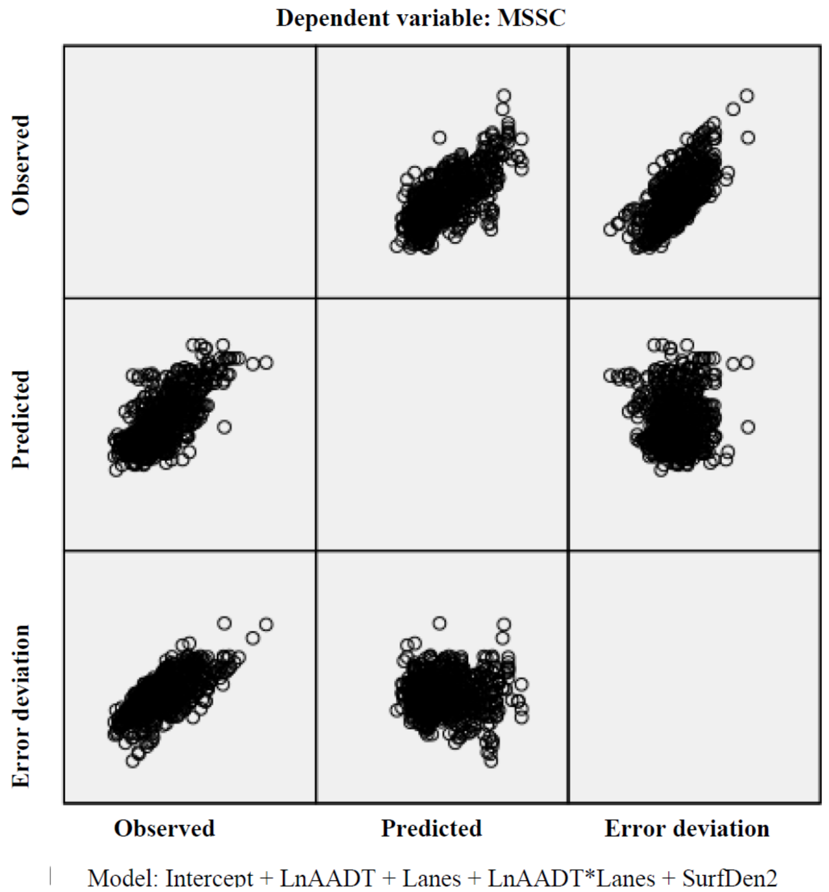
9.7 irudiaren erroreen grafikoaren bidez ikusten da erroreak ausazkoak dira eta independenteak direla beraien artean. Iragarritako balioen eta errore estandarizatuen arteko grafikoa ausazkoa denez (ez dago patroirik), erroreak independenteak dira. Erroreen bariantzak homogeneoak dira errore estandarizatuen dispertsioa antzeko da iragarritako balio guztien zehar. Neurtutako eta iragarritako balioek patroi lineala erakusten dute, determinazio-koefizientearen bidez irudikatzen dena.

Efektuen profilaren grafikoa (9.8 irudia) erakusten du lineak ez direla zeharkatzen, beraien arteko iterazioa esanguratsua dela adieraziz. Puntu bakoitzak mendeko aldagaiaren balioa adierazten du azpitalde bakoitzean *Lanes* eta *SurfDen2* mailak konbinatu eta gero. Neurtutako eta iragarritako balioen arteko korrelazioa detailez ikusteko 9.9 irudia erakusten da.

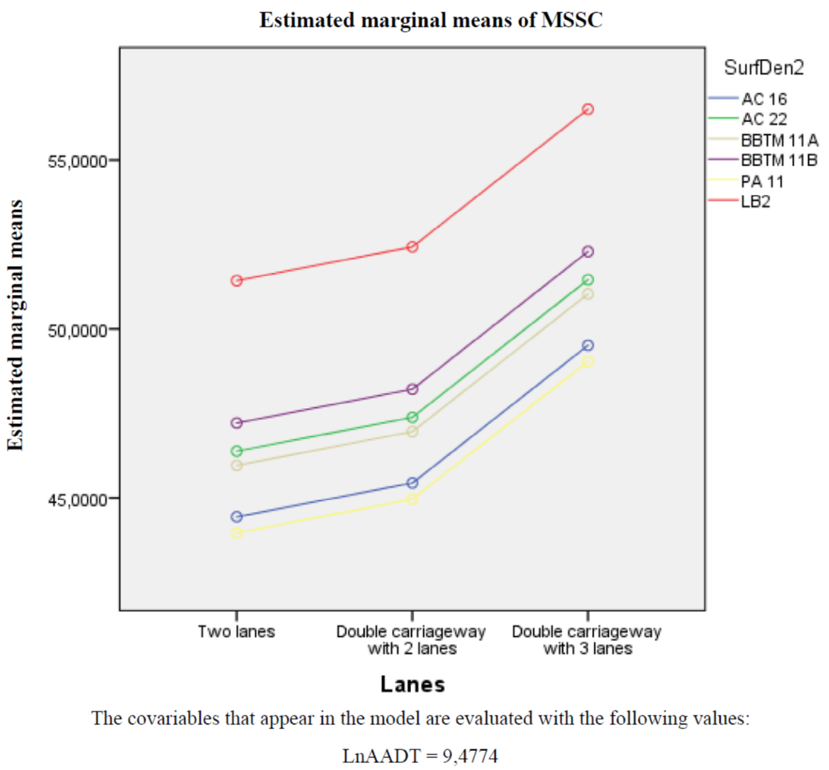




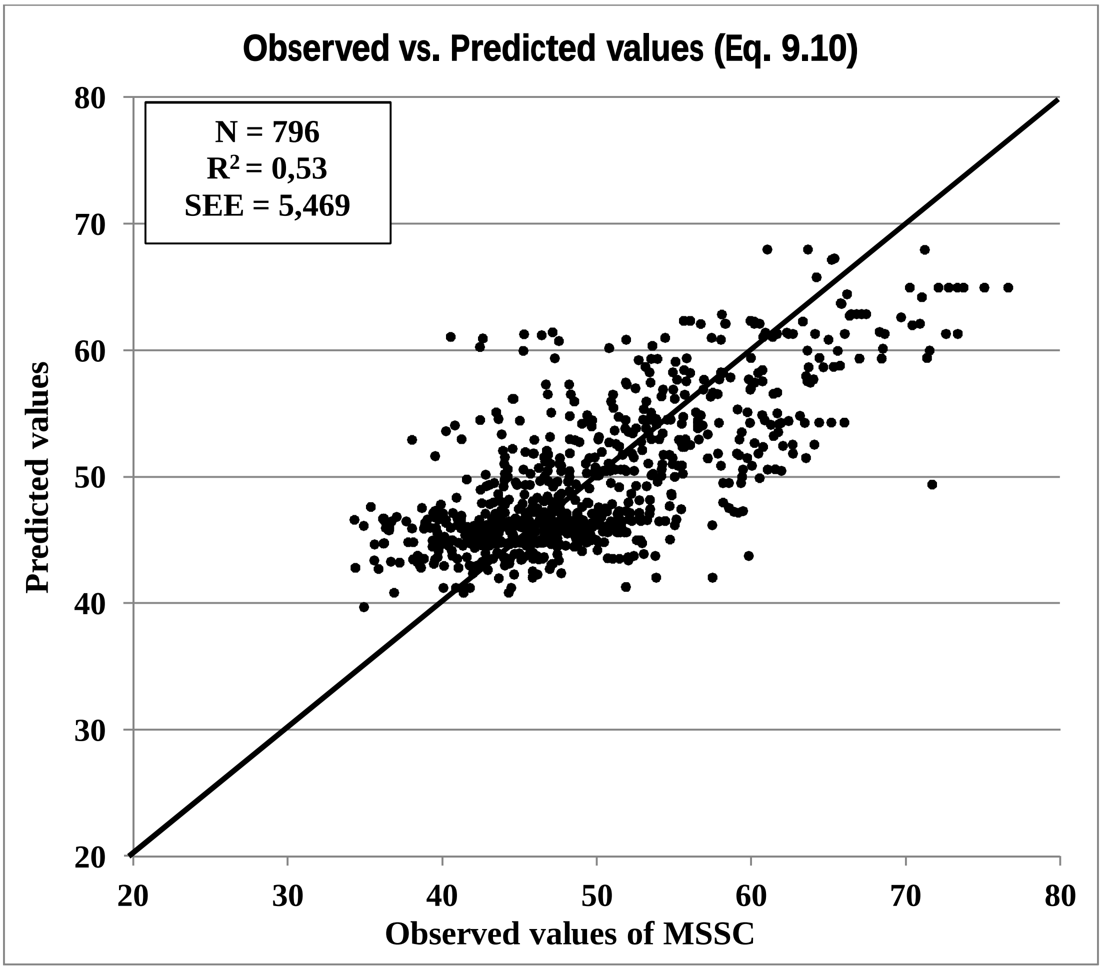
9.6 irudia. 9.10 ekuazioaren dispertsio-diagramak mailarekik. a) Desbideratze tipikoa, b) Bariantza.



9.7 irudia. Estandarizatutako erroreen, neurtutako balioak eta iragarritako balioak 9.10 ekuazioaren bidez



9.8 irudia. Lanes eta SurfDen2 faktoreen efektuen profilen grafikoa 9.10 ekuazioaren modeloarekin



9.9 irudia. Neurtutako eta 9.10 ekuazioarekin iragarritako balioen grafikoa.

9.11 ekuazioan proposatutako modeloari dagokionez, modeloaren Subjektu-arteko Efektuen testa 9.125 taulan erakusten da. Ikusten denez, faktore eta aldagai guztiak %95 baino esangura handiagoa daukate (p-balioak < 0,05). Modelo luzearen (9.11) parametroen estimazioak (koefizienteak) 9.126 taulan erakusten dira.

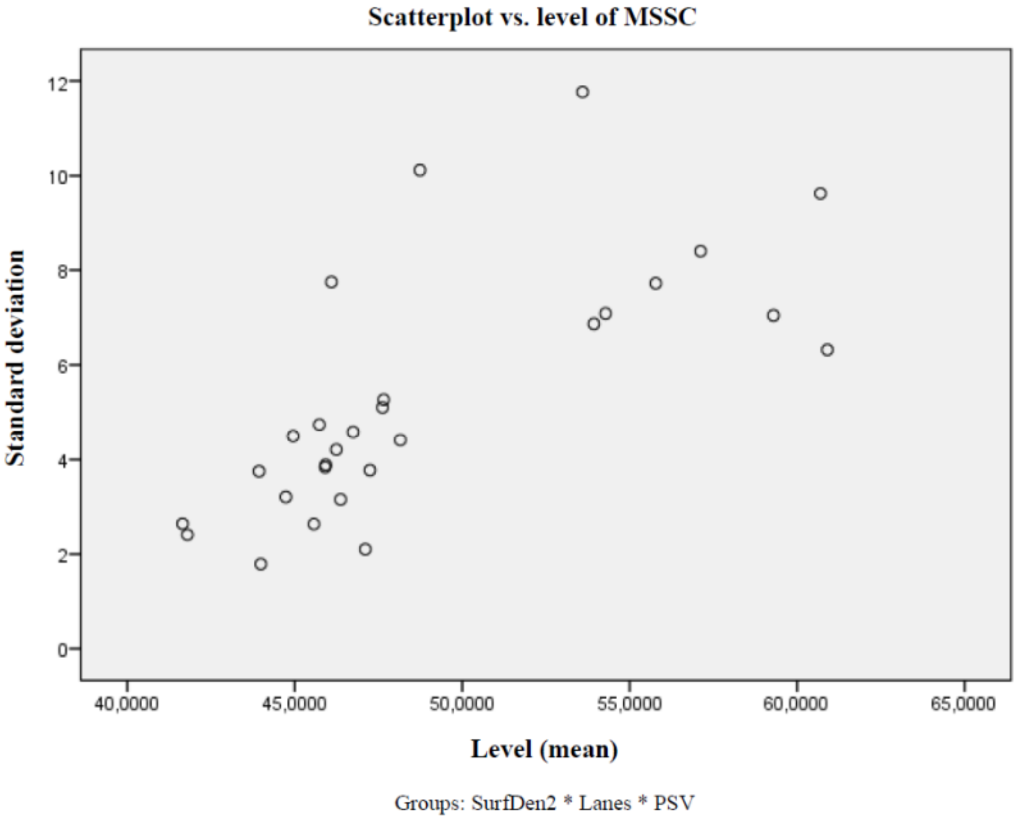
9.125 taula. 9.11 ekuazioaren modeloaren Subjektu-arteko Efektuen testa, proposatutako modelo luzea

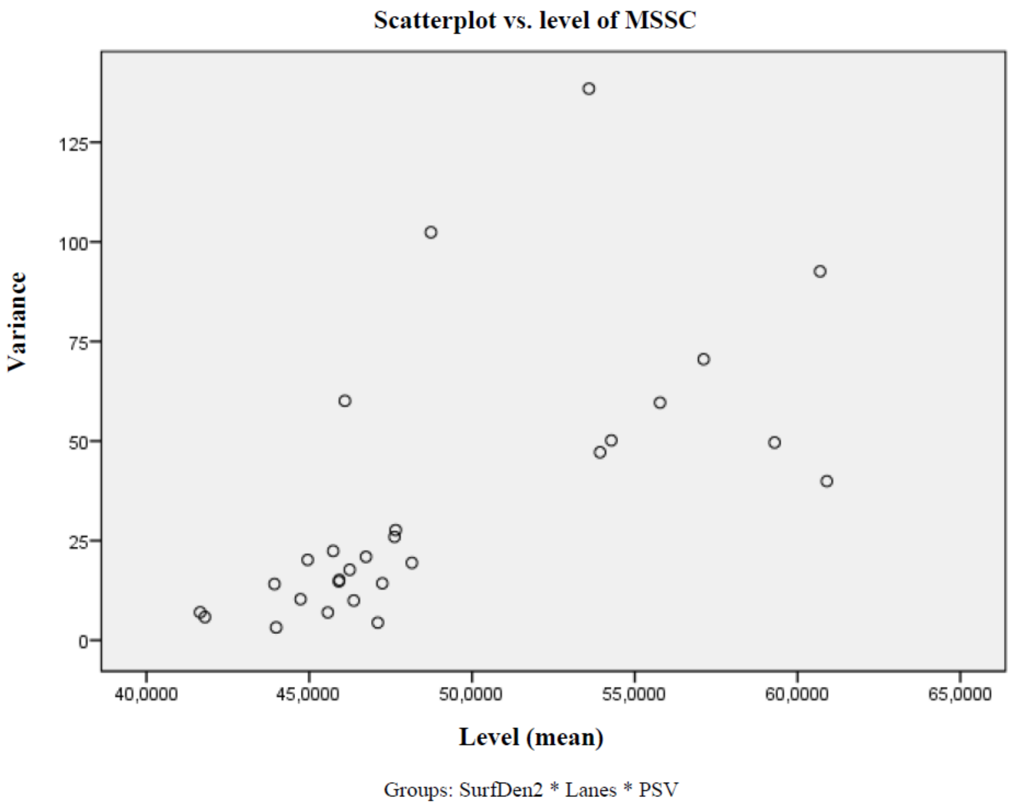
Jatorria	Karratuen batuketa III mota	Askatasun-graduak	Batez besteko karratua	F	Esang.	Eta karratu partziala	Ez zentralitatearen parametroa	Ikusitako botera
Modelo zuzendua	28298,071[a]	43	658,095	22,226	< 0,001	0,560	955,701	1,000
Konstantea	1711,608	1	1711,608	57,806	< 0,001	0,071	57,806	1,000
LnAADT	652,245	1	652,245	22,028	< 0,001	0,028	22,028	0,997
Lanes	263,004	2	131,502	4,441	0,012	0,012	8,882	0,763
Lanes*LnAADT	870,993	2	435,496	14,708	< 0,001	0,038	29,416	0,999
SurfDen2	899,968	5	179,994	6,079	< 0,001	0,039	30,394	0,996
PSV * SurDen2 * Lanes * LnAADT	1482,21	33	44,915	1,517	0,033	0,062	50,058	0,992
Errorea	22266,529	752	29,610					
Totala	2015919,12	796						
Totala zuzendua	50564,6	795						

9.126. taula. 9.11 ekuazioaren modeloaren parametroen estimazioak, proposatutako modelo luzea

Parametroak	B	Errore estandarra	t	Esang	%95 konfiantza-tartea		Ikusitako botera
					Behekoa	Goikoa	
Konstantea	114,946	26,275	4,375	0,000	63,364	166,527	0,992
LnAADT	-2,065	2,420	-0,853	0,394	-6,817	2,686	0,136
[Lanes=1]	9,473	25,685	0,369	0,712	-40,949	59,896	0,066
[Lanes=2]	-19,756	24,738	-0,799	0,425	-68,319	28,807	0,125
[Lanes=3]	0[a]	.	.	.	.	.	.
[Lanes=1] * LnAADT	-5,719	2,497	-2,291	0,022	-10,620	-0,818	0,629
[Lanes=3] * LnAADT	1,817	2,227	0,816	0,415	-2,554	6,189	0,129
[Lanes=3] * LnAADT	0[a]	.	.	.	.	.	.
[SurfDen2=1]	-34,317	7,609	-4,510	0,000	-49,255	-19,379	0,995
[SurfDen2=2]	63,566	37,759	1,683	0,093	-10,560	137,692	0,390
[SurfDen2=3]	-40,151	10,373	-3,871	0,000	-60,515	-19,787	0,972
[SurfDen2=4]	-0,722	50,872	-0,014	0,989	-100,590	99,147	0,050
[SurfDen2=5]	-47,639	18,074	-2,636	0,009	-83,121	-12,157	0,749
[SurfDen2=6]	0[a]	.	.	.	.	.	.
[PSV=40]*[SurfDen2=1]*[Lanes=1]* LnAADT	3,243	0,923	3,515	0,000	1,432	5,054	0,939
[PSV=40]*[SurfDen2=2]*[Lanes=1]*LnAADT	-7,108	3,993	-1,780	0,075	-14,946	0,731	0,428
[PSV=40]*[SurfDen2=2]*[Lanes=2]*LnAADT	-10,560	4,398	-2,401	0,017	-19,194	-1,926	0,669
[PSV=40]*[SurfDen2=6]*[Lanes=1]*LnAADT	-1,060	0,842	-1,258	0,209	-2,713	0,594	0,242
[PSV=44]*[SurfDen2=1]*[Lanes=1]*LnAADT	3,063	0,939	3,262	0,001	1,220	4,906	0,903
[PSV=44]*[SurfDen2=2]*[Lanes=1]*LnAADT	-10,389	5,242	-1,982	0,048	-20,679	-0,099	0,508
[PSV=44]*[SurfDen2=6]*[Lanes=1]*LnAADT	-0,374	0,238	-1,570	0,117	-,842	0,094	0,348
[PSV=45]*[SurfDen2=1]*[Lanes=1]*LnAADT	3,115	0,844	3,691	0,000	1,459	4,772	0,958
[PSV=45]*[SurfDen2=1]*[Lanes=2]*LnAADT	-1,595	1,684	-0,947	0,344	-4,900	1,710	0,157
[PSV=45]*[SurfDen2=1]*[Lanes=3]*LnAADT	-1,090	1,627	-0,670	0,503	-4,284	2,105	0,103
[PSV=45]*[SurfDen2=2]*[Lanes=1]*LnAADT	-7,302	4,254	-1,716	0,087	-15,654	1,050	0,403
[PSV=45]*[SurfDen2=2]*[Lanes=3]*LnAADT	-9,791	3,716	-2,635	0,009	-17,086	-2,496	0,749
[PSV=45]*[SurfDen2=3]*[Lanes=1]*LnAADT	4,993	1,401	3,565	0,000	2,243	7,742	0,945
[PSV=45]*[SurfDen2=5]*[Lanes=1]*LnAADT	5,007	2,237	2,239	0,025	,617	9,398	0,609
[PSV=45]*[SurfDen2=5]*[Lanes=2]*LnAADT	-0,328	0,423	-0,776	0,438	-1,159	0,503	0,121
[PSV=45]*[SurfDen2=6]*[Lanes=1]*LnAADT	-0,435	0,143	-3,038	0,002	-0,716	-0,154	0,859
[PSV=50]*[SurfDen2=1]*[Lanes=1]*LnAADT	2,903	0,842	3,446	0,001	1,249	4,557	0,931
[PSV=50]*[SurfDen2=1]*[Lanes=2]*LnAADT	-1,130	1,666	-0,678	0,498	-4,401	2,141	0,104
[PSV=50]*[SurfDen2=2]*[Lanes=1]*LnAADT	-8,084	4,293	-1,883	0,060	-16,511	0,343	0,468
[PSV=50]*[SurfDen2=2]*[Lanes=3]*LnAADT	-9,397	3,716	-2,529	0,012	-16,692	-2,102	0,714
[PSV=50]*[SurfDen2=3]*[Lanes=1]*LnAADT	3,834	1,125	3,406	0,001	1,624	6,043	0,925
[PSV=50]*[SurfDen2=3]*[Lanes=2]*LnAADT	-0,663	1,424	-0,465	0,642	-3,459	2,133	0,075
[PSV=50]*[SurfDen2=3]*[Lanes=3]*LnAADT	-0,392	1,384	-0,283	0,777	-3,109	2,326	0,059
[PSV=50]*[SurfDen2=4]*[Lanes=1]*LnAADT	-0,468	5,442	-0,086	0,931	-11,151	10,215	0,051
[PSV=50]*[SurfDen2=4]*[Lanes=2]*LnAADT	-4,427	5,199	-0,851	0,395	-14,633	5,780	0,136
[PSV=50]*[SurfDen2=5]*[Lanes=1]*LnAADT	4,482	1,939	2,311	0,021	0,675	8,289	0,636
[PSV=50]*[SurfDen2=5]*[Lanes=2]*LnAADT	-0,091	0,357	-0,254	0,799	-0,792	0,610	0,057
[PSV=50]*[SurfDen2=5]*[Lanes=3]*LnAADT	0,112	0,264	0,423	0,672	-0,407	0,630	,071
[PSV=50]*[SurfDen2=6]*[Lanes=1]*LnAADT	0[a]	.	.	.	.	.	.
[PSV=50]*[SurfDen2=6]*[Lanes=2]* LnAADT	-3,604	1,801	-2,001	0,046	-7,140	-0,068	0,515
[PSV=55]*[SurfDen2=3]*[Lanes=2]*LnAADT	-0,626	1,408	-0,445	0,657	-3,390	2,138	0,073
[PSV=55]*[SurfDen2=3]*[Lanes=3]*LnAADT	-0,320	1,390	-0,230	0,818	-3,049	2,410	0,056
[PSV=56]*[SurfDen2=3]*[Lanes=2]*LnAADT	-0,570	1,408	-0,405	0,686	-3,334	2,194	0,069
[PSV=56]*[SurfDen2=3]*[Lanes=3]*LnAADT	-0,456	1,385	-0,329	0,742	-3,175	2,264	0,062
[PSV=56]*[SurfDen2=5]*[Lanes=2]*LnAADT	0[a]	.	.	.	.	.	.
[PSV=56]*[SurfDen2=5]*[Lanes=3]*LnAADT	0[a]	.	.	.	.	.	.

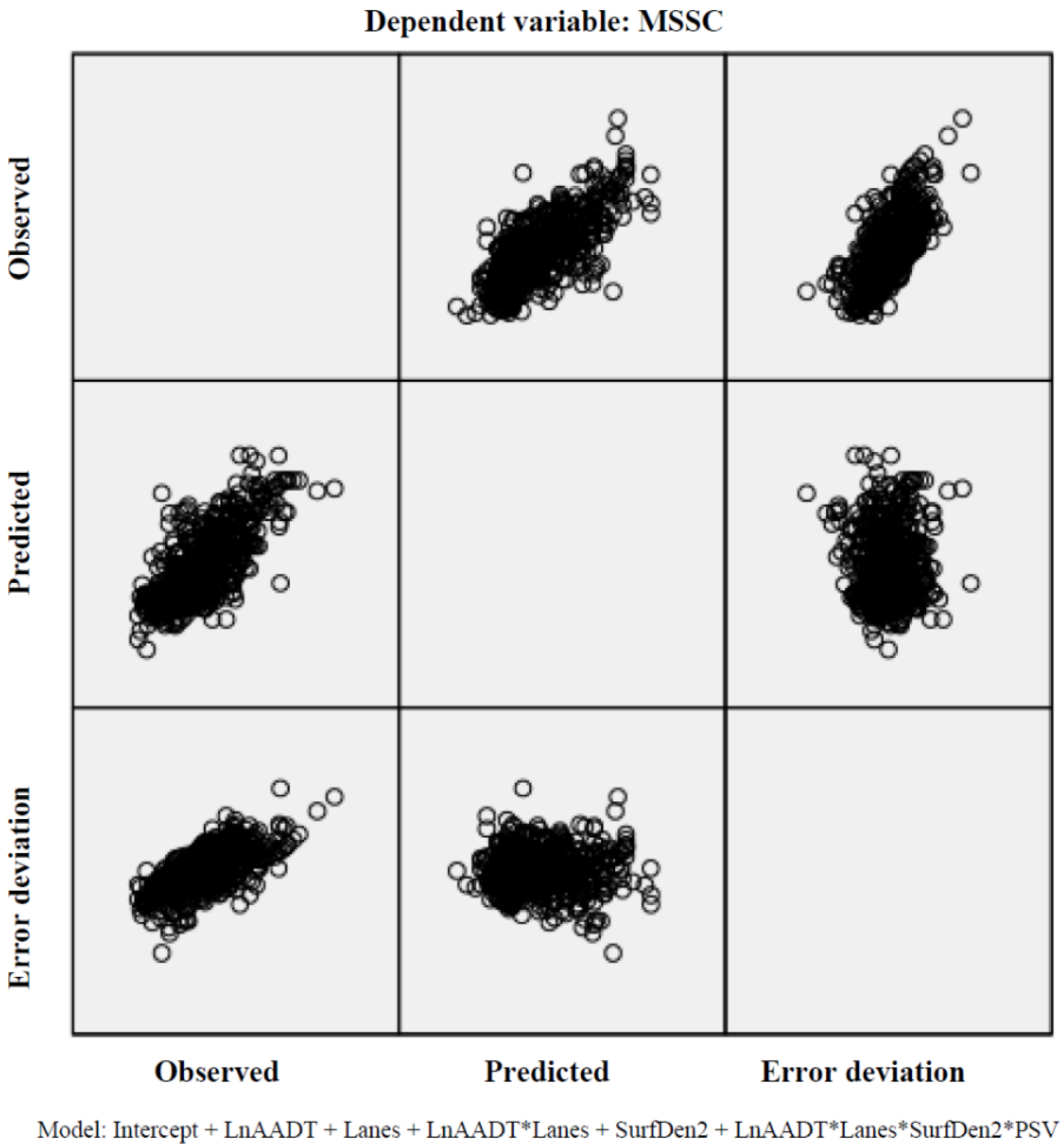
9.10 irudiak dispertsio-diagramak maila bakoitzarekiko erakusten ditu. Ikusten denez, puntuak bi grafikoetan ez daude horizontalki alienatuta, mailen artean ez dagoela bariantza homogeneorik adieraziz.





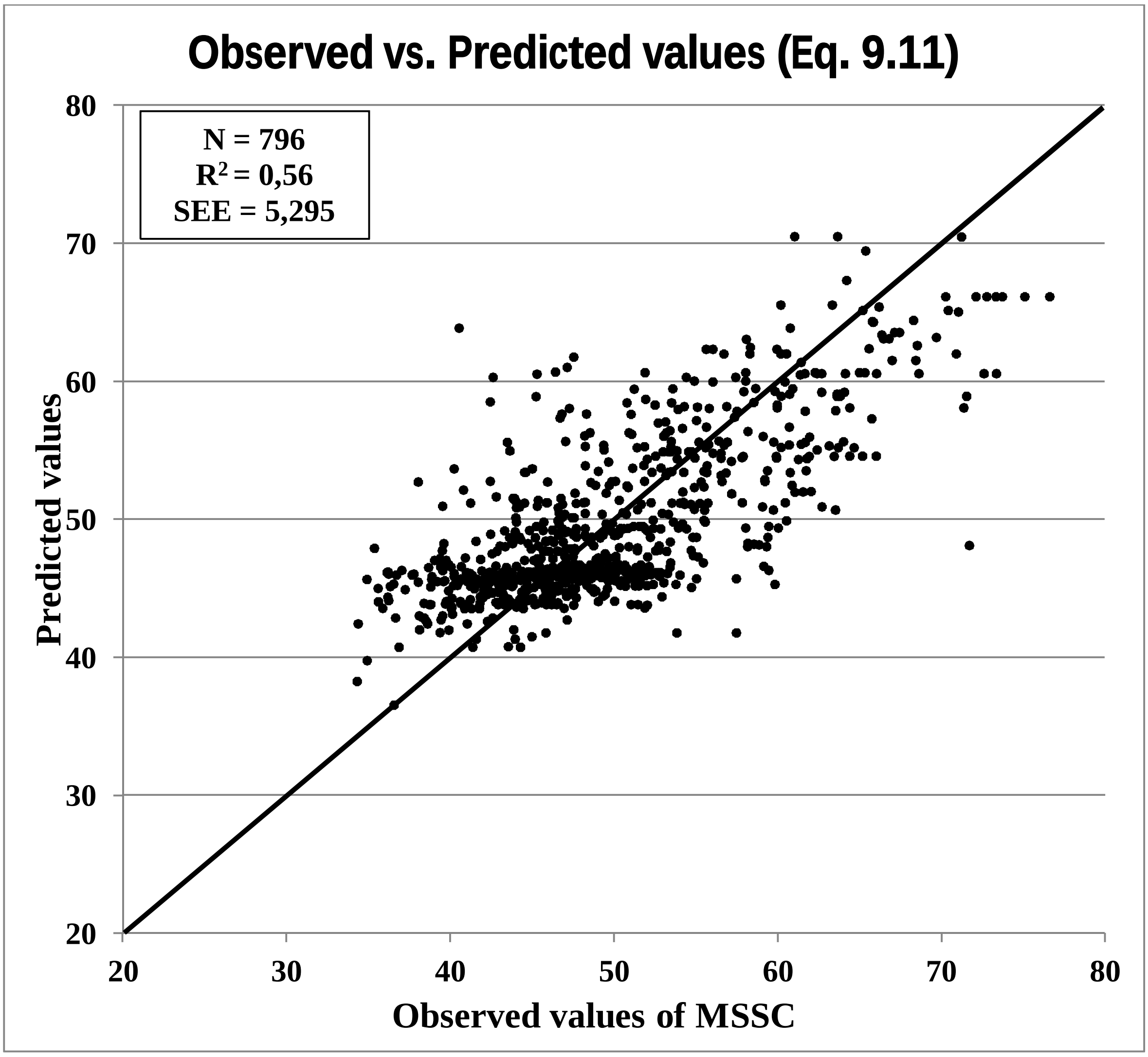
9.10 irudia. 9.11 ekuazioaren dispertsio-diagramak mailarekik. a) Desbideratze tipikoa, b) Bariantza.

9.11 irudiak erakusten du erroreen grafikoa. Iragarritako balioen eta errore estandarizatuen grafikoan ikusten denez, ez dago patroirik eta beraz, erroreak independenteak dira. Grafiko horrek ere erakusten du dispertsioa antzekoa da iragarritako balio guztien zehar, erroreen bariantzak homogeneoak direla adieraziz.



9.11. irudia. Estandarizatutako erroreen, neurtutako balioak eta iragarritako balioak 9.10 ekuazioaren bidez.

Neurtutako balioak eta iragarritako balioen arteko grafikoa detaile handiagoz erakusten da 9.12 irudian, 9.11 ekuazioaren modelorako.



9.12. irudia. Neurtutako eta 9.10 ekuazioarekin iragarritako balioen grafikoa.

IRI modeloetan ibilgailu astunen datuak nahiago ziren trafiko totalaren aurrean. Kasu hartan, *TotH.Veh* erabiltzea nahiago izan zen *TotVeh* aldagaiaren aurrean. Labainketarekiko erresistentziaren modelorako trafiko osoaren Eguneko Batez Besteko Intentsitatea (*AADT*) erabiltzea nahiago izan da *H.AADT*-en ordez. Marruskadurarako proposatutako ekuazioen egitura berdinean (9.10 et a9.11 ekuazioak), determinazio-koefizientean diferentzia handia ikusten da, *H.AADT* erabiltzen baldin bada (9.127 taula).

9.127 taula. 9.10 eta 9.11 ekuazioetan determinazio koefizienteak LnAADT eta LnH.AADT erabiliz

Trafikoarekin erlazionatutako aldagai sartua	9.10 ekuazioan proposatutako modeloan	9.11 ekuazioan proposatutako modeloan
LnAADT	0,530	0,560
LnH.AADT	0,464	0,510

Ikus daitekeenez, diferentziak marruskadura iragartzean IRI modeloetan baino handiagoak dira. Gainera, trafikoarekin erlazionatutako aldagaien arteko korrelazioak ez dira IRI modeloetan (8.22 taula) bezain handiak, 9.128 taulan erakusten den bezala

9.128 taula. Eguneko Batez Besteko Trafikoarekin erlazionatutako aldagaien arteko korrelazioak (Pearson-en koefizientea)

	AADT	**H.AADT**	**LnAADT**	**LnH.AADT**
AADT	1	0,864	0,843	0,769
H.AADT	0,864	1	0,759	0,846
LnAADT	0,843	0,759	1	0,911
LnH.AADT	0,769	0,846	0,911	1

Trafikoarekin erlazionatutako aldagaien arteko korrelazioan sorten den diferentzia handiagoa logikoa da marruskaduraren portaera modeloaren kasuan errepide mota handiagoa aztertzen da eta datu kopuru handiago erabiltzen da (IRI-erako 105, MSSC-rako 796).

Horren ondorioz, kontsideratu da datu gutxiko analisi partzial baten kasua ez dela. Analisi honetan, Bizkaiko lurraldeko errodadura ezaguneko tarte guztiak erabili dira eta emaitzek erakusten dute Eguneko Batez Besteko Intentsitatearekin (AADT) korrelazio hobeagoa dagoela. Honek Szatkwoski and Hosking (1972) eta Transit NZ (2002a) proposatutako modeloekin diferentzia bat suposatzen du. Hala ere, Szatkwoski and Hosking (1972)-ek aipatu zuten trafiko osoaren AADT sartzen bazela modeloan (5.69 ekuazioa) korrelazio altua ere aurkitzen zen ($R = 0{,}84$) (5.70 ekuazioa) baina Qcv-rekin baino txikiagoa ($R = 0{,}91$) (Qcv ibilgailu komertzialen, pisua > 1.500 kg) Eguneko Batez Besteko Intentsitatea). Hala ere, geroago, ekuazioa berriro egiaztatu zutenean praktiko balio errealekin, ikusi zen korrelazioa ez zela hain handia

Horrez gain, zaila da jakitea laborategietan sortutako leuntze-jarduera ibilgailu astun batek sortzen duenaren antzekoa den edo ibilgailu guztiek sortzen dutenaren antzekoa. Leuntze-jarduera, 5.47 eta 5.53 irudietan erakusten den bezala, heltzen da oreka fase batera leuntze-ziklo batzuen ostean eta hortik aurrera, marruskadura ez da txarragoa.

Hala eta guztiz ere, laborategiko baldintzek ez dute irudikatzen errealitateko baldintzak, non ibilgailu ezberdinek leuntzen dute errodadura geruzako agregakinak eta eguraldiak, urte-sasoietan marruskadura aldatzen duena.

Horren ondorioz, azaldutako arrazoiak kontuan hartuta, labainketarekiko erresistentziarako Bizkaia, Eguneko Batez Besteko Intentsitatea (eta ez ibilgailu astunen Eguneko Batez Besteko Intentsitatea) erabiltzen duten bi ekuazio aukeratu dira portaera modeloak bezala.

9.5. Proposatutako modeloan laburpena

Udako Batez Besteko SCRIM Koefizientea (MSSC, Mean Summer SCRIM Coefficient) iragartzeko edozein motako errepidean honako modelo hauek proposatzen dira, errodadura geruzaren materialaren, Eguneko Batez Besteko Intentsitatearen, errepide-motaren eta eskatutako Azeleratutako Leuntze Koefizientearen arabera:

Modeloan erabil daitezkeen errodadura geruzako materialak (SurfDen2) honako hauek dira:

a) Hormigoi bituminosoa, agregakinaren tamaina maximoa 16 mm (AC 16) edo 22 mm (AC 22) izanik

b) Nahaste etenak, BBTM 11A eta BBTM 11B barne

c) Nahaste drainatzailea, PA 11

d) Kare-esnea, eta espezifikoki, LB2

Modeloan sar daitezkeen errepide-motak noranzko bakoitzean dauden erreien kopuruaren araberakoa da (*Lanes*):

a) Bi erreiko errepide konbentzionala, errei bat noranzko bakoitzean

b) Galtzada banandutako autobidea, autobia edo errei anitzeko errepide, bi errei noranzko bakoitzean

c) Galtzada banandutako autobidea, autobia edo errei anitzeko errepide, hiru errei edo gehiago noranzko bakoitzean

Eskatutako Azeleratutako Leuntze Koefizientea (*Polished Stone Value*, *PSV*) ezartzen da Espainiako arauen arabera, eta errodadura geruzaren materialaren, zerbitzuan jartzen den momentuaren proiektuko erreiaren Trafiko Astuneko Kategoriaren eta errepide zerbitzuan jarri zen momentuaren (momentu horretan indarrean zegoen araua) araberakoa da.

Modelo laburra, bere determinazio-koefizientea 0,530 izanik egiaztatu zena, 9.12 ekuazioaren bidez azaltzen da:

$$MSSC = 78{,}497 - 2{,}321 * LnAADT + A_{LANES} + B_{SURF} + C_{LANES} * LnAADT \qquad [9.12]$$

Non

MSSC: Udako Batez besteko SCRIM Koefizientea, 0tik 100rako eskala batean adierazita.

Ln*AADT*: errepidearen Eguneko Batez Besteko Intentsitatearen logaritmo naturala, noranzko bietan, ibilgailu/egun-tan adierazita.

A_{LANES}: errepide-mota kontuan hartzen duen koefizientea eta 9.129 taularen balioak hartzen ditu.

9.129 taula. A_{LANES} koefizientearen balioak 9.12 ekuazioan

Errepide-mota	A_{LANES}
Bi erreiko errepidea, errei bat noranzko bakoitzean	26,693
Galtzada banandutako errepidea, bi errei noranzko bakoitzean	16,663
Galtzada banandutako errepidea, hiru errei edo gehiago noranzko bakoitzean	0

B_{SURF}: errodadura geruzaren materiala kontuan hartzen duen koefizientea eta 9.130 taularen balioak hartzen ditu:

9.130 taula. B_{SURF} koefizientearen balioak 9.12 ekuazioan

Errodadura geruzaren materiala	B_{SURF}
AC 16	-6,989
AC 22	-5,047
BBTM 11A	-5,468
BBTM 11B	-4,213
PA 11	-7,467
LB2	0

C_{LANES}: Ln*AADT*-ren balioan eragina duen koefizientea, eskuineko erreiaren trafikoaren banaketa errealago bat irudikatzeko eta 9.131 taularen balioak hartzen ditu.

9.131 taula. C_{LANES} koefizientearen balioak 9.12 ekuazioan

Errepide-mota	C_{LANES}
Bi erreiko errepidea, errei bat noranzko bakoitzean	-3,352
Galtzada banandutako errepidea, bi errei noranzko bakoitzean	1,328
Galtzada banandutako errepidea, hiru errei edo gehiago noranzko bakoitzean	0

Balio zehatzago kalkulatzeko, modelo luzea proposatzen da, 9.13 ekuazioa, determinazio-koefiziente hobeago bat lortzen zuena (R^2 = 0,560), eta 9.12 ekuazioko aldagaiez gain (beste balioekin), beste termino bat dauka eskatutako *PSV*-ren, errodadura geruzaren materialaren, errepide-motaren eta errepidearen bi noranzkoko *AADT*-aren konbinazioa irudikatzeko.

$$MSSC = 114{,}946 - 2{,}065 \cdot LnAADT + A_{LANES} + B_{SURF} + C_{LANES} \cdot LnAADT + D_{S-P-L} \cdot LnAADT \quad [9.13]$$

Non

MSSC: Udako Batez besteko SCRIM Koefizientea, 0tik 100rako eskala batean adierazita

Ln*AADT*: errepidearen Eguneko Batez Besteko Intentsitatearen logaritmo naturala, noranzko bietan, ibilgailu/egun-tan adierazita

A_{LANES}: errepide-mota kontuan hartzen duen koefizientea eta 9.132 taularen balioak hartzen ditu

9.132 taula. A_{LANES} koefizientearen balioak 9.13 ekuazioan, proposatutako modelo luzea

Errepide-mota	A_{LANES}
Bi erreiko errepidea, errei bat noranzko bakoitzean	9,473
Galtzada banandutako errepidea, bi errei noranzko bakoitzean	-19,756
Galtzada banandutako errepidea, hiru errei edo gehiago noranzko bakoitzean	0

B_{SURF}: errodadura geruzaren materiala kontuan hartzen duen koefizientea eta 9.133 taularen balioak hartzen ditu:

9.133 taula. B_{SURF} koefizientearen balioak 9.13 ekuazioan, proposatutako modelo luzea

Errodadura geruzaren materiala	B_{SURF}
AC 16	-34,317
AC 22	63,566
BBTM 11A	-40,151
BBTM 11B	-0,722
PA 11	-47,639
LB2	0

C_{LANES}: Ln*AADT*-ren balioan eragina duen koefizientea, eskuineko erreiaren trafikoaren banaketa errealago bat irudikatzeko eta 9.134 taularen balioak hartzen ditu.

9.134 taula. C_{LANES} koefizientearen balioak 9.13 ekuazioan, proposatutako modelo luzea

Errepide-mota	**C_{LANES}**
Bi erreiko errepidea, errei bat noranzko bakoitzean	-5,719
Galtzada banandutako errepidea, bi errei noranzko bakoitzean	1,817
Galtzada banandutako errepidea, hiru errei edo gehiago noranzko bakoitzean	0

$D_{S\text{-}P\text{-}L}$: errodadura geruza, arauetan eskatutako agregakinen Azeleratutako Leuntze Koefizientea eta errepide-motaren konbinazioa kontuan hartzen duen koefizientea. Faktore hauen mailen arteko konbinazio anizkoitza dagoenez konbinazio bakoitzerako zerrenda luzea aurkezten da. Balioak errepide-motaren arabera zerrendatzen dira, hiru mota baino ez dituena (9.133 taula, 9.134 taula eta 9.135 taulak).

9.135 taula. $D_{S\text{-}P\text{-}L}$ koefizientearen balioak bi erreiko errepide batean, errei bat noranzko bakoitzean 9.13 ekuaziorako, proposatutako modelo luzea.

Errodadura geruzaren materiala	**PSV**					
	40	**44**	**45**	**50**	**55**	**56**
AC 16	3,243	3,063	3,115	2,903	-	-
AC 22	-7,108	-10,389	-7,302	-8,084	-	-
BBTM 11A	-	-	4,993	3,834	-	-
BBTM 11B	-	-	-	-0,468	-	-
PA 11	-	-	5,007	4,482	-	-
LB2	-1,060	-0,374	-0,435	0	-	-

9.136 taula. $D_{S\text{-}P\text{-}L}$ koefizientearen balioak galtzada banandutako errepidean, bi errei noranzko bakoitzean 9.13 ekuaziorako, proposatutako modelo luzea.

Errodadura geruzaren materiala	**PSV**					
	40	**44**	**45**	**50**	**55**	**56**
AC 16	-	-	-1,595	-1,130	-	-
AC 22	-10,560	-	-	-	-	-
BBTM 11A	-	-	-	-0,663	-0,626	-0,570
BBTM 11B	-	-	-	-4,427	-	-
PA 11	-	-	-0,328	-0,091	-	0
LB2	-	-	-	-3,604	-	-

9.137. taula. D_{S-P-L} koefizientearen balioak galtzada banandutako errepidean, hiru errei edo gehiago noranzko bakoitzean 9.13 ekuaziorako, proposatutako modelo luzea.

Errodadura geruzaren materiala	**PSV**					
	40	**44**	**45**	**50**	**55**	**56**
AC 16	-	-	-1,090	-	-	-
AC 22	-	-	-9,791	-9,397	-	-
BBTM 11A	-	-	-	-0,392	-0,320	-0,456
BBTM 11B	-	-	-	-	-	-
PA 11	-	-	-	0,112	-	0
LB2	-	-	-	-	-	-

Ikus daitekeenez, hiru faktore hauen arteko konbinazio posible guztiak ez dira adierazten. Arauetatik, konbinazio batzuk ez dira posible, ez litzateke posible izango T00 kategoria izan bi erreiko errepide konbentzional atean, non bere PSV=56 izan behar izango litzateke.

Era berean, hiru erreiko galtzadak dituen autobia batek PSV handiagoak behar izango ditu. Horrez gain, trafiko handiko autobide eta autobietarako, Bizkaian nahiago da nahaste etenak edo drainatzaileak erabiltzea, euri prezipitazioak handiak dira urte osoa zehar klima ozeanikoagatik.

Tauletan (9.133 – 9.135 taulak) agertzen ez diren konbinazio baten kasuan, modelo laburra erabiltzea gomendatzen da (9.12 ekuazioa).

9.6. Ondorioak

Kapitulu honetan, labainketarekiko erresistentzian eragina duten faktoreen analisi zehatza gauzatu da. Lehenengo pauso baten, literaturan aurkitutako urte-sasoiko aldakuntzak kontuan hartuta, 2011ko neguan hartutako datuak udako datu bihurtu ziren, haien minimoa irudikatzeko. Eraldaketa gauzatu zen Gipuzkoan, Bizkaia ondoan dagoen beste probintzia, klima bera daukana, gauzatutako ikerketan ikusitako aldakuntza erabilita. Eragina zuten faktoreak trafiko astuneko Eguneko Batez Besteko Intentsitatea eta eskatutako PSV-a ziren.

2016an, Bizkaiko errepide-sarean labainketarekiko erresistentziaren datuak udan bildu ziren eta beraz, onar daiteke balio horiek marruskaduraren balio minimoak irudikatzen dituzte.

Lehenengo eta behin, IRI analisirako erabilitako bide-zoruaren egitura ezaguneko tarteen analisia, bi errei errepideetako bide-zoruaren egitura guztiz ezaguneko tarteak baino ez ziren aztertu. Emaitzek erakutsi zuten bide-zoru motak (malgua edo erdi-zurruna), geruza bituminosoen lodiera osoak eta burututako lan motak (sekzioa trazatu berri batetik edo mantentze- edo errehabilitazio-lan batetik etor daiteke) ez zeukatela eraginik marruskaduran. Benetan labainketarekiko erresistentzian eragina zuten aldagaiak eguneko batez besteko trafikoa (totala edo ibilgailu astunena), errodadura geruzako materiala eta eskatutako PSV-a.

Hau kontuan hartuta, posible da errepideen luzera osoa aztertzea baldin eta errodadura geruza eta trafiko bolumenak ezagunak diren. Analisia Eremu 1, 2 eta 3ko bi erreiko errepide-tarte guztietara zabaldu zen eta emaitzak onak ziren. Eremu hauetako (1, 2 eta 3) galtzada banandutako tarteak ere sartu ziren eta emaitza hobeagoak lortu ziren. Horren ondorioz, Kontserbazio Eremuko 4ko errepideak, batez ere Bilbo inguruan daude autobide eta autobiek osatua dagoena, modeloan ere sartu ziren.

Aldagai berria sartu zen, *Lanes*, errepide bakoitzeko errei kopurua kontuan hartzeko. Errodadura geruzaren materialen sailkapen gehigarriak ere sartu ziren materialak talde homogeneotan batzeko. Erregresio lineal anizkoitzeko modelo eta Erregresio Lineal Orokor Anizkoitzeko modelo asko aztertu ondoren, bi modelo proposatzen dira.

Modelo laburra, oinarrizko modelo bezala ikus daitekeena, errepidearen errei kopurua, errodadura geruzaren materiala eta errepide osoaren Eguneko Batez Besteko Intentsitatea kontuan hartzen ditu.

Modelo luzeak, modelo osagarri bezala ikus daitekeena, arauen araberako eskatutako Azeleratutako Leuntze Koefizientea ere kontuan hartzen du.

PART IV. CONCLUSIONS AND FUTURE RESEARCH LINES

Chapter 10. Conclusions and future research lines

10.1. Conclusions

In this PhD thesis IRI and skid resistance performance models were developed for being applied in the road network of Biscay in the all the road network levels except for the local network. The study of the models employed in the literature for pavement performance modelling, the analysis of available data in the pavement management system of Biscay and the development of the specific models for roads in Biscay provide some conclusions.

First of all, one of the most important conclusions obtained is that no model type can be defined as the best one to predict the pavement evolution. In general, probabilistic and deterministic models are said to be the most important ones since they have been the most widely used in pavement management around the world. Nonetheless, neither one of them nor another type can be stated as the most appropriate to forecast the evolution of any of the characteristics or indexes that describe the road state. All the kind of models (deterministic, probabilistic, including Markov chains, Bayesian, Artificial Neural Networks, subjective, etc.) have their advantages and disadvantages and based on the characteristics of the road network and the available data in the pavement management system, a model should be chosen. Consequently, according to the circumstances of the road agency and available data, one model will be the most suitable to be applied.

Taking into account the previous conclusion, the performance model that was thought to best adapt to the pavement management systems that the Regional Government of Biscay is developing was a deterministic and empirical model. Apart from geometric and traffic data, there is a huge quantity of data available about pavement structure and layers in the database as a result of an intensive examination of the all the projects carried out since the Regional Government of Biscay have started managing the road network of the territory in the 80s. The existing layers in the pavement structure and their thickness are known and introduced in Gestivía, the software of the PMS, by means of projects conducted in the road network. Hence, very detailed information about the existing pavement and rehabilitation and maintenance activities carried out is available. Consequently, a deterministic model, which can take advantage of the exhaustive and reliable information, was chosen. Moreover, it was decided to develop an empirical model because all the data came from the observation in the field and no laboratory test were made. Selected indexes to be modelled were IRI, which describes the irregularities of the pavement surface and is said to be one of the most interesting characteristics for users as it represents the riding comfort, and the skid resistance, by means of the SCRIM Coefficient, which describes the friction between the tyre and the pavement and is directly related to road safety. Moreover, these indexes were collected in various data collections and covered the entire road network, providing a complete dataset.

Regarding to the IRI performance modelling, the following conclusions can be extracted:

- The usual differentiation between flexible and semi-rigid pavements is necessary due to the difference performance that can be observed and factors that influence the IRI evolution.

- There is a wide variation for IRI values within a stretch with the same characteristics of pavement structure, age and traffic volumes, which makes necessary to predict the mean IRI value when using deterministic models. A more accurate analysis of the variance of IRI with identical affecting variables is carried out with probabilistic models. The probabilistic models are able to forecast the proportion of the pavement that will be in the predetermined states, while the deterministic models can forecast the mean value, regardless the existing variance.
- For new two-lane roads with flexible pavement, a model was developed, which includes the age of the pavement since it was constructed, the total thickness of the bituminous layers and the accumulated total number of heavy vehicles as the necessary variables. These factors are usual in the literature. The total thickness of the bituminous layers represents the strength of the pavement to withstand the loads of the traffic. Some attempts to create parameters to takes into account the influence of the different bituminous materials by means of the Young modulus did not show a better correlation, and therefore, they were discarded. It seems that the Young modulus was not a correct factor to reflect the strength of the materials. The same ideas was extended to the unbound material, crushed stone in all the observations, but not the thickness of the layer not a global parameter considering the bituminous and unbound materials did not show enough good correlations. The fact that the thickness of the unbound material is not present in the model suggests that it is not an influencing factor since the bituminous layers are responsible of the most part of the strength of the structure. When developing the model, a better coefficient of determination was obtained if the accumulated total number of vehicles crossing the section was included. Despite the better correlation of this model (an improvement of 3,5% in R^2), it was preferred to discard it because in the literature it is recognized the determinant influence of the heavy vehicles, due to its higher weight. Observing the correlation between both variables, the coefficient of Pearson was very high, due to the fact that the percentages of heavy traffic in considered two-lane roads were similar. Consequently, it is thought that the equation including heavy vehicles is more accurate, and can be extrapolated to other regions or countries.
- For new two-lane roads with semi-rigid pavements, the proposed equation for IRI prediction include the age of the pavement, the total accumulated number of total vehicles and heavy vehicles and a factor to take into consideration the different materials that can be introduced in the one or two layers that can compose the subbase of the semi-rigid pavement. In this model, the loads over the pavement are represented by a combination of the accumulated total vehicles and heavy vehicles, fact that can be interpreted as a higher influence of heavy vehicles, as it could be expected. For taking into account the strength of the semi-rigid pavement structure a factor that considers the materials of the subbase is needed. It does not need the thickness of these layers, since they are usually designed following the standard in force. It could be included in the model a factor that considered the materials of the layers of the subbase and the thickness of the bituminous layers. However, the model does not show any improvement and, since pavement design are conducting according to the standards, it was estimated that it was not necessary.

With regard to the skid resistance performance, the following conclusions can be drawn:

- Seasonal variations of skid resistance values, which are extensively described in the literature and in a research conducted in Gipuzkoa, another province of the Basque Country with the same oceanic climate, were also certified in Biscay. Data from winter in 2011 and from summer in 2016 clearly show a great variance for the same stretches and with very similar traffic volumes.
- A skid resistance prediction was possible to be developed if the variation range from winter to summer was established. The model indicated that the Average Annual Daily Traffic of heavy vehicles and the required Polished Stone Values in the Spanish regulations were the only predictor for it, in concordance with other models proposed in Great Britain and in New Zealand. Variables like the age of the pavement, the total thickness of the bituminous layers or the pavement type (flexible or semi-rigid) did not influence the available friction. The type of surface had to be taken into account to establish the variation range to be subtracted.
- Nevertheless, as in summer friction values are at their lowest and their variances is lower, it seems more reasonable to directly predict the lowest values from values in summer, avoiding the necessity of supposing a variation range between winter and summer
- With summer data and in stretches were the pavement structure was completely known in two-lane roads, the model development showed that factors such as the pavement type, which distinguish between flexible and semi-rigid pavements; and the work type, which considered differently pavements from new construction and from rehabilitation and maintenance activities, did not influence the skid resistance. Moreover, quantitative variables as the age of the pavement, the total thickness of bituminous layers, the rainfall data during the 15 days previous to the data collection, and the accumulated number of vehicles that have crossed the section were also non influencing factors for the friction prediction. The factor that were selected as determinant for forecasting were the traffic volumes of the year when the data collection was conducted, both total traffic and heavy traffic volumes; the surface layer material and the required Polished Stone Value in the aggregates according to the Spanish regulations.
- The fact that the age of the pavement and the total number of vehicles and heavy vehicles that have circulated in the stretch do not affect the skid resistance certifies the usually adopted performance of the friction evolution, which indicates that after an initial phase of improvement and quick polishing action, the surface arrives to an equilibrium phase where the friction only varies seasonally. This performance is also verified by the fact that observation that have not arrived to the equilibrium phase must be removed, since their values are higher. The consideration of 2 years as the minimum age for being included in the calculation was satisfactory.
- Furthermore, the fact that only the Annual Average Daily Traffic of total and heavy traffic, the existing surface layer and the required Polished Stone Value are the only decisive factors for predicting implies that the analysis can be conducted in all the stretches where the surface layer is known, implying that the complete pavement section is not necessary to be known to forecast the skid resistance. It certifies the difference between structural properties of the pavement sections and the surface characteristics of the pavement. They are not correlated and for surface characteristics it is only necessary to know surface properties. Consequently, the observations for the skid resistance modelling were extended to the entire road network of Biscay road, including double carriageway

motorways, as long as surface details were known.

- The final model developed for friction prognosis is composed of two equations. The short one includes the Annual Average Daily Traffic of the year of the data collection, a factor depending on the surface layer and a factor to consider if the road is a two-lane road, a double carriageway road with two lanes or three or more lanes per direction. The long one, which is slightly more accurate, includes an additional factor to the previous ones which considers the Annual Average Daily Traffic, the surface type, the type of road and the required Polished Stone Value globally. For a quick assessment, the short equation is enough for obtaining an approximate result. For a more accurate determination, the long equation is suggested.
- During all the phases of the skid resistance modelling, it was observed that the Annual Average Daily Traffic (*AADT*) of all the traffic volume in the year of the data collection provided a better correlation than the Annual Average Daily Traffic of the heavy traffic (*H.AADT*), considering a heavy traffic the vehicles that weight more than 3.500 kg. In the literature, it has been always proposed the employment of *H.AADT* instead of *AADT*, because it was stated that heavy vehicles determined the polishing action. Nevertheless, this model developed with 792 observations indicates that *AADT* better correlates than *H.AADT*. This could indicate that all the vehicles polish the aggregates and affect the friction and not only the heavy vehicles. It is possible that laboratory polishing tests are reproducing the polishing action of passenger cars and not the action of heavy vehicles.
- While the *H.AADT* registered in Spain shows the volume of heavy vehicles that circulates in the lane with the highest number of heavy vehicles, the data of *AADT* indicate the total traffic volume in the road in both direction, independently the type of road and the number of lanes per direction. Consequently, it was necessary to introduce a factor to consider this variability, denotes as Lanes, in the modelling. It appears in both proposed equations.

10.2. Future research lines

Regarding to the IRI modelling, once models for new two-lane roads with flexible and semi-rigid pavements, and a model for rehabilitated and maintained two-lane roads with flexible pavement were developed the following future research lines are proposed:

- Since the predicted IRI values indicate the mean in the stretch, analysis the variation range for the IRI values in the same stretch, by mean of the standard deviation observed in the examined stretches.
- Develop a model for double carriageway roads, for the lane in the right, for new roads with flexible or semi-rigid pavement.
- Develop a model for double carriageway roads, for the lane in the right, for maintained and rehabilitated roads with flexible or semi-rigid pavement
- Compare models of double carriageways motorways with the ones of two-lane roads to establish the influence of the lanes in the same carriageway with less heavy traffic volume.

With regard to the skid resistance, once that the minimum skid resistance available in the roads can be predicted, the following future research lines are proposed:

- As the predicted values represent the mean value of friction in the stretch, calculate the variation range for the values in the same stretch, by means of the standard deviation observed in the analyzed stretches.
- Develop a model to calculate the seasonal variation of the skid resistance, from winter to summer, by means of the stretches that have the same surface in winter in 2011 and in summer in 2016
- Analyze the factor that affect the texture of a pavement, measured by means of the Mean Profile Depth, and develop a model to represent and predict its performance.
- Extend the skid resistance modelling to include the SCRIM Coefficient and the texture data in a unique parameter like the IFI.
- Establish the thresholds (desired and minimum) to be adopted by the Regional Government of Biscay with regard to the skid resistance
- Write some recommendations for pavement structure design, especially with regard to the surface layer, to select the materials and characteristics to be required in projects in order to assure the maximum friction in the road.

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ANNEXES

Annex 1. List of roads managed by the Regional Government of Biscay

A.1.1 Introduction

In this annex, the entire list of the roads managed by the Regional Government of Biscay is presented. Roads are classified according to their network type, following the classification indicated in Table 6.3, after Norma Foral 2/2011, de 24 de marzo, de Carreteras de Bizkaia (BOB, 2011). The network types are preferential interest, basic, complementary, provincial and local. Additionally, each road is divided according to the road type: motorways, multilane highways and conventional roads (autopistas, autovías, carreteras multicarril y carreteras convencionales), after the Ley 37/2015, de 29 de septiembre, de carreteras [Law 37/2015, of 29 of September, about Roads] (BOE, 2015). Finally, a summary of the length by road type in each network is included. Showed data are extracted from Pérez-Acebo (2018).

A.1.2. Preferential Interest Network (Red Network)

Table A.1.1. Roads of the Preferential Interest Network (Red Network) of the Regional Government of Biscay (Pérez-Acebo, 2018)

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
AP-8	A-8	Autopista del Cantábrico (Peaje)	L.T.H. Gipuzkoa – E. Urgoiti (El Gallo)	APP	74,905 - 106,040	31,052
AP-8		Autopista del Cantábrico (Peaje)	Variante Sur de A-8 (Tramo Bilbao)	APP	115,085 - 126,500	12,555
AP-8	A-8	Autopista del Cantábrico (Peaje)	VSM. E. Trápaga – E. Puerto (Compartido A-8)	APP	126,500 - 129,504	2,924
A-8	A-8	Autopista del Cantábrico	E. Urgoiti (El Gallo) - Barakaldo (Cruces)	APL	106,040 - 122,040	16,010
A-8	A-8	Autopista del Cantábrico	Barakaldo (Cruces) - E. Trápaga	APL	122,040 - 126,500	4,367
A-8	A-8	Autopista del Cantábrico	E. Puerto - L.T.H. (Cantabria)	APL	129,504 - 139,219	9,632
N-240	N-240	Tarragona – Bilbao (Por Barazar)	El Gallo - L. T. H. Araba	CC	11,220 - 43,820	32,916
N-629	C-629	Burgos - Colindres (N-634)	L.T.H. - L.T.H.	CC	60,810 - 64,675	3,870
N-633	BI-631	Acceso al aeropuerto de Loiu (Por Aldekone)	N-637 - Aldekone (Compartido con BI-631)	AV	9,310 - 11,820	2,519
N-633		Acceso al aeropuerto de Loiu (Por Aldekone)	Aldekone - Aeropuerto	AV	11,820 - 13,730	1,905
N-634	N-634	Donostia/S. Sebastián a Santander y La Coruña	L.T.H. (Gipuzkoa) - Bilbao (Atxuri)	CM-CC	65,630 - 110,040	44,043
N-634	N-634	Donostia/S. Sebastián a Santander y La Coruña	Barakaldo (Burtzeña) - BI-628 (Nocedal)	CM-CC	116,990 - 126,270	7,740
N-634	N-634	Donostia/S. Sebastián a Santander y La Coruña	BI-628 (Nocedal) - L.T.H. (Cantabria)	CC	126,270 - 136,140	9,665
N-636	BI-632	Beasain a Durango (Por Kanpazar)	L.T.H. Gipuzkoa - Elorrio	CC	34,980 - 40,641	5,686
N-636	BI-632	Beasain a Durango (Por Kanpazar)	Elorrio - Durango	CM-CC	40,641 - 49,130	8,543
N-637		Cruces a Erletxes (Por Pte. Rontegi)	Enlace Cruces - Kukularra	AV	8,000 - 11,142	3,106
N-637		Cruces a Erletxes (Por Pte. Rontegi)	Kukularra - N-633	AV	11,142 - 18,550	7,413
N-637		Cruces a Erletxes (Por Pte. Rontegi)	N-633 - Erletxes	AV	18,550 - 28,180	9,626
N-639	C-639	Acceso al puerto por Zierbena	Puerto de Santurtzi - N-634	CC	15,880 - 24,080	8,185
N-644		Autovía del Puerto	A-8 – Puerto de Santurtzi	AV	129,504 - 132,220	2,726
Not managed by the Regional Government of Biscay						
AP-68	A-68	AUTOPISTA BILBAO - ZARAGOZA (**No foral**)	A-8 - L.T.H. Araba	APP	0,000 - 22,350	22,350
					TOTAL km	**246,833**

Note: APP: Autopista de peaje (Paying motorway); APL: Autopista libre (Not paying motorway); AV: autovía (Motorway); CM: Carretera Multicarril (Multilane highway); CC: Carretera convencional (Conventional road)

Table A.1.2. Length by type of road in the Preferential Interest Network of the Regional Government of Biscay (Pérez-Acebo, 2018)

Road type	Length (km)*
Autopistas de peaje (Paying motorways)	68,881
Autopistas libres (Not paying motorways)	30,009
Autovías (Motorways)	27,295
Carretera multicarril (doble calzada) (Multilane highways)	13,854
Carretera convencional (Conventional roads)	106,794
TOTAL	**246,833**

* Including AP-68 (Paying motorway not belonging to the RGB)

A.1.3. Basic Network (Orange Network)

Table A.1.3. Roads of the Basic Network (Orange Network) of the Regional Government of Biscay (Pérez-Acebo, 2018)

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-604	BI-3704	Bilbao a Asua (por Enekuri)	Bilbao – La Cadena (BI-2704)	CM-CC	2,740 - 7,454	4,782
BI-623	C-6211	Durango a Vitoria/Gasteiz	N-634 - L.T.H. Araba (Gomilaz)	CM-CC	27,808 - 48,830	20,862
BI-624	C-6210	Altube a Balmaseda	L.P. Burgos - Balmaseda (BI-636)	CC	64,440 - 67,070	2,580
BI-625	N-625	Orduña a Bilbao	Orduña - L. T. H. Araba	CC	351,370 - 354,210	2,960
BI-625	N-625	Orduña a Bilbao	L.T.H. Araba - Cruce N-634	CM-CC	372,560 - 387,310	14,770
BI-625	N-625	Orduña a Bilbao	Ibarsusi (N-634) - Santutxu	CM	391,031 - 392,440	1,420
BI-625	N-625	Orduña a Bilbao	La Salve - Ugazko	CC	395,000 - 396,115	1,084
BI-626		Túnel de Artxanda S/La Salve(Peaje)	Puente La Salve - N-637	CC	1,757 - 3,718	1,961
BI-627		Túnel de Artxanda S/Ugazko (Peaje)	Ugazko - BI-626	CC	2,109 - 3,621	1,512
BI-628	BI-644	Eje del Ballonti	Carmen - Kueto	CM-CC	9,790 - 11,450	2,080
BI-628		Eje del Ballonti	Kueto - Markonzaga	CM	11,450 - 12,810	1,330
BI-628		Eje del Ballonti	Markonzaga - Portugalete	CM	12,810 - 14,210	1,260
BI-628		Eje del Ballonti	Balparda - NOCEDAL	CM-CC	15,830 - 17,470	1,655
BI-630	C-6210	Balmaseda a Carranza	Balmaseda (BI-636) - L.P. CANTABRIA	CC	30,270 - 42,420	12,348
BI-630	C-6210	Balmaseda a Carranza	L. P. Cantabria (La Escrita) - L. P. Cantabria	CC	48,972 - 60,190	10,990
BI-631		A-8 a Bermeo	A-8 - Miraflores	AV	0,000 - 1,415	1,415
BI-631		A-8 a Bermeo	Ibarsusi - Derio (N-633)	AV-CM-CC	3,245 - 9,360	6,028
BI-631	C-6313	A-8 a Bermeo	Aldekone (N-633) - Mungia	AV	11,820 - 19,831	8,037
BI-631	C-6313	A-8 a Bermeo	Mungia - BI-2235	CC	19,831 - 37,340	17,892
BI-633	BI-134	Durango a Ondarroa (Por Trabakua)	Matiena - Berriz	CM-CC	31,450 - 33,780	2,280
BI-633	BI-140	Durango a Ondarroa (Por Trabakua)	Berriz - Markina	CC	33,780 - 47,515	13,895
BI-633	C-6213	Durango a Ondarroa (Por Trabakua)	Markina - Ondarroa	CC	47,515 - 59,545	11,586
BI-634	C-6320	Berango a Mungia	Sopelana (BI-637) - Sopelana (BI-2122)	CC	19,610 - 21,030	1,530
BI-634	BI-3121	Berango a Mungia	Elorza (BI-2704) - Mungia (BI-631)	CC	24,970 - 33,740	7,807
BI-635	BI-P-1325	Lemoa a Gernika	Lemoa - Amorebieta	CC	18,000 - 21,000	2,890
BI-635	C-6315	Lemoa a Gernika	Amorebieta - Gernika (BI-2238)	CM-CC	23,960 - 37,450	14,160
BI-636		Corredor del Cadagua	Bilbao (A-8) - VTE. Zalla	AV	4,170 - 22,110	17,717
BI-636	C-6318	Corredor del Cadagua	VARIANTE DE ZALLA - L.P. BURGOS	CC	22,110 - 34,440	12,236
BI-637	BI-103	KUKULARRA A SOPELANA	N-637 - Artaza	AV	7,910 - 13,360	5,108
BI-637		Kukularra a Sopelana	Artaza - Sopelana (Uribe-Kosta)	AV-CC	13,360 - 18,690	5,402
BI-638	C-6212	Deba a Ondarroa	L.T.H. Gipuzkoa - Ondarroa	CC	8,000 - 8,900	0,900
					TOTAL km	**210,477**

Note: APP: Autopista de peaje (Paying motorway); APL: Autopista libre (Not paying motorway); AV: autovía (Motorway); CM: Carretera Multicarril (Multilane highway); CC: Carretera convencional (Conventional road)

Table A.1.4. Length by type of road in the Basic Network of the Regional Government of Biscay (Pérez-Acebo, 2018)

Road type	Length (km)
Autopistas de peaje (Paying motorways)	0,000
Autopistas libres (Not paying motorways)	0,000
Autovías (Motorways)	39,437
Carretera multicarril (doble calzada) (Multilane highways)	19,470
Carretera convencional (Conventional roads)	151,570
TOTAL	**210,477**

A.1.4. Complementary Network (Blue Network)

Table A.1.5. Roads of the Complementary Network (Blue Network) of the Regional Government of Biscay (Pérez-Acebo, 2018)

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-644	BI-3744	Kareaga a Sestao (Vega Vieja)	N-634 - BI-628	CM	9,480 - 11,550	2,180
BI-647	BI-3734	Acceso a la Universidad	Axpe - BI-2731	CM-C	0,000 - 4,545	4,564
BI-712	BI-3712	Basauri a Bolueta	Basauri – Puete S/Nervión	CC	387,675 - 388,550	0,795
BI-732		Montefuerte a San Fausto	N-636 - N-634	CC	49,130 - 51,390	2,260
BI-735	BI-3735	Lutxana a Asua	Ribera de Erandio - BI-737	CC	8,520 - 11,220	2,740
BI-737	N- 637	Erandio a Erletxes	BI-735 – La Cadena	CC	13,430 - 13,605	0,175
BI-737	N- 637	Erandio a Erletxes	La Cadena - Gaztañaga	CC	13,605 - 14,086	0,481
BI-737	N- 637	Erandio a Erletxes	Gaztañaga - Derio (N-633)	CC	14,086 - 18,630	3,405
BI-737	N- 637	Erandio a Erletxes	VTE. Larrabetzu - Erletxes	CC	27,880 - 29,310	1,420
BI-738	BI- 637	Enlace de Erandio a Artaza	Enlace N-637 (Vías de servicio) – Enlace Udondo	CM	8,900 - 11,308	2,268
BI-745	BI-3745	Barakaldo a Valle de Trápaga/Trapagaran	Barrio de San Vicente - N-634	CC	9,090 - 11,580	2,465
Not managed by the Regional Government of Biscay						
BI-711	C-6311	Bilbao a Getxo (No Foral)	Elorrieta – Las Arena (Calle Amaya)	CC	BI-711	C-6311
					TOTAL km	**28,883**

Note: APP: Autopista de peaje (Paying motorway); APL: Autopista libre (Not paying motorway); AV: autovía (Motorway); CM: Carretera Multicarril (Multilane highway); CC: Carretera convencional (Conventional road)

Table A.1.6. Length by type of road in the Complementary Network of the Regional Government of Biscay (Pérez-Acebo, 2018)

Road type	Length (km)*
Autopistas de peaje (Paying motorways)	0,000
Autopistas libres (Not paying motorways)	0,000
Autovías (Motorways)	0,000
Carretera multicarril (doble calzada) (Multilane highways)	4,803
Carretera convencional (Conventional roads)	24,080
TOTAL	**28,883**

* Including BI-711 (Conventional road not belonging to the RGB)

A.1.5. Provincial Network (Green Network)

Table A.1.7. Roads of the Provincial Network (Green Network) of the Regional Government of Biscay (Pérez-Acebo, 2018)

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-2101	BI-3101	Larrauri a Bakio	BI-631 - Bakio	CC	20,540 - 25,340	4,650
BI-2120	C-6320	Mungia a Plentzia	Mungia a Plentzia	CC	15,550 - 26,565	11,053
BI-2121	BI-120	Mungia a Gernika	Mungia (BI-631) - Fruniz	CC	16,135 - 22,000	5,691
BI-2121	BI-121	Mungia a Gernika	Fruniz - Muxika	CC	22,880 - 28,780	5,380
BI-2121	BI-122	Mungia a Gernika	Muxika-Vista Alegre (BI-635)	CC	28,780 - 34,360	5,225
BI-2122	BI-634	Sopela a Plentzia	BI-634 - Plentzia (BI-2704)	CC	21,070 - 27,080	5,870
BI-2153	BI-3153	Andraka a Armintza	Andraka - Goikoerrota	CC	23,380 - 26,140	1,760
BI-2153	BI-3151	Andraka a Armintza	Goikoerrota - Armintza	CC	26,140 - 27,000	1,070
BI-2224	BI-3241	Gernika a Markina	Gernika (BI-635) - Ibartxu	CC	33,830 - 41,470	6,880
BI-2224	BI-124	Gernika a Markina	Ibartxu - ArbatzegiI	CC	41,470 - 45,100	3,860
BI-2224	BI-130	Gernika a Markina	Arbatzegi - Iruzubieta (BI-633)	CC	47,230 - 54,640	6,810
BI-2235	BI-635	Gernika a Bermeo	BI-2238 - Bermeo	CC	37,010 - 50,270	12,910
BI-2237	BI-3237	Muruetagane a Puerto de Elantxobe	Muruetagane - Puerto	CC	39,600 - 45,200	4,850
BI-2238	BI-638	Gernika a Lekeitio	BI-2235 – Rotonda Lekeitio	CC	31,830 - 53,230	21,605
BI-2301	BI-3301	Ermua a Arrangizgana	Ermua - BI-633	CC	41,640 - 48,030	6,230
BI-2405	BI-3405	Plazakola a Lekeitio	Plazakola - Oleta	CC	55,120 - 63,350	7,880
BI-2405	BI-3447	Plazakola a Lekeitio	Oleta - Lekeitio (BI-2238)	CC	64,000 - 67,090	3,040
BI-2521	BI-P-5321	Orduña a Altube	Orduña (BI-625) - L.T.H. Araba	CC	40,060 - 44,520	4,420
BI-2522	BI-521	Areta a Ziorraga (Por Orozko)	Areta – Variante de Orozko	CC	20,320 - 23,900	3,440
BI-2522		Areta a Ziorraga (Por Orozko)	Variante de Orozko	CC	23,900 - 25,090	1,170
BI-2522	BI-522	Areta a Ziorraga (Por Orozko)	Variante de Orozko - L.T.H. Araba	CC	25,040 - 30,030	4,910
BI-2543	BI-3543	Igorre a Otxandio (Por Dima)	Igorre - Dima	CC	22,660 - 25,000	2,270
BI-2543	BI-3543	Igorre a Otxandio (Por Dima)	Dima - Otxandio	CC	26,000 - 41,160	14,960
BI-2617	BI-P-5017	Villaverde de Trucios a Trucios	La Iglesia - L.P. Cantabria	CC	43,000 - 45,400	2,360
BI-2625	N-625	Pto. Orduña a Orduña	L.T.H. Araba - Orduña	CC	350,580 - 351,370	0,810
BI-2632	C-6322	Elorrio a Bergara	Elorrio - L.T.H. Gipuzkoa	CC	39,450 - 43,220	3,785
BI-2636	C-6213	Markina a Elgoibar	P.K. 50+860 - L.T.H. Gipuzkoa	CC	50,860 - 58,070	6,360
BI-2701	BI-500	Muskiz a Malabrigo (Por Sopuerta)	Muskiz (N-634) - Merkadillo	CC	20,120 - 28,270	8,320
BI-2701	BI-501	Muskiz a Malabrigo (Por Sopuerta)	Merkadillo - Malabrigo (BI-630)	CC	28,270 - 35,710	7,230
BI-2704	BI-3704	Asua a Plentzia	La Cadena (BI-604) - Plentzia (BI-2120)	CC	8,000 - 24,190	13,790
BI-2713	BI-3751	Larrabetzu a Alto de Gerekiz	Larrabetzu (SUR) - BI-2121	CC	12,180 - 24,075	11,265
BI-2731	BI-3731	UPV a Akarlanda	Campus UPV Leioa - Akarlanda (BI-2704)	CM-CC	14,790 - 18,120	3,230
BI-2757	BI-3757	Gallarta a La Arboleda	Variante de Gallarta	CC	0,000 - 1,125	1,125
BI-2757		Gallarta a La Arboleda	El Campillo – La Arboleda	CC	2,825 - 6,830	3,990
BI-2794	BI-3794	Enlace A-8 - La Arena	Eenlace A-8 - La Arena	CC	23,050 - 24,768	1,718
					TOTAL km	**209,917**

Note: APP: Autopista de peaje (Paying motorway); APL: Autopista libre (Not paying motorway); AV: autovía (Motorway); CM: Carretera Multicarril (Multilane highway); CC: Carretera convencional (Conventional road)

Table A.1.8. Length by type of road in the Provincial Network of the Regional Government of Biscay (Pérez-Acebo, 2018)

Road type	Length (km)
Carretera multicarril (doble calzada) (Multilane highways)	1,070
Carretera convencional (Conventional roads)	208,847
TOTAL	**209,917**

A.1.6. Local Network (Yellow Network)

Table A.1.9. Roads of the Local Network (Yellow Network) of the Regional Government of Biscay (Pérez-Acebo, 2018)

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-3101	BI-V-1001	Bakio a Bermeo	Bakio - Bermeo	CC	29,000 - 38,450	9,490
BI-3102	BI-V-1202	Mungia a Astoreka	PK 17+230 - BI-2713	CC	17,230 - 27,130	8,570
BI-3103	BI-V-1103	Mungia A Billela	Mungia - BI-2120	CC	0,750 - 1,450	0,690
BI-3104	BI-V-1204	Soietxe a Gamiz-Fika	BI-2121 - BI-3102 Cedida	CC	20,170 - 21,930	1,760
BI-3105	BI-V-1105	Gatika a Butroe (Por Laukiz)	Gatika - BI-634	CC	18,490 - 24,010	4,750
BI-3108	BI-V-1008	ELordui – Markaida (Mungia)	BI-631 - Markaida	CC	18,050 - 21,170	3,070
BI-3111	BI-V-1111	Astienza (Maruri) a Igartua (Gatika)	BI-2120 - BI-634	CC	20,500 - 22,010	1,470
BI-3112	BI-V-1112	Unbegana a Laukiz	BI-2704 - PK 16+440	CC	13,990 - 16,440	2,450
BI-3117		Astienza a Portumes	BI-2120 - BI-3152	CC	19,670 - 25,326	5,496
BI-3122	BI-2121	Travesía de Mungia	Mungia - BI-2121	CC	16,480 - 16,928	0,448
BI-3123	BI-V-1203	Soietxe a Altamira-S. Kristobal	BI-2121 - Ikasta	CC	20,040 - 26,750	6,430
BI-3123		Soietxe a Altamira-S. Kristobal	Ikasta - BI-2235	CC	26,750 - 34,930	8,131
BI-3124	BI-V-1024	Larrabasterra a Casarreina	PK 20+440 - BI-2704	CC	20,440 - 23,300	2,860
BI-3131	BI-V-1201	Zabalondo a Laukariz (Mungia)	BI-4117 (CEDIDA) - BI-3715	CC	14,000 - 16,020	1,880
BI-3148	BI-V-1148	Ibaizabal a Andra-Mari (Morga)	BI-2121 - BI-2713	CC	27,510 - 32,800	4,770
BI-3151	BI-V-1051	Plentzia a Goikoerrota	Plentzia - BI-2153	CC	26,000 - 29,760	3,080
BI-3152	BI-3151	Armintza a Bakio	Armintza - BI-2101 CEDIDA	CC	32,000 - 46,020	13,710
BI-3154	BI-V-1054	Krutzeta a Sagastikoetxe (Gorliz)	PK 0+330 - BI-2120	CC	0,330 - 1,060	0,730
BI-3174	BI-V-1004	Ramal de Emerando (Meñaka)	BI-2101 - BI-631	CC	20,790 - 22,510	1,650
BI-3213	BI-V-1213	Errigoitiolea a Gernika	BI-2121 – Cementerio de Gernika	CC	25,070 - 34,740	9,540
BI-3214	BI-V-1214	Aldai a Jainko (Arrieta)	BI-3213 - Jainko	CC	25,610 - 27,380	1,770
BI-3222	BI-V-1322	Dudoletagana a Marmiz (Por Mendata)	BI-3332 - BI-2224	CC	31,340 - 44,250	11,650
BI-3223	BI-123	Ramal de la BI-2238 a Gautegiz-Arteaga	Alto Ereño – Gautegiz Arteaga	CC	37,940 - 39,720	1,690
BI-3224	BI-2224	Gernika a Ibartxu	BI-2238 Cedida - Ibartxu	CC	33,350 - 41,130	7,050
BI-3231	BI-131	Ramal de la BI-3332 a Munitibar	Zugastieta (BI-3332) - Munitibar	CC	25,440 - 37,580	12,180
BI-3234	BI-V-1234	Gautegiz-Arteaga a Ibarrangelu (Por Laida)	BI-2238 - BI-4236	CC	37,920 - 50,660	12,600
BI-3238	BI-V-1238	Ibarrangelu a Ispaster	BI-2237 - BI-2238	CC	43,860 - 55,600	10,750
BI-3239	BI-V-1239	Arriederra a Natxitua (Ea)	BI-2237 - BI-3238	CC	41,970 - 43,920	1,900
BI-3242	BI-V-1242	Uarka a Ereño (Por Nabarniz)	Uarka (BI-2224) - PK 48+260	CC	35,200 - 48,260	12,340
BI-3302	BI-V-1402	Gomezeaga a Ermua (Por Mallabia)	BI-633 a límite Ermua	CC	38,080 - 41,800	2,900
BI-3313	BI-V-1333	Donibane a Santiago (Por Axpe)	Donibane - BI-4332	CC	37,700 - 39,130	1,430
BI-3321	BI-V-1221	Berriz a Elorrio	Olakueta - Ozontxueta	CC	35,570 - 40,250	4,530
BI-3331	BI-632	Travesia de Elorrio	Travesía de Elorrio	CC	39,550 - 40,320	0,790
BI-3332	BI-132	Enlace de la BI-635 a la N-634	Zugastieta – La Pilastra	CC	25,680 - 37,530	11,440
BI-3334	N-634	Travesía de Amorebieta	Travesía de Amorebieta	CC	92,830 - 93,260	0,430

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-3338	BI-2238	Antigua BI-2238 (Lekeitio)	BI-2238 - BI-3338 cedida	CC	51,840 - 53,360	1,520
BI-3340	BI-633	Aantigua BI-633 en Berriz y Mallabia	Besoitabeitia – Zengotita auzoa	CC	35,250 - 37,540	1,415
BI-3341	BI-V-1341	Urreta a Gerediaga (Por Garai)	N-634 - BI-633	CC	29,520 - 39,430	9,480
BI-3342	BI-142	Ramal de Berriz a la N-634	BI-3335 Cedida - Areitio	CC	34,730 - 38,570	3,850
BI-3344	N-634	Antigua N-634 (Ermua)	L.T.H. A Glorieta N-634	CC	65,570 - 65,980	0,410
BI-3438	BI-638	Lekeitio a Ondarroa	Lekeitio a Ondarroa	CC	55,150 - 67,000	11,710
BI-3447	BI-V-1247	Munitibar a Oleta	PK 38+510 - Oleta	CC	38,510 - 52,670	13,205
BI-3448	BI-V-1248	Aulesti a Markina-Xemein	BI-3447 - BI-633	CC	43,620 - 51,540	7,778
BI-3481	BI-V-1281	Ea a su Lim. Municipal. (Por Bedarona)	BI-3238 - BI-3238	CC	49,300 - 53,410	3,940
BI-3511	C-6211	Travesía de Otxandio	Travesía de Otxandio	CC	45,810 - 46,270	0,460
BI-3513	BI-V-5213	Orozko a Artea	Beraza - BI-3524	CC	25,340 - 39,010	13,710
BI-3524	BI-V-5124	Ugao-Miraballes a Artea	C/S.Bartolomé - PK 26+110	CC	12,690 - 26,110	11,450
BI-3527	N-240	Antigua N-240 (Igorre)	N-240 a límite con Aranzazu	CC	19,820 - 22,850	3,044
BI-3542	BI-V-1442	Zubizabal a Otxandio	N-240 - BI-623	CC	40,300 - 45,020	4,700
BI-3601	BI-P-5001	Cotarros a Las Muñecas (Sopuerta)	PK 30+000 - L. P. Cantabria	CC	30,000 - 33,275	3,275
BI-3602	BI-502	Güeñes a Malabrigo (Por Zalla)	PK 24+920 - Retola	CC	24,920 - 27,480	2,570
BI-3611	BI-V-5011	El Carral a La Herbosa (Sopuerta)	BI-2701 - BI-630	CC	30,600 - 35,440	4,890
BI-3614	BI-V-5014	Sopuerta a Traslaviña	BI-2701 - BI-630	CC	28,760 - 36,020	7,090
BI-3621	BI-V-5101	Gordexola a Zalla	PK 19+840 - BI-636	CC	19,840 - 25,820	5,913
BI-3621		Gordexola a Zalla	Variante final en Aranguren	CC	25,820 - 26,470	0,667
BI-3622	BI-V-5022	Ambasaguas (Carranza) a Lanestosa	Concha - N-629	CC	54,220 - 66,140	11,940
BI-3629	BI-V-5029	Concha a Alto de Treto (Por Presa)	BI-3622 - BI-3622	CC	54,470 - 61,920	7,430
BI-3631	BI-V-5031	Güeñes a Sopuerta	Güeñes - BI-2701	CC	21,150 - 34,100	12,820
BI-3632	BI-V-5032	Umaran a El Arenao (Galdames)	BI-3631 - BI-2701	CC	24,730 - 32,460	7,205
BI-3634	BI-V-5034	Sogarai a S. Francisco (Galdames)	BI-3631 - BI-3632	CC	0,000 - 0,360	0,360
BI-3636	BI-V-5036	Zalla a Ibarrazubi	P.K. 24,670 - BI-6351	CC	24,670 - 25,520	0,840
BI-3641	BI-P-5041	Padura a Maiorga	BI-636 - L.T.H. Araba	CC	16,940 - 21,010	3,870
BI-3651	BI-636	Antigua BI-636 Bilbao - Reinosa	Kastrexana a Arbuio	CC	6,500 - 11,030	4,500
BI-3651	BI-636	Antigua BI-636 Bilbao - Reinosa	Ibarra (Güeñes) -Sodupe	CC	13,490 - 15,060	1,580
BI-3651	BI-636	Antigua BI-636 Bilbao - Reinosa	Zeribai(Güeñes) - Artxube (Güeñes)	CC	17,000 - 17,925	0,935
BI-3651	BI-636	Antigua BI-636 Bilbao - Reinosa	Variante de Zalla conexión con BI-3621	CC	21,500 - 21,715	0,215
BI-3651	BI-636	Antigua BI-636 Bilbao - Reinosa	Allendelagua a La Herrera	CC	23,770 - 27,640	4,070
BI-3701	BI-V-5201	Arrigorriaga a Arkotxa (Por Zaratamo)	BI-625 - BI-3720	CC	8,250 - 13,830	5,590
BI-3702	BI-V-5202	Arrigorriaga a Zaratamo (Por Atxandio)	BI-3701 - BI-3701	CC	8,500 - 15,520	7,010
BI-3707	BI-V-1107	Loiu a Derio	Loiu - Derio	CC	9,900 - 13,490	3,670
BI-3709	BI-V-1109	Larrakoetxe a Boteola (Loiu-Gatika)	BI-3707 - BI-3105	CC	10,960 - 17,170	6,330
BI-3715	C-6313	Galbarriatu a Mungia (Por Derio)	Galbarriatu - Aldekone	CC	5,385 - 6,690	1,315
BI-3715	C-6313	Galbarriatu a Mungia (Por Derio)	Aldekone a Zabalondo	CC	10,890 - 11,750	1,000
BI-3720	BI-520	Ramal de BI-625 a Galdakao	Urbi (BI-625) – Galdakao (N-634)	CC	6,510 - 8,620	2,060
BI-3723	BI-V-5123	Abusu a Arrigorriaga	Zamakola – Límite Arrigorriaga	CC	2,300 - 5,070	2,696
BI-3726	BI-V-1136	Lezama a Gaztelumendi	Lezama - Gaztelumendi	CC	12,610 - 19,700	7,140
BI-3732	BI-V-1132	Egirleta/Santo Domingo a El Gallo (Galdakao)	BI-3741 - N-634	CC	3,480 - 16,117	12,813
BI-3733	BI-V-1023	Berango a Eguzkitza	Berango – Cantera Eguskitza	CC	17,980 - 20,230	2,330
BI-3736	BI-636	Basurto a Siete Campas	Basurto (N-634) – Alto de Kastrexana	CC	3,475 - 4,920	1,455

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-3737	BI-637	Gobela a Berango	Fadura - Ollaretxe	CC	15,310 - 16,195	0,946
BI-3737	BI-634	Gobela a Berango	Rotonda Ormaza - Berango	CM-CC	17,000 - 18,200	0,795
BI-3739	C-639	Sestao a Portugalete	BI-3739 Cedida - BI-3791	CC	12,640 - 12,850	0,230
BI-3740	N-639	Antiguo acceso al puerto por Zierbena	N-639 – Límite Abanto-Zierbena	CC	22,870 - 23,470	0,636
BI-3741	BI-V-1041	Enekuri a Egirleta/Santo Domingo (Bilbao)	BI-604 - BI-631	CC	5,200 - 10,860	5,645
BI-3742	BI-V-5042	Zorrotza a Kastresana (Bilbao)	N-634 Cedida - BI-3651	CC	5,140 - 7,910	2,710
BI-3746	BI-V-1046	Ugarte a Sestao (Por Galindo)	N-634 - BI-628	CC	10,070 - 11,790	1,720
BI-3747	BI-V-1047	Trapaga a Urioste (Por Galindo)	N-634 – Límite Ortuella	CC	10,960 - 13,730	2,830
BI-3748	BI-V-1048	Valle Trápaga/Trapagaran a Ballonti	Límite Trápaga - BI-3749	CC	0,487 - 0,727	0,240
BI-3749	BI-V-1049	Portugalete a Ortuella	BI-3749 Cedida - Urioste	CM-CC	13,920 - 14,812	0,892
BI-3750	BI-631	Begoña a Monte Abril	Begoña a Monte Abril	CC	2,040 - 3,040	1,000
BI-3752	BI-V-1042	Ramal al Casino de Artxanda (Bilbao)	BI-3741 - Explanada	CC	0,000 - 0,330	0,330
BI-3755	BI-V-5045	Valle Trápaga/Trapagaran a La Arboleda	BI-3755 – La Arboleda (Exc. La Reineta)	CC	13,100 - 20,075	6,630
BI-3784	BI-V-1184	Acceso a La Granja de Derio	BI-737 Cedida - BI-3784 Cedida	CC	0,020 - 0,335	0,315
BI-3791	BI-V-1091	Portugalete a Nocedal	BI-3739 - N-634	CC	12,830 - 17,000	2,225
BI-3794	BI-V-1094	Muskiz a Zierbena	N-634 – La Arena	CC	20,490 - 24,200	2,856
BI-3795	BI-V-1095	San Julian a Cobaron	BI-3796 – El Cobaron	CC	22,690 - 27,440	4,520
BI-3796	BI-V-1096	Lavalle a San Julian de Muskiz	BI-3794 - BI-3795	CC	0,000 - 0,270	0,270
BI-3931	BI-V-5131	Orduña a Izoria	La Antigua - L.P. Araba	CC	40,370 - 48,790	8,400
BI-3941	BI-V-1441	Otxandio a Kruzeta	BI-623 - L.P. Araba	CC	46,940 - 48,910	2,000
BI-3942		Otxandio a BI-3941	Otxandio a BI-3941	CC	46,940 - 47,320	0,460
BI-3950	BI-V-1302	Eibar a Etxebarria	BI-3950 Cedida - L.P. Gipuzkoa	CC	53,850 - 56,740	2,800
BI-4104	BI-V-1104	Ramal a Birlokoerrota (Mungia)	BI-2120 - Molino	CC	0,000 - 0,950	0,950
BI-4105	BI-V-1205	Larrauri a Belako	BI-631 - BI-2121	CC	19,530 - 23,601	4,083
BI-4118	BI-V-1218	Fruniz a Ganbe (Por Botiola)	BI-2121 Cedida - Escuelas	CC	22,160 - 26,050	3,890
BI-4137	BI-V-1137	Gerekiz a Eskerika (Morga)	BI-2713 - Eskerika	CC	28,870 - 31,840	3,000
BI-4155	BI-V-1055	Txatxarro a Saratxaga (Gorliz)	BI-3154 - Saratxaga	CC	0,000 - 0,700	0,700
BI-4203	BI-V-1003	Ramal a Matxitxako (Bermeo)	BI-3101 – Faro Matxitxako	CC	35,300 - 37,750	2,520
BI-4207	BI-V-1207	Acceso a la emisora TVE (Sollube)	BI-631 - Estación TV	CC	28,010 - 32,560	4,540
BI-4209	BI-V-1209	Bermeo a Artika	BI-4209 Cedida - Molino	CC	34,250 - 36,250	1,980
BI-4211	BI-V-1211	Errigoiti a San Lorenzo	BI-3213 - BI-4212 Cedida	CC	29,260 - 30,880	1,620
BI-4215	BI-V-1215	Gernika a Lumo	Gernika - Lumo	CC	32,620 - 34,135	1,635
BI-4221	BI-V-1321	Gomeztegi a Ibarruri (Muxika)	BI-3332 - Ibarruri	CC	26,780 - 27,550	0,770
BI-4223	BI-V-1323	Arrueta a Maume (Ibarruri-Muxika)	BI-3222 - Mauma	CC	33,510 - 36,550	3,040
BI-4236	BI-V-1236	Ibarrangelu a Akoda	BI-3234 - Akorda	CC	50,670 - 53,010	2,360
BI-4244	BI-V-1244	Kortezubi a Basondo (Gernika)	BI-2238 - Basondo	CC	36,040 - 38,530	2,540
BI-4251	BI-V-1151	Ziritza a Gorozika (Muxika)	BI-635 - Gorozika	CC	24,490 - 25,800	1,310
BI-4283	BI-V-1283	Ibinaga a Elantxobe	BI-2237 - Elantxobe	CC	44,100 - 45,130	1,030
BI-4332	BI-V-1332	Atxondo a Arrazola	BI-4332 Cedida - Antiguo F.C.	CC	36,320 - 40,550	4,190
BI-4337	BI-V-1337	Euba a Bernagoitia (Amorebieta-Etxano)	N-634 - Bernagoitia	CC	23,070 - 26,760	3,690
BI-4342	BI-V-1142	San Pedro a Boroa (Amorebieta-Etxano)	BI-3334 - Boroa	CC	17,150 - 18,670	1,400
BI-4343	BI-V-1143	Amorebieta a Aldana	BI-4343 CEDIDA – Aldana Goikoa	CC	20,600 - 24,780	4,160
BI-4344	BI-V-1144	Ramal a Aurtenengoa (Amorebieta-Etxano)	BI-4343 - Aurtenengoa	CC	0,000 - 0,400	0,400
BI-4346	BI-V-1146	Ramal de Antxia (Amorebieta-Etxano)	BI-4343 - Antxia	CC	0,000 - 0,300	0,300

Present code	Previous code	Denomination	Location of the stretch	Road type	Kilometre posts	Length (km)
BI-4401	BI-V-1301	Bolibar a Colegiata de Ziortza	BI-2224 - Colegiata	CC	52,530 - 54,410	1,890
BI-4403	BI-V-1303	Urberuaga a Larruskain (Markina)	BI-633 - Larruskain	CC	53,570 - 59,210	5,600
BI-4404	BI-V-1304	Markina-Xemein a Iturreta	BI-2636 Cedida - Lasarte	CC	50,680 - 55,610	4,910
BI-4406	BI-V-1306	Zulueta a Amoroto	BI-2405 – Entrada Amoroto	CC	63,030 - 64,900	1,880
BI-4449	BI-V-1249	Isuntza a Mendexa	BI-3438 - Mendexa	CC	55,240 - 57,300	2,060
BI-4482	BI-V-1282	Bedaroa a Punta de Ea	BI-3481 – Punta de Ea	CC	51,400 - 52,670	1,260
BI-4501	BI-V-5221	Zeanuri a Ipiñaburu	Zeanuri - Ipiñaburu	CC	31,240 - 36,850	5,600
BI-4511	BI-V-5211	Ramal a Murueta (Orozko)	BI-2522 - Murueta	CC	21,800 - 22,530	0,720
BI-4512	BI-V-5212	Orozko a Elexalde	Antigua BI-3522 - Iglesia	CC	24,680 - 25,460	0,780
BI-4514	BI-V-5214	Ibarra a Urigoiti (Orozko)	BI-3513 - Urigoiti	CC	29,240 - 33,140	3,840
BI-4516	BI-V-5126	Isla a Aracaldo	BI-625 - BI-4516 Cedida	CC	18,040 - 18,590	0,550
BI-4521	BI-V-5121	Ugao-Miraballes a Zollo	BI-625 - Zollourruti	CC	12,580 - 16,470	3,900
BI-4522	BI-V-5222	Ramal de Ipiña (Zeanuri)	BI-4501 - Ipiña	CC	36,520 - 37,150	0,630
BI-4523	BI-V-5223	Zeanuri a Otzerimendi	Antigua N-240 - Ermita	CC	30,850 - 34,670	3,710
BI-4531	BI-V-5231	Gezala a Igorrebaso (Igorre)	Igorre - Igorrebaso	CC	21,090 - 23,540	2,468
BI-4532	BI-V-5132	Ramal de Lendoño de Arriba (Orduña)	BI-3931 – Lendoño de Arriba	CC	42,540 - 45,180	2,640
BI-4533	BI-V-5133	Ramal de Mendeika (Orduña)	BI-3931 - Mendeika	CC	45,480 - 46,860	1,380
BI-4534	BI-V-5134	Orduña a Zedelika	BI-3931 Cedida - Zedelika	CC	40,040 - 42,160	2,110
BI-4543	BI-V-1443	Zelaieta a Gordobil (Otxandio)	BI-3542 - Gordobil	CC	44,280 - 45,740	1,440
BI-4544	BI-V-1444	Ramal a Mekoleta (Otxandio)	BI-4543 - Mekoleta	CC	0,000 - 0,130	0,130
BI-4545	BI-V-1445	Dima a Oba	San Balentin - Oba	CC	26,150 - 30,800	4,520
BI-4546	BI-V-1446	Santiago a Artaun (Dima)	BI-4545 - Artaun	CC	26,570 - 29,600	3,120
BI-4549	BI-V-1449	Ibarra a Baltzola	BI-4545 Cedida - Baltzola	CC	25,720 - 29,690	3,960
BI-4602	BI-V-5002	Olla a Montellano (Galdames)	BI-2701 – Caminos municipales	CC	24,480 - 26,750	2,220
BI-4612	BI-V-5012	Ramal de Avellaneda (Sopuerta)	BI-2701 – Casa Juntas	CC	0,000 - 0,120	0,120
BI-4613	BI-V-5013	Traslaviña a Santa Cruz (Artzentales)	BI-630 – Santa Cruz	CC	33,960 - 38,760	4,740
BI-4615	BI-V-5015	Labarrieta a Alen (Sopuerta)	BI-3614 – Camino Forestal	CC	32,940 - 38,130	5,110
BI-4618	BI-V-5018	Trucios a Cueto	BI-2617 Cedida - Cueto	CC	42,440 - 43,990	1,560
BI-4619	BI-V-5019	Espinal a Estación Villaverde de Trucios	BI-630 - Estación F.C.	CC	0,000 - 0,330	0,330
BI-4623	BI-V-5023	Ambasaguas a Matienzo (Carranza)	BI-3622 Cedida - Matienzo	CC	52,680 - 55,400	2,710
BI-4625	BI-V-5025	Concha a Lanzasagudas (Carranza)	La Tejera - Lanzasagudas	CC	54,870 - 60,410	5,580
BI-4626	BI-V-5026	El Cuadro a Bernales (Carranza)	BI-4625 - Bernales	CC	56,750 - 60,160	3,450
BI-4627	BI-V-5027	La Tejera a Pando (Carranza)	BI-4625 - Pando	CC	54,950 - 58,450	3,450
BI-4671	BI-V-5071	Pontarron a Buen Suceso (Carranza)	BI-630 - Monumento	CC	46,580 - 47,330	0,750
BI-4672	BI-V-5072	Concha a Aldeacueva (Carranza)	BI-4625 Cedida - Aldeacueva	CC	54,710 - 59,530	4,810
BI-4675	BI-V-5075	Culebrera a Paules (Carranza)	BI-630 - Paules	CC	51,690 - 55,230	3,520
BI-4678	BI-V-5078	Rioseco a Santecilla (Carranza)	BI-630 - Santecilla	CC	54,610 - 58,060	3,400
BI-4679	BI-V-5079	Ramal a Ranero	BI-4678 - Ranero	CC	56,130 - 58,250	2,130
BI-4703	BI-V-5203	Ramal a Burbustu (Zaratamo)	BI-3702 - Burubustu	CC	9,150 - 10,390	1,240
BI-4735	BI-V-1135	Ramal al sanatorio de Santa Marina	BI-631 - Sanatorio	CC	3,250 - 5,460	2,210
BI-4736	BI-V-1036	Ramala a Plaiabarri (Lutxana)	BI-735 - Plaiabarri	CC	0,000 - 0,150	0,150
BI-4906	BI-V-5322	Ramal a Delika	BI-2625 - L.P. Araba	CC	0,000 - 0,740	0,740
BI-4907	BI-V-5323	Ramal a Artomaña	BI-2521 - L.P. Araba	CC	36,910 - 37,260	0,350
					TOTAL km.	**610,902**

Table A.1.10. Length by type of road in the Local Network of the Regional Government of Biscay (Pérez-Acebo, 2018)

Road type	**Length (km)**
Carretera multicarril (doble calzada) (Multilane highways)	1,217
Carretera convencional (Conventional roads)	609,685
TOTAL	**610,902**

Annex II. Road files

In this annex road files created to analyze each road of the network managed by the Regional Government of Biscay is included. They are presented according to the Conservation Area each road belongs to.

A.2.1. Conservation Area 1

Road Denomination	**BI-631**	
Itinerary	A-8 – Bermeo (Mungia – Bermeo stretch only)	
Marker post (KP)	**From:** 19+0870	**To:** 37+0300
Transferred stretches		
-		

General comments:

The first recorded actuation was the "Pavement Plan" (Plan de Firmes, PROJ-260) of 1990. The entire road was rehabilitated, from beginning to the final (of this stretch). 2 layers of asphalt concrete of 5 cm were extended, over unbound material, but the thickness of the unbound material layer is unknown..

In 2003, after the rainfalls of 2002, new rehabilitation projects were performed (PROJ-381, PROJ-382), adding more bituminous layers to existing pavement, but the entire pavement structure is unknown.

In 2012, the connexion with BI-2101 (Larrauri-Bakio) (PROJ-280) is performed and the stretch 23+0000-23+0780 is reconstructed. In some parts, existing pavement is employed, in others, total reconstruction is performed. They are considered as initial pavement in the PMS

In 2015 the Bypass of Bermeo (PROJ-1776), a new outline, is performed, so the entire pavement structure is known.

Main actuations:

1989: PROJ-283 (01/04/1989) (supposed)
- 20+0900 – 21+0300: AC 16 surf S 5 cm + AC 22 base G 5 cm

1990: PROJ 260 (01/06/1990) (supposed)
- 19+0870 – 34+0940: AC 16 surf S 5 cm + AC calcareous 5 cm

2003: PROJ-382 (01/06/2003)
- 19+0500 – 22+0500: AC 16 surf S 5 cm
- 22+0500 – 23+0200: AC 16 surf S 5 cm + AC 22 bin G 5 cm + AC 32 base G 5 cm

2003: PROJ-381 (01/06/2003)
- 23+0200 – 24+0575: AC 16 surf S 5 cm + AC 22 bin G 5 cm + AC 32 base G 6 cm
- 24+0575 – 28+0000: AC 16 surf S 5 cm + AC 22 base G 5 cm
- 28+0000 – 31+0979: AC 16 surf S 5 cm + AC 22 bin G 5 cm + AC 32 base G 6 cm
- 31+0979 – 33+0072: AC 16 surf S 5 cm + AC 22 base G 5 cm
- 33+0072 – 34+0775: AC 16 surf S 5 cm + AC 22 bin G 5 cm + AC 32 base G 6 cm
- 34+0775 – 34+0940: AC 16 surf S 5 cm + AC 22 base G 5 cm

2004: PROJ-287 (01/06/2004)
- 22+0190 – 23+0080: Remove 5 cm + AC 16 surf S 5 cm

2008: PROJ-1336 (01/02/2008)
- 20+0000 – 21+0100: AC 16 surf S 5 cm

2012: PROJ-280 (01/06/2012)
- 23+0320 – 23+0560: AC 16 surf S 6 cm + AC 32 bin S 11 cm + AC 32 base S 8 cm + Gravel-cement 25 cm.
- 23+0560 – 23+0680: AC 16 surf S 6 cm + AC 32 bin S 11 cm + AC 32 base S 8 cm + MG (ZA) 25 cm

2015: PROJ-1776 (01/05/2015)
- 34+710 – 37+0300: BBTM 11A 3 cm + S-12 (AC 16 bin S) 8 cm + GC 29 cm + S-EST 25 cm

From 2010 to 2015, many maintenance works, mainly with slurries
- PROJ-1339: 23+0780 – 31+0860: Slurry 01/01/2012
- PROJ 1341: 31+0860 – 34+0940: Slurry 01/07/2011

IRI:

2000:

No complete pavement section is known.

2004:

Rehabilitation works are carried out but the entire pavement structure is unknown

2007:

No actuation is recorded from 2004 to 2007, but the entire pavement structure is unknown

2011:
There are maintenance works from 2007 to 2011, but the entire pavement structure is unknown
2016:
The new stretch corresponding to the Bypass of Bermeo (01/05/2015), 34+0710 – 37+0300, is analyzed as new outline. Reconstruction due to PROJ-280 (23+0320 – 23+0680) is introduced as new outline, like in the PMS. There is a part with semi-rigid pavement and other with flexible pavement. The stretch where a roundabout is placed is discarded as spurious since the IRI data is very high (IRI = 3,40 after 3 years).
Skid resistance. SFC:
2011:
No complete pavement structures are known in 2011.
2016:
The new outline, 34+710 – 37+0300, is analyzed, with just a one-year-old pavement. The stretch from PROJ-280 is also introduced in the modelling.
Skid resistance of surface layers in 2016
2016
19+1040 – 19+1480: PROJ-1334 (01/07/2011) LB2. 20+0000 – 21+0600: PROJ-1335 (01/07/2015) LB2. 21+0600 – 22+0190: PROJ-1337 (01/07/2011) LB2. 23+0000 – 23+0320: PROJ-280 (01/06/2012) AC 16 surf S 6 cm. 23+0680 – 23+0800: PROJ-280 (01/06/2012) AC 16 surf S 6 cm. 23+0800 – 31+0860: PROJ-1339 (01/06/2012) LB2. 31+0860 – 34+0750: PROJ-1341 (01/07/2011) LB2.

Road Denomination	**BI-634**	
Itinerary	Berango- Mungia	
Marker post (KP)	**From:** 19+0610	**To:** 21+0030
	From: 24+0970	**To:** 33+0740
Transferred stretches		
21+0030 – 24+0970: Sopela and Urduliz		

General comments:

The BI-634 road is divided in two parts, as it crosses the villages of Sopela and Urduliz.

There are no projects about initial pavement structure. The most ancient conserved projects are from 1995 (PROJ-181, 20+0290 – 21+0030) and 1997 (PROJ-189 20+0170 – 20+0400), where rehabilitations are recorded.

In the second part, in 2000, a new stretch was performed due to PROJ-28 (33+0250 – 33+0740), which connects the road with the bypass of Mungia (double carriageway).

In 2007, there is some partial rehabilitation from 24+0970 to 33+0250 and slurries are extended in the entire stretch, but no previous pavement structure is known.

In the first stretch (19+0610 – 21+0030), from 2009 to 2011, some rehabilitation and maintenance works are carried out.

In the new stretch related to the bypass of Mungia, 5 cm of S-12 are extended in 2012 (PROJ-1348; 32+0550 – 33+0750.

Briefly, it is known the width of bituminous layers that have been extended, but there are no references about previous pavement structure. Therefore, only the new stretch related to the bypass of Mungia is analyzed.

Main actuations:

1995: PROJ-181 (01/06/1995)
- 20+0290 – 21+0030: AC 22 surf S 6 cm + AC 32 base G 7 cm + ZA/GE 25 cm. Doubts.

1997: PROJ-189 (01/06/1997)
- 20+0170 – 20+0400: AC 16 surf S 6 cm + (G20 7 cm + AC 22 base G 7 cm + ZA 25 cm) partially

2000: PROJ-28 (01/07/2000)
- 33+0250 – 33+0740: AC 16 surf S 6 cm + AC 22 base G 11 cm + MG 25 cm + E2

2006: PROJ-1342 (01/02/2006)
- 19+0610 – 19+0810: PA-11 4 cm

2007: PROJ-183 (01/01/2007)
- 24+0970 – 33+0250: Partial repairs + Slurry (LB-2 and LB-4)

2007: PROJ-1346 (01/06/2009)
- 20+0930 - 21-0030: AC 16 surf S 5 cm

2011: PROJ-1344 (01/01/2011)
- 19+0810 – 20+0700: AC 16 surf S 5 cm

2011: PROJ-1772 (30/08/2011)
- 20+0520 – 20+0720: BBTM 11A 4 cm

2012: PROJ-1348 (01/01/2012)
- 32+0550 – 33+0750: AC 16 surf S 5 cm

2014: PROJ-1350 (01/08/2014)
- 33+0370 – 33+0720: LB2

IRI:

2000:

The IRI data collection of 2000 was carried out before the stretch related to the bypass of Mungia was finished. Hence, no data from 2000 are employed.

2004:

Stretch from 33+0250 to 33+0740 is analyzed (related to the bypass of Mungia).

2007:

Stretch from 33+0250 to 33+0740 is analyzed (related to the bypass of Mungia).

2011:

Although no rehabilitation or maintenance works are recorded in the stretch 33+0250 - 33+0740, global and individual IRI improvements are observed. Therefore, data from 2011 are no employed. Perhaps, the PROJ-1348 (01/01/2012) was performed before IRI data collection in 2011.

2016:
From 32+0550 to 33+0750. 5 cm of AC 16 surf S are extended (PROJ-1348) in 2012. Furthermore, slurry is extended from 33+0370 to 33+0720 (PROJ-1350) in 2014. The performance of the slurry and the performance of the stretch 33+0250 – 33+0370 without slurry (only the rehabilitation of 5 cm of AC 16 surf S) are introduced.
Skid resistance. SFC:
2011:
Completely known pavement sections of 2016 were maintained after 2011. No section from 2011 is introduced.
2016:
The performance of the skid resistance of the AC 16 surf S layer (01/01/2012) and the slurry (01/08/14) is analyzed in the stretch 32+0550 – 33+0750, where the whole pavement section is known.
Skid resistance of surface layers in 2016
2016
19+0610 – 19+0810: PROJ-1343 (01/06/2010) PA 11 4 cm 19+0810 – 20+0520: PROJ-1342 (01/06/2011) AC 16 surf S 5 cm. 20+0520 – 20+0720: PROJ-1772 (01/06/2011) BBTM 11A 4 cm. 20+0720 – 20+0930: PROJ-1345 (01/06/2009) LB2. 20+0930 – 21+0030: PROJ-1346 (01/06/2009) AC 16 surf S 5 cm. 32+0690 – 33+0370: PROJ-1348 (01/06/2012) AC 16 surf S 5 cm.

Road Denomination	**BI-635**	
Itinerary	Lemoa –- Gernika	
Marker post (KP)	**From:** 18+0000	**To:** 21+0000
	From: 23+0520	**To:** 37+0480
Transferred stretches		
21+0000 – 23+0520: Amorebieta-Zornotza		

General comments:

The BI-635 road is divided in two parts because it crosses the village of Amorebieta-Zornotza (21+0000 – 23+0520).

In the first part, the initial pavement structure is unknown. The first actuation is a reconstruction work (PROJ-324) in 1997, where 10 cm of bituminous layers are extended. Later, rehabilitation and maintenance works are carried out. No data from this part is introduced in the analysis.

In the second part, the first project is registered in 1975 (PROJ-193), which indicates that a rehabilitation of the entire length of this stretch was carried out. 2 layers of 5 cm of bituminous material were extended, but the existing materials are unknown. It cannot be considered as the initial pavement structure. In 1990 (PROJ-228, PROJ-229, PROJ-230) another rehabilitation of 10 cm of bituminous layers is performed over the existing road pavement.

In 23/12/2000 (PROJ-191), additional rehabilitation with bituminous layers is added.

In 01/11/2003, a new stretch, the Bypass of Gernika (PROJ-30), is concluded, adding more kilometres to the road, 34+100 – 37+0480. In 2005, from 34+0100 to 36+0729, 5 cm of AC 16 surf S are extended in the lane 1 of the Bypass of Gernika.

In 2009, 6 cm of S-12 are added from 32+0800 to 33+0100 (PROJ-278).

From 2008 onwards, series of maintenance actuations is carried out on the entire road.

There is a project, PROJ-1924, which includes a Bypass of part of the road by means of two tunnels, providing a double carriageway road. In 31/10/2015, some parts, before and after the tunnel were opened to traffic (24+0700 – 25+0710 and 26+0950 – 27+0300), with new pavement structure and double carriageway. There are only IRI data from the ascending carriageway.

Main actuations:

1975: PROJ-193 (01/12/1975)
    22+0900 – 35+0900: AC 16 surf S 5 cm + S12 (AC 16 base S) 5 cm
1990: PROJ-228 (01/06/1990)
    23+0520 – 27+0000: AC 16 surf D 6 cm + S-12 (AC 16 base S) 4 cm
1990: PROJ-229 (01/06/1990)
    27+0000 – 31+0000: AC 16 surf D 6 cm + S-12 (AC 16 base S) 4 cm
1990: PROJ-230 (01/06/1990)
    31+0000 – 34+0000: AC 16 surf D 6 cm + S-12 (AC 16 base S) 4 cm
1997: PROJ-324 (01/06/1997)
    18+000 – 21+0000: AC 16 surf S 5 cm + G-20 5 cm
2000: PROJ-191 (23/12/2000)
    22+0880 – 23+0500: AC 16 surf S 5 cm
    26+0000 – 27+0450: BBTM 11A 3 cm + AC 22 bin S 6 cm+ AC 22 base G 5 cm
    27+0450 – 32+0310: BBTM 11A 3 cm + AC 22 bin S 6 cm+ AC 22 base G 5 cm
2003: PROJ-30 (01/11/2003)
    34+0100 – 37+0480: BBTM 11A 4 cm + S-12 4 cm + S-12 4 cm + GE 28
2005: PROJ-323 (01/06/2005)
    32+0600 – 36+0729: AC 16 surf S 4 cm Lane 1
2006: PROJ-1355 (01/01/2006)
    18+0780 – 19+0245: AC 16 surf S 5 cm
2008: PROJ-1374 (01/06/2008)
    36+0750 – 37+0460: LB2
2009: PROJ-278 (01/06/2009)
    32+0800 – 33+0100: AC 16 surf S 6 cm
2010: PROJ-1372 (01/06/2010)
    33+0250 – 34+0250: LB2
2013: PROJ-1373 (01/07/2013)
    34+0100 – 37+0490: LB2

2015: PROJ-1924 (31/10/2015) 24+0700 – 25+0710: Double carriageway, BBTM 11A 3 cm + S12 4 cm + S12 5 cm + SC 33 cm 26+0950 – 27+0300: Double carriageway, BBTM 11A 3 cm + S12 4 cm + S12 5 cm + SC 33 cm
IRI:
2000:
No complete pavement structure is known.
2002:
No complete pavement structure is known.
2004:
There are data from the new stretch, the Bypass of Gernika (34+0100 – 37+0480), which is considered as New Layout.
2007:
It is analyzed the performance of the new stretch, the Bypass of Gernika. Stretch 32+0600 – 36+0729 was maintained with 4 cm of AC surf S in lane 1, included in the analysis.
2011:
There are rehabilitation activities in 2008 (PROJ-1374) and in 2010 (PROJ-1372) in the Bypass of Gernika. It is analyzed as maintenance work performance, with different ages. The stretch from 34+0250 to 36+0750 is rejected because it includes improvements (global and local) and no activities are registered.
2016:
In 2013 the whole Bypass of Gernika is resurfaced with a Slurry layer (PROJ-1373, 01/07/2013). Its performance until 2016 is analyzed. The ascending carriageway of the PROJ-1297 is analyzed (24+0700 – 25+0710 and 26+0950 – 27+0300).
Skid resistance. SFC:
2011:
Data collected in winter 2011 cannot be compared with those from summer 2016 because a maintenance activity was performed in 2013. Therefore, no data is analyzed.
2016:
The values of the Slurry performance from 01/07/2013 are analyzed in the stretch of the Bypass of Gernika, where all the pavement structure is known. No data from new outline carried out in PROJ-1924 is available.
Skid resistance of surface layers in 2016
2016
18+0000 – 18+0500: PROJ-1352 (01/08/2013) LB2. 18+0500 – 18+0890: PROJ-1354 (01/07/2014) LB2. 18+0890 – 19+0250: PROJ-1356 (01/08/2013) LB2. 19+0250 – 19+0700: PROJ-1980 (01/06/2012) LB2. 19+0700 – 20+0040: PROJ-1981 (01/07/2014) LB2. 20+0040 – 20+0500: PROJ-1360 (01/06/2012) S-12 5 cm. 27+0310 – 28+0010: PROJ-1943 (30/06/2016) F-10 3 cm. 28+0630 – 28+1060: PROJ-1366 (01/07/2014) LB2. 29+0410 – 30+0050: PROJ-1367 (01/06/2012) LB2. 30+0400 – 32+0110: PROJ-1368 (01/06/2010) LB2. 32+0110 – 32+0600: PROJ-1369 (01/06/2012) LB2. 32+0820 – 33+0800: PROJ-1370 (01/07/2014) LB2. 33+0800 – 34+0100: PROJ-1372 (01/06/2010) LB2.

Road Denomination	**BI-735**	
Itinerary	Lutxana - Asua	
Marker post (KP)	**From:** 8+0520	**To:** 11+0250
Transferred stretches		
-		

General comments:

The initial pavement structure is unknown. Except from maintenance actuations from 2008, there are only 3 construction projects registered in this road.

In the first one, PROJ-421, in 1987, in 9+0950 – 10+0300, where it is said that 5+7 cm of hot mix asphalt is extended over 45 cm of unbound aggregate. Important deterioration is observed in 2000 and in 2004. Improvement is regarded in 2007, although the maintenance activity is registered in 2008. Data from 2007 is decided not to be included in the analysis.

In 1994 (01/12/1994), under PROJ-301, 13 cm of HMA over 35 cm of aggregates treated with slag were extended (11+0100 – 11+0250), but this complete pavement section is not introduced in the PMS, due to some doubts according to some pavement tests taken in 2000, not reflecting the materials indicated.

The other project, PROJ-307, carried out in 01/02/2013, was a rehabilitation of the initial part of the road (8+0520 – 8+0750), due to the renovation of the initial crossroad, but the entire pavement section is unknown. Moreover, some existing layers were reused, but the exact length is unknown. It is not introduced as initial pavement in the PMS.

The rest of the projects are maintenance activities registered since 2008, which show the extension of slurry layers

Main actuations:

1987: PROJ-421 (01/07/1987)

9+0950 – 10+0300. AC 16 surf S 5 cm + AC 22 base S 7 cm + Unbound material 45 cm

1994: PROJ-301 (01/12/1994)

11+0100 – 11+0250: S-12 6 cm + G-25 7 cm + GE 25 cm (not verified with samples taken in 2000).

2008: PROJ-1377 (01/01/2008)

10+0860 – 11+0200: LB-2

2008: PROJ-1382 (01/07/2008)

9+0750 – 10+0330: LB-2

2013: PROJ-307 (01/12/2003)

8+0520 – 8+0750: BBTM 11A 3 cm + S-20 6 cm + G-20 9cm + GC 25 cm (not in the entire length)

IRI:

2000:

The evolution of the stretch 9+0950 – 10+0300 is analyzed. After 13 years and not actuations, an average IRI of 4,03 is obtained, which could be real.

The evolution of the stretch 11+0100 – 11+0250 is rejected because samples taken do not show the section of the .project.

2004:

The evolution of the stretch 9+0950 – 10+0300 is analyzed. After 17 years and not actuations, an average IRI of 5,47 is obtained, which could be real.

2007:

In the stretch 9+0950 – 10+0300, a maintenance work is dated in 2008, but improvement in IRI is observed. Therefore, the rehabilitation should be carried out in 2007, before data collection. Consequently, data from 2007 in this part are discarded.

2011:

The evolution of the stretch 9+0950 – 10+0300 maintained with a slurry layer (LB2) in 01/01/2008 is analyzed from 2008 to 2011

2016:

The evolution of the stretch 9+0950 – 10+0300 maintained with a slurry layer (LB2) in 01/01/2008 is analyzed from 2008 to 2016.

Skid resistance. SFC:

2011:

Skid resistance data in 2011 were collected in summer, 08/08/2011. Consequently, data can be used for Mean Summer SCRIM Coefficient calculation. Data from the stretch 9+0950 – 10+0300, maintained with a slurry layer (LB2) in 2008, are analyzed from 2008 to 2011.

2016:
The evolution of the slurry, LB2, of the stretch 9+0950 – 10+0300 are analyzed from 2008 to 2016.
Skid resistance of surface layers in 2016
2016
8+0520 – 8+0750: PROJ-307 (01/02/2013) BBTM 11A 3 cm 8+0750 – 8+0990: PROJ-1379 (01/05/2008) LB2. 9+0000 – 9+0940: PROJ-1380 (01/07/2011) LB2. 10+0300 – 11+0100: PROJ-1376 (01/07/2011) LB2 11+0120 – 11+0220: PROJ-301 (01/12/1994) AC 16 surf S 6 cm

Road Denomination	**BI-737**	
Itinerary	Erandio - Erletxes	
Marker post (KP)	**From:** 13+0890	**To:** 16+0740
	From: 17+0000	**To:** 18+0010
	From: 18+0540	**To:** 18+0630
	From: 27+0890	**To:** 29+0300

Transferred stretches

16+0740 – 17+0000: Loiu
18+0010 – 18+0540: Sondika
18+0630 – 27+0890: Derio – Zamudio - Lezama

General comments:

There are almost no project registered of this road, but it is an important road, with traffic and it is obvious that there have been many works.

The only construction project (no maintenance activity) is PROJ-301 (01/12/1994), related to the roundabout of Asua (also commented in road BI-735), and in the initial part of the road, the pavement structure was totally renewed, specifically from 13+0890 to 14+0150 and from 14+0550 to 14+0630, zones related to roundabouts. These stretches are introduced as initial pavement structure in the PMS In the intermediate part between these two roundabouts, the existing pavement structure was partially used and new layers were extended.

Apart from this project, the rest are maintenance activities from 2005 onwards.

Main actuations:

1994: PROJ-301 (01/12/1994)

13+0890 – 14+0150: AC 16 surf S 6 cm + AC 32 base G 7 cm + GE 25 cm
14+0550 – 14+0630: AC 16 surf S 6 cm + AC 32 base G 7 cm + GE 25 cm

IRI:

2000:

13+0890 – 14+0150: data introduced as new outline, evolution from 1994.
14+0550 – 14+0630: data introduced as new outline, evolution from 1994.

2004:

13+0890 – 14+0150: IRI data show an improvement, both individually and globally. If this improvement is not taken into account, data from subsequent years show great IRI progression. It is decided not to take these values into account. A different tester is supposed.

14+0550 – 14+0630: IRI data show an improvement, both individually and globally. It suggests that a maintenance work, not recorded, was carried out. A maintenance work was recorded in 01/06/2005, but it could be carried out before, in 2004, according to the data.

2007:

13+0890 – 14+0150: IRI data are taken into account to see IRI progression when no maintenance works are carried out, which seem to be realistic, according to values.

14+0550 – 14+0630: Values can be regarded as possible, taking into account the maintenance work of 2005. However, values from 2011 are better, and this stretch is discarded from this data. Moreover, the short distance, under 100 m, makes it dependent on parts with other pavement section.

2011:

13+0890 – 14+0150: IRI data are taken into account, *with caution*, to see IRI progression when no maintenance works are carried out, which seem to be realistic, according to data..

2016:

13+0890 – 14+0150: IRI data are taken into account, *with caution*, to see IRI progression when no maintenance works are carried out, which seem to be realistic, according to data. Only data from 13+0890 – 13+0990. The other section shows improvement and, hence, is discarded.

Skid resistance. SFC:

2011:

Data from 13+0890 – 14+0150 are introduced. They were collected in summer.

2016:

Data from 13+0890 – 14+0000 are introduced.

Skid resistance of surface layers in 2016
2016
14+0160 – 14+0720: PROJ-1385 (01/06/2005) LB2. 14+0720 – 16+0730: PROJ-761 (01/08/2010) LB2. 17+0420 – 17+0560: PROJ-1390 (01/01/2005) AC 16 surf S 5 cm 17+0560 – 18+0010: PROJ-762 (01/08/2005) LB2. 18+0540 – 18+0030: PROJ-762 (01/08/2005) LB2. 27+0950 – 29+0310: PROJ-1394 (01/07/2013) LB2.

Road Denomination	BI-2101	
Itinerary	Larrauri (BI-631) - Bakio	
Marker post (KP)	**From:** 20+0580	**To:** 25+0340
Transferred stretches		
-		

General comments:

The initial pavement structure is unknown. From test performed in 2000, the bituminous layer thickness varies from 3.5 cm to 16 along the stretch.

In 2003, project PROJ-282 is carried to reinforce the pavement structure, which was damaged due to the rainfalls of 2002. The total length of the road is rehabilitated, from 20+0580 to 25+0340. The length is divided in parts where 8 cm of asphalt concrete is extended and in other parts, 16 cm of asphalt concrete are applied. However, the total pavement structure is unknown

In 2012, a new roundabout in the beginning of the road, with the aim of improving the linking with the BI-631 is performed, but no complete new pavement structure is carried out.

In 2011, slurry seal is applied to the entire length.

Main actuations:

2003: PROJ-282 (01/06/2003)

- 20+0540 – 21+3000 (8): AC 16 surf S 4 cm + AC 16 base S 4 cm
- 21+0300 – 23+0030 (16): AC 16 surf S 4 cm + AC 16 bin S 4 cm + AC 32 base G 7 cm
- 23+0030 – 23+0430 (8): AC 16 surf S 4 cm + AC 16 base S 4 cm
- 23+0430 – 24+0430 (16): AC 16 surf S 4 cm + AC 16 bin S 5 cm + AC 32 base G 7 cm
- 24+0430 – 25+0150 (8): AC 16 surf S 4 cm + AC 16 base S 4 cm
- 25+0150 – 25+0340 (16): AC 16 surf S 4 cm + AC 16 bin S 5 cm + AC 32 base G 7 cm

2011: PROJ-1221 (01/07/2011)

- Slurry + Cracking repairs.

2012: PROJ-280 (01/06/2012)

- The total length is less than 100 m, so it is included as rehabilitation. Some parts of previous pavements have been reused. Difficult to state the real structure.

IRI:

2000:

- No data from 2000 have been used because the pavement structure is unknown.

2004:

- Pavement structure is rehabilitated with extension of different bituminous layers. However, the complete pavement structure is unknown.

2007:

- No complete pavement structure is known.

2011:

- Maintenance activities are carried out but the complete pavement structure is unknown.

2016:

- No complete pavement structure is known.

Skid resistance. SFC:

2011:

- No complete pavement structure is known.

2016:

- No complete pavement structure is known.

Skid resistance of surface layers in 2016

2016

- 20+0540 – 20+0640: PROJ-280 (01/06/2012) AC 16 surf S 6 cm.
- 20+0640 – 25+0340: PROJ-1221 (01/07/2011) LB2

Road Denomination	**BI-2120**	
Itinerary	Mungia - Plentzia	
Marker post (KP)	**From:** 15+0550	**To:** 26+0565
Transferred stretches		
-		

General comments:

The first registered in this project was PROJ-291, from 1984, which includes a rehabilitation work of previous layers and a new surface layer in the entire road. In the database, it is introduced as 5 cm of HMA surface layer + 5 cm HMA. Next project was under the Pavement Plan of 1990, and included the stretch 25+0000 – 26+0000.

A new outline was carried out as a result of the bypass of Mungia in 2000 (PROJ-28), for connecting with it, in 2000, involving stretch 15+0550 – 16+0100. Also in 2000, a new roundabout was performed in 25+0550 – 25+0750.

In 01/01/2012, 5 cm of S-12 are carried out in 15+0550 – 15+0680. In 01/08/2012, slurry (LB2) is extended in 15+0550 – 17+0210. In 30/05/2013, additional 6 cm of S-12 are added in 22+0800 – 23+0400 (PROJ-289).

In 30/09/2014, under PROJ-290, a new outline of stretch 19+01000 – 19+0860 is performed and a double carriageway stretch is constructed in stretch 18+0480 – 19+0090.

Main actuations:

1984: PROJ-291 (01/12/1984)

15+0550 – 25+0400: AC 16 surf S 5 cm + AC 32 base G 5 cm

1990: PROJ-264 (01/06/1990)

26+0000 – 26+0565: AC 16 surf S 5 cm + AC 32 base G 5 cm

2000: PROJ-333 (01/06/2000)

25+0550 – 25+0750: AC 22 surf S 5 cm + AC 32 base G 7 cm + ZA 25 cm + E1 60 cm

2000: PROJ-28 (01/07/2000)

15+0550 – 16+0100: AC 16 surf S 5 cm + AC 22 base G 11 cm + ZA 25 cm + E2 60 cm

2012: PROJ-1222 (01/01/2012)

15+0550 – 15+0680: AC 16 surf S 5 cm

2012: PROJ-518 (01/08/2012)

15+0550 – 17+0210: LB2

2013: PROJ-289 (30/05/2013)

22+0800 – 23+0400. AC 16 surf S 6 cm

2014: PROJ-290 (30/09/2014)

18+0530 – 19+0100: AC 16 surf S 6 cm + AC 22 bin S 6 cm + AC 32 base 8 cm + ZA 25 cm + E2 Double carriageway

19+0100 – 19+0300: AC 16 surf S 6 cm + AC 22 bin S 8 cm + AC 32 base S 8 cm + ZA 25 cm

19+0300 – 19+0860: AC 16 surf S 6 cm + AC 22 bin S 6 cm + AC 32 base S 8 cm + ZA 25 cm + E2

IRI:

2000:

No complete pavement structure is known.

2004:

Data from Bypass of Mungia (15+0550 – 16+0100) are used as New Outline.

Data from 25+0550 – 25+0750 are used as New Outline. IRI values are high, due to the roundabout and are discarded.

2007:

Data from Bypass of Mungia (15+0550 – 16+0100) are used as New Outline.

Data from 25+0550 – 25+0750 are not collected in 2007.

2011:

Data from Bypass of Mungia (15+0550 – 16+0100) are better than in 2007, which implies that works dated in 2012 were carried out previously. As works are dated in 2012, data from 2011 are not introduced in the calculation.

Data from 25+0550 – 25+0750 are better than in 2004, which implies that non recorded works were made. Hence, these data are not employed.

2016:	
	Data from 15+0555 – 16+0100, Bypass of Mungia, are introduced, as Maintenance, to see the slurry evolution. Stretch 15+0550 – 15+0680 had an extended layer AC 16 surf S previously to the slurry. Data from 18+0480 – 19+0100 are introduced as new outline Data from 22+0800 – 23+0400 are not used: a layer of 6 cm of AC 16 surf S is extended, but the previous pavement structure is unknown.
Skid resistance. SFC:	
2011:	
	Data from stretch 15+0555 – 16+0100, Bypass of Mungia, are not introduced as it was maintained in 2012 and no comparison is possible with data from 2016.
2016:	
	Data from 15+0555 – 16+0100, Bypass of Mungia, are introduced, as Maintenance, to see the slurry evolution. Data from 18+0480 – 19+0100 are introduced as New outline.
Skid resistance of surface layers in 2016	
2016	
	16+0100 – 17+0230: PROJ-518 (01/08/2012) LB2. 17+0230 – 17+0480: PROJ-288 (01/02/2010) AC 16 surf S 6 cm. 17+0480 – 17+0640: PROJ-1226 (01/06/2006) LB2. 20+0200 – 20+0370: PROJ-2021 (01/06/2005) LB2. 20+0370 – 21+0430: PROJ-1228 (01/02/2012) LB2.. 21+0430 – 22+0800: PROJ-1230 (01/06/2007) AC 16 surf S 5 cm. 22+0800 – 23+0400: PROJ-289 (30/05/2013) AC 22 surf S 6 cm. 23+0400 – 24+0890: PROJ-1231 (01/07/2011) LB2. 24+0890 – 26+0560: PROJ-1233 (01/07/2013) LB2.

Road Denomination	**BI-2121**	
Itinerary	Mungia - Gernika	
Marker post (KP)	**From:** 16+0135	**To:** 22+0000
	From: 22+0880	**To:** 25+0080
	From: 25+0660	**To:** 34+0360

Transferred stretches

22+0000 – 22+0880: Fruiz
25+0080 – 25+0660: Olabarri

General comments:

The first project on this road, PROJ-72, indicates that one layer of S-12 of 5 cm and another layer of S-12 of 5 cm were extended in 1985 over existing pavement (17+0310 – 28+0000).

In 1991, under the Pavement Plan, PROJ-275 indicates that in stretch 32+0500 – 34+0300, two layers of 5 cm each of HMA and a unbound material layer were extended.

There are some documented projects, PROJ-67, PROJ-68, PROJ-69, PROJ-70), from 1991, which were not performed.

As a consequence of the Bypass of Mungia, PROJ-28, a new outline is carried out in 16+0135 – 17+0100, finished in 01/07/2000.

During the construction of the Bypass of Gernika, PROJ-30 (01/11/2003), a new outline was carried out in stretch 28+0640 – 29+0070.

PROJ-66, documented in 2001 was performed in 2011, but it is located in a transferred part (25+0100 – 25+0600), so no data are available.

In 2013, PROJ-292 is carried out in 18+0600 – 20+0200, when a total new pavement structure is constructed in 18+0900 – 19+0320.

From 2007, many rehabilitation and maintenance works are recorded, especially in the stretches where no works were recorded since 1985 or 1991.

Main actuations:

1985: PROJ-78 (18/12/1985)
 17+0310 – 28+0000: AC 16 surf S 5 cm + S12 cal 5 cm
1991: PROJ-275 (01/06/1991)
 32+0500 – 34+0300: AC 16 surf S 5 cm + S12 cal 5 cm
2000: PROJ-28 (01/07/2000)
 16+0135 – 17+0100: AC 16 surf S 5 cm + AC 22 base G 7 cm + MG (ZA) 20 cm
2003: PROJ-30 (01/11/2003)
 28+0640 – 29+0070: AC 16 surf S 4 cm + S12 4 cm + GE 22 cm
2007: PROJ-1254 (01/02/2007)
 28+0160 – 28+0640: LB2
2011: PROJ-66 (01/06/2011)
 25+0100 – 25+0600: Ac 22 surf S 5 cm + AC 22 base G 10 cm + MG (ZA) 25 cm
2012: PROJ-1235 (01/01/2012)
 16+0140 – 17+0380: LB2 + Patching
2013: PROJ-292 (30/05/2013)
 18+0600 – 18+0900 and 19+0320 – 20+0200: S-20 6 cm added to existing pavement
 18+0900 – 19+0320: AC 22 surf S 6 cm + AC Bin 32 S 11 cm + MG (ZA) 25 cm + E2

IRI:

2000:

Only two projects are known before 2000 (PROJ-78 and PROJ-275), in 1984 and in 1991 but no complete pavement section is known. PROJ-28 was performed after data collection.

2004:

There are no data from stretch of the Bypass of Mungia (16+0135 – 17+0100).

2007:

Data from Bypass of Mungia (16+0135 – 17+0100) are available, introduced as New Outline.

Some values in stretch 28+0400 – 29+0000 seem to be very high and those data are discarded as spurious. The rest are introduced.

2011:	
	Data from Bypass of Mungia (16+0135 – 17+0100) are introduced as New Outline. Two data were better and were discarded. Values in stretch 28+0400 – 29+0000 seem reasonable and are introduced.
2016:	
	In the section from Bypass of Mungia (16+0135 – 17+0100) a maintenance operation was performed in 2012 (PROJ-1235). Data are introduced as Maintenance. Data from stretch 18+0900 – 19+0320 (2013) are introduced as New Outline. The other stretches of the project are not introduced as all the pavement structure is unknown. Values in stretch 28+0400 – 29+0000 are higher than in 2011 and are introduced
Skid resistance. SFC:	
2011:	
	Data from Bypass of Mungia are not employed, as values from 2011 cannot be compared to those of 2016, as works were performed in possible stretches. Data from stretch 28+0400 – 29+0000 are introduced.
2016:	
	The data from Bypass of Mungia (16+0135 – 17+0100) after a maintenance operation were introduced Data from stretch 18+0900 – 19+0320 (2013) are introduced as new outline. Data from stretch 28+0400 – 29+0000 are introduced as new outline.
Skid resistance of surface layers in 2016	
2016	
	17+0100 – 17+0380: PROJ-1235 (01/06/2012). LB2. 17+0380 – 18+0720: PROJ-1237 (13/07/2013). LB2. 18+0720 – 18+0900: PROJ-292 (30/05/2013). AC 22 surf S 6 cm. 19+0320 – 20+0200: PROJ-292 (30/05/2013) AC 22 surf S 6 cm. 20+0200 – 20+0490: PROJ-1240 (01/07/2011) LB2. 20+0490 – 21+0360: PROJ-1242 (01/06/2007) LB2. 21+0360 – 21+0510: PROJ-1243 (01/07/2015) LB2. 21+0510 – 21+0930: PROJ-1244 (01/07/2011) LB2. 22+0880 – 23+0730: PROJ-1242 (01/06/2007) LB2. 23+0730 – 24+0120. PROJ-1246 (01/02/2011) LB2. 24+0120 – 24+0930: PROJ-1247 (01/06/2007) LB2. 26+0080 – 26+0270: PROJ-1253 (01/06/2007) LB2. 26+0270 – 27+0010: PROJ-1778 (30/03/2015) AC 16 surf S 5 cm 27+0010 – 27+0060: PROJ-1253 (01/06/2007) LB2. 27+0060 – 27+0400: PROJ-1778 (30/03/2015) AC 16 surf S 5 cm. 27+0400 – 28+0150. PROJ-1946 (01/06/2016) AC 16 surf S 5 cm. 28+0150 – 28+0380: PROJ-1780 (30/03/2015) AC 16 surf S 5 cm. 28+0380 – 28+0490: PROJ-1946 (01/06/2016) AC 16 surf S 5 cm. 28+0490 – 28+0640: PROJ-1254 (01/02/2007) LB2. 29+0070 – 29+0410: PROJ-1258 (01/06/2012) LB2. 29+0410 – 29+0720: PROJ-1259 (01/06/2007) LB2. 30+0000 – 30+0290. PROJ-1946 (01/06/2016) AC 16 surf S 5 cm. 30+0290 – 31+0140: PROJ-1259 (01/06/2007) LB2. 31+0140 – 32+0420: PROJ-1947 (01/12/2016) AC 16 surf S 5 cm 32+0420 – 32+0720: PROJ-1259 (01/06/2007) LB2. 32+0720 – 34+0290: PROJ-1260 (01/07/2011) LB2. 34+0290 – 34+0300: PROJ-1948 (01/03/2016) AC 16 surf S 5 cm. 34+0330 – 34+0360: PROJ-1262 (01/02/2010) S-12 5 cm.

Road Denomination	BI-2122	
Itinerary	Sopela - Plentzia	
Marker post (KP)	**From:** 21+0070	**To:** 27+0090
Transferred stretches		
-		

General comments:

There is a project, PROJ-181, finished in 1995, which rebuilt the stretch 21+0000 – 24+0700 with two layers of hot mix asphalts (6 cm of S-20 and 7 cm of G-25) and a layer of slag and aggregates. However, samples taken in 2000 showed that there were sections with only unbound material and no slag. Therefore, in some parts (unknown), previously existing pavement was reused. The project is recorded in the dataset of the PMS, but it is not reflected as initial pavement and the complete pavement structure is not indicated.

In 2010, project PROJ-293 was carried out, which implied the introduction of two roundabouts in 24+0260 – 24+0650 and 26+0900 – 27+0230, where a new complete pavement section is carried out in the new outline of the roundabouts. These sections are used for data analysis as new outline. In the previous and posterior sections of the roundabout existing layers were employed.

There are maintenance works registered from 2007.

Main actuations:

1995: PROJ-181 (01/06/1995)

21+0000 – 24+0700: S-20 6 cm + G-25 7 cm + ¿GE/ZA 25 cm?

2010: PROJ-293 (01/12/2010)

24+0260 – 24+0650: AC 16 surf S 5 cm+ AC 22 bin S 6 cm + AC 22 base G 9 cm + ZA 25 cm + E2 in new stretch (24+0500 – 24+0600, roundabout), in the other, reuse of existing pavements with AC 16 surf S in surface layer.

26+0900 – 27+0230: AC 16 surf S 5 cm+ AC 22 bin S 6 cm + AC 22 base G 9 cm + ZA 25 cm + E2 in new stretch (27+0000 – 27+0080, roundabout), in the other, reuse of existing pavements with AC 16 surf S in surface layer.

2010: PROJ-1265 (01/06/2010)

21+0290 – 21+0800: LB2

2007: PROJ-1266 (01/06/2007)

21+0800 – 22+0100: LB2

2010: PROJ-1268 (01/01/2010)

24+0680 – 26+0880: LB2

2007: PROJ-2022 (31/08/2007)

22+0100 – 23+0490: LB2

2010: PROJ-2024 (01/06/2010)

23+0490 – 26+0600: LB2

2010: PROJ-2026 (01/06/2010)

26+0600 – 26+0900: LB2

IRI:

2000:

No stretch with complete pavement structure is known.

2004:

No stretch with complete pavement structure is known.

2007:

No stretch with complete pavement structure is known.

2011:

Data from stretch 24+0500 – 24+0600 is very high, IRI = 4,00 after 1 year and are discarded as spurious data. Effect of the roundabout impacts the IRI values

There is not data for stretch 27+0000 – 27+0080.

2016:

Data from stretches 24+0500 – 24+0600 is reasonable (lower than in 2011) and is introduced as new outline.

Data from stretch 27+0000 – 27+0080 are also introduced.

Skid resistance. SFC:
2011:
Stretches 24+0500 – 24+0600 and 27+0000 – 27+0080 are introduced. Data was collected in summer.
2016:
Stretches 24+0500 – 24+0600 and 27+0000 – 27+0080 are introduced.
Skid resistance of surface layers in 2016:
2016:
21+0070 – 21+0190: PROJ-1346 (01/06/2009). S-12 5 cm 21+0290 – 21+0800: PROJ-1265 (01/06/2010). LB2 21+0800 – 22+0100: PROJ-1266 (01/06/2007). LB2 22+0100 – 23+0490: PROJ-2022 (31/08/2007). LB2 23+0490 – 24+0260. PROJ-2024 (01/06/2010). LB2 24+0700 – 26+0600. PROJ-2024 (01/06/2010). LB2 26+0600 – 26+0900: PROJ-2026 (01/06/2011). LB2

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-2153</td></tr>
<tr><td>Itinerary</td><td colspan="2">Andraka - Armintza</td></tr>
<tr><td rowspan="2">Marker post (KP)</td><td>From: 23+0380</td><td>To: 24+0760</td></tr>
<tr><td>From: 25+0730</td><td>To: 27+0000</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">24+0760 – 25+0730: Lemoiz</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">The first document project, PROJ-294, from 1985 was not conducted. Later, in 1988, PROJ-295 was carried out, which implied the extension of 2 bituminous layers, 5 cm of AC 16 surf Sand 5 cm of G20, over the existing pavement, in stretch 23+0530 – 26+0100.
In 1990, under the Pavement Plan, PROJ-296 was performed, which supposed the extension of 2 bituminous layers of 5 cm over existing pavement in stretch 26+0100 – 27+0000.
In 2013, related to the improvement of the BI-2120 road, the intersection with BI-2120 was resolved with a roundabout and a new outline was constructed in 23+0380 – 23+0530.
There have been two maintenance works, in 2007 and in 2011.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">1988: PROJ-295 (01/06/1990)
23+0530 – 26+0100: AC 16 surf S 5 cm + G20 6 cm
1990: PROY-296 (01/06/1990)
26+0100 – 27+0000: HMA 5 cm + HMA 5 cm
2007: PROJ-1272 (01/06/2007)
26+0600 – 27+0000: LB2.
2011: PROJ-561 (01/08/2011)
23+0600 – 26+06000: LB2.
2013: PROJ-289 (30/05/2013)
23+0380 – 23+0530: AC 22 surf S 6 cm + AC 22 bin S 6 cm + AC 32 base S 8 cm + MG (ZA) 25 cm + E2
23+0530 – 23+0600: S-20 6 cm</td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">2000:</td></tr>
<tr><td colspan="3">There is no stretch where all the pavement structure is known.</td></tr>
<tr><td colspan="3">2004:</td></tr>
<tr><td colspan="3">There is no stretch where all the pavement structure is known.</td></tr>
<tr><td colspan="3">2007:</td></tr>
<tr><td colspan="3">There is no stretch where all the pavement structure is known.</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">There is no stretch where all the pavement structure is known.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">Stretch 23+0380 – 23+0530 is introduced in the calculation as New Outline.</td></tr>
<tr><td colspan="3">Skid resistance. SFC:</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">There is no stretch where all the pavement structure is known.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">Stretch 23+0380 – 23+0530 is introduced in the calculation as New Outline.</td></tr>
<tr><td colspan="3">Skid resistance of surface layers in 2016</td></tr>
<tr><td colspan="3">2016</td></tr>
<tr><td colspan="3">23+0530 – 23+0600: PROJ-289 (30/05/2013) S-20 6 cm
23+0600 – 26+0600: PROJ-561 (01/08/2011) LB2.
26+0600 – 27+0000: PROJ-1272 (01/06/2007) LB2.</td></tr>
</table>

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-2235</td></tr>
<tr><td>Itinerary</td><td colspan="2">Gernika - Bermeo</td></tr>
<tr><td>Marker post (KP)</td><td>From: 37+0100</td><td>To: 50+0310</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">-</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">It is a road with consecutive rehabilitation and maintenance works.
In 1985, the Bypass of Mundaka, PROJ-328 (47+0600 – 49+0000) was carried out.
The first rehabilitation activities were:
PROJ-81: 39+0000 – 50+0400 (01/06/1990) LB2 ad AC 22 surf D in different stretches
PROJ-91: 37+0100 – 39+0000 (01/06/1992) AC 16 surf S 5 cm and repairs
Some intersections were transformed into roundabouts:
PROJ-325: 45+0300 – 45+0400 (01/06/2008)
PROJ-326: 44+0250 – 44+0350 (01/06/2009)
Other some new outlines:
PROJ-1776: 49+0970 – 49+1040 (01/05/2015)
And some important maintenance works:
PROJ-766: (37+0010 – 37+0110 (01/08/2009) AC 16 surf S
PROJ-1275: 37+0110 – 39+0470 (08/08/2013) LB2
PROJ-1276: 39+0470 – 44+0238 (01/02/2007) S-12 5 cm
PROJ-1278: 44+0238 – 48+0190 (01/07/2014) LB2
.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">1985: PROJ-328 (01/06/1985)
47+0600 – 49+0000: AC 16 surf S 5 cm + AC 22 bin S 7 cm + MG (ZA) 55 cm.
1990: PROJ-81 (01/06/1990)
39+0000 – 50+0400: LB2 and AC 22 surf D 7 cm in different stretches
1992: PROJ-91 (01/06/1992)
37+0100 – 39+0000: AC 16 surf S 5 cm and repairmen of depressions
2002: PROJ-89 (01/06/2004)
37+0010 – 47+0060: BBTM 11 A 3 cm + HMA 5 cm (A variaty of reinforcement in the stretch, it is supposed a continuos reinforcement of 8 cm.
2008: PROJ-325 (01/06/2008)
45+0300 – 45+0400: BBTM 11A 3 cm + AC 22 bin S 7 cm+ AC 22 base G 15 cm + ZA 25 cm
2009: PROJ-326 (01/06/2009)
44+0250 – 44+0350: AC 16 surf S 5 cm + AC 22 bin S 9 cm+ AC 22 base G 11 cm + MG (ZA) 25 cm
2009: PROJ-766 (01/08/2009)
37+0010 – 37+0110: AC 16 surf S
2012: PROJ-1279 (01/01/2012)
47+0030 – 49+0270: Superficial treatment and patching
2014: PROJ-1278 (01/07/2014)
44+0230 – 48+0190: LB2
2015: PROJ-1776 (01/05/2015)
49+970 – 49+1040: BBTM 11A 3 cm + S-12 8 cm + GC 29 cm + SEST 25 cm</td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">2000:</td></tr>
<tr><td colspan="3">The pavement structure of the Bypass of Mundaka (47+0600 – 49+000) is known and introduced in the calculation. The average IRI value is 3,03; after 15 years.</td></tr>
<tr><td colspan="3">2004:</td></tr>
<tr><td colspan="3">No data were collected.</td></tr>
<tr><td colspan="3">2007:</td></tr>
<tr><td colspan="3">The pavement structure of the Bypass of Mundaka (47+0600 – 49+000) is known and introduced in the calculation. The average IRI value is 3,31; after 22years.</td></tr>
</table>

2011:	
	The pavement structure of the Bypass of Mundaka (47+0600 – 49+000) is known and introduced in the calculation. The average IRI value is 3,34; after 26 years. There are some new stretch with known pavement structure: Stretch 44+0250 – 44+0350. (01/06/2009). IRIm: 5,99. After 2 years. Data are considered spurious. Effect of the roundabout is recorded. Stretch 45+0300 – 45+0400 (01/06/2008). Mean IRI: 3,49, after 3 years. Data are discarded as spurious. Effect of the roundabout is recorded.
2016:	
	The stretch of the Bypass of Mundaka (47+0600 – 49+000) received a superficial treatment and patching was carried out in 2012 and in 2014 LB2 was partially extended (47+0600 – 48+0190). It is included in the calculation as maintenance. New outline in stretch 49+0970 – 49+1040, from 2015. Mean IRI: 4,54. It seems to be due to a bridge joint in the section. Discarded.
Skid resistance. SFC:	
2011:	
	Data from 2011 in all the stretches where pavement section is known cannot be compared due to maintenance works between 2011 and 2016 in those stretches.
2016:	
	Analyzed stretch with complete pavement structure: 44+0250 – 44+0350; 45+0300 – 45+0400; 47+0600 – 48+0850 and 49+0970 – 49+1040.
Skid resistance of surface layers in 2016	
2016	
	37+0010 – 37+0110: PROJ-776 (01/08/2009) AC 16 surf S 5 cm 37+0110 – 39+0480: PROJ-1275 (08/08/2013) LB2. 39+0480 – 44+0230: PROJ-1276 (01/02/2007) AC 16 surf S 5 cm. 44+0230 – 44+0250: PROJ-1278 (01/07/2014) LB2 44+0350 – 45+0300: PROJ-1278 (01/07/2014) LB2. 45+0400 – 47+0600: PROJ-1278 (01/07/2014) LB2. 49+0140 – 49+0620: PROJ-1280 (01/02/2008) Ac 16 surf S 5 cm.

Road Denomination	BI-2237	
Itinerary	Muruetagane – Port of Elantxobe	
Marker post (KP)	**From:** 39+0600	**To:** 43+0120
	From: 43+0790	**To:** 45+0200
Transferred stretches		
43+0120 – 43+0790: Ibarrangelu		

General comments:
The first projects documented in this road are PROJ-297 (39+0600 – 42+0000) and PROJ-298 (42+0000 – 43+0200), in 1986. The first one indicates that a new pavement structure was carried out: AC 16 surf S 5 cm, AC 22 bin S + MG (ZA) 20 cm. In the second one only bituminous layers were extended (AC 16 surf S 5 cm + G20 7 cm) and existing pavement was used. The first one is included as new pavement structure in the Pavement Management System and the second one not. In 1990, PROJ-93 (43+0790 – 44+0000) is carried out, indicating that 2 bituminous layers were extended, with a total thickness of 12 cm. It is not introduced as new pavement structure in the PMS. In 2008 the intersection with road BI-2238, in the beginning of the road is carried out, constructing a new roundabout, but only borders are widened and only a new surface layer is extended. It only affects 40 m of the road. There are maintenance works in 2009, 2011 and 2015.
Main actuations:
1986: PROJ-297 (01/06/1986) 39+0600 – 42+0000: AC 16 surf S 5 cm + AC 22 bin S 7 cm + MG (ZA) 20 cm 1986: PROJ-298 (01/06/1986) 42+0000 – 43+0200: AC 16 surf S 5 cm + G20 7 cm 1999: PROJ-93 (01/06/1999) 43+0200 – 44+0300: AC 16 surf S 5 cm + AC 22 bin S 7 cm 2008: PROJ-92 (01/06/2008) 39+0600 – 39+0640: Ac 16 surf S 5 cm 2009: PROJ-1282 (01/01/2009) 39+0690 – 44+0890: Slurry + Patching 2009: PROJ-1283 (01/06/2009) 40+0470 – 43+0120: LB2 2011: PROJ-1285 (01/06/2011) 44+0850 – 45+0180: AC 16 surf S 2014: PROJ-1284 (01/07/2014) 44+0100 – 44+0900: Superficial treatment + Patching 2014: PROJ-574 (01/08/2014) 44+0900 – 45+0200: Superficial treatment 2015: PROJ-1281 (01/07/2015) 39+0640 – 42+0800: LB2

IRI:
2000:
The only stretch where a complete pavement section is known is 39+0600 – 42+000. After 14 years, the stretch has an average IRI value of 2,23.
2004:
Only data from 44+0120 onwards are available, where the pavement structure is unknown.
2007:
Data from stretch 39+0600 – 42+000 has an average IRI value of 2,48
2011:
There is a maintenance work in 2009 (PROJ-1282 and PROJ-1283) in the entire stretch 39+0600 – 42+000. LB2 was extended. Therefore, IRI data from this stretch are introduced as maintenance.

2016:
There is a maintenance work in 2015 (PROJ-1281) in the entire stretch 39+0600 – 42+000. LB2 has been extended. Therefore, IRI data from this stretch are introduced as new maintenance
Skid resistance. SFC:
2011:
Data from 2011 and 2016 cannot be compared as there is a maintenance work in 2015.
2016:
Data from 39+0600 – 42+0000 is introduced, the complete pavement section is known.
Skid resistance of surface layers in 2016
2016
39+0600 – 39+0640: PROJ-92 (01/06/2008) AC 16 surf s 5 cm. 39+0640 – 39+0690: PROJ-1281 (01/07/2015) LB2. 42+0000 – 42+0800: PROJ-1281 (01/07/2015) LB2. 42+0800 – 43+0120: PROJ-1283 (01/06/2009) LB2. 43+0790 – 44+0300: PROJ-93 (01/06/1999) AC 16 surf S 5 cm 44+0850 – 45+0180: PROJ-1285 (01/06/2011) AC 16 surf S 5 cm.

Road Denomination	**BI-2238**	
Itinerary	Gernika - Lekeitio	
Marker post (KP)	**From:** 31+0830	**To:** 32+0700
	From: 32+0780	**To:** 53+0320
Transferred stretches		
32+0700 – 32+0780: Gernika		

General comments:

It is a long road with very few documented projects.

The first documented project, in 1992, PROJ-96 (32+0900 – 35+0500), establishes a complete pavement section, but specimens taken in 2000 do not corroborate it. In the PMS the bituminous layers are not introduced as initial pavement.

In 1992, project PROJ-95 (36+0650 – 39+0700) is carried out, where a new complete section is performed. In the PMS, the section is introduced as initial pavement.

In 2009, the second part of the Bypass of Gernika is carried out, PROJ-4 (01/06/2009) with a new outline. In 2009, the Bypass of Lekeitio (PROJ-3, 51+0700 – 53+0330) is finished, with a new outline.

In the vast majority of the road, the pavement structure is unknown. Only recorded maintenance works from 2007 are the unique knowledge about some sections. Specimen tests showed a variety of pavement structures, ranging from some narrow bituminous layers to other with more than 30 cm of HMA. There are even some stretches where no information is available.

Main actuations:

1992: PROJ-96 (01/06/1992)
32+0900 – 35+0500: AC 16 surf S 5 cm + G-20 7 cm

1992: PROJ-95 (01/06/1992)
36+0650 – 39+0700: AC 16 surf S 5 cm + G-20 7 cm + ZA 20 cm

2009: PROJ-1291 (01/06/2009)
36+0810 – 37+0910: LB2

2007: PROJ-1292 (01/02/2007)
37+0910 – 38+0080: AC 16 surf S

2009: PROJ-1293 (01/06/2009)
38+0080 – 38+0420: LB2

2009: PROJ-1294 (01/06/2009)
39+0000 – 39+0720: LB2

2009: PROJ-4 (01/06/2009)
31+0830 – 32+0630: AC 16 surf S 6 cm + AC 22 bin S 8 cm + GE 30 cm

2009: PROJ-3 (23/10/2009)
51+0300 – 53+0330: AC 16 surf S 5 cm + AC 22 bin S 8 cm + GC 30 cm

IRI:

2000:

Stretch 36+0650 – 39+0700, after 8 years IRIm = 3,74. It is introduced in the analysis.

2004:

Stretch 36+0650 – 39+0700, after 12 years, IRIm = 3,39. Hence, these data are discarded as spurious.

2007:

There are not data about stretch 36+0650 – 39+0700.

2011:

In the 31+0830 – 32+0630, in the first part (31+0830 – 32+0120) there is a bridge, and its joints determine the value. Hence, data from this first stretch are discarded.

Stretch 32+0120 – 32+0630 is introduced as new outline (IRI_m = 2,26)

Stretch 51+0300 – 53+0330 is introduced as new outline (IRI_m = 2,05)

The stretch 36+0680 – 39+0700 is divided according to performed activities:

- 36+0650 – 36+0810: Initial pavement section. IRI_m = 4,52
- 36+0810 – 37+0910: Initial pavement section + LB2 (2009). IRI_m = 3,04
- 37+0910 – 38+0080: Initial pavement section + AC 16 surf S 5 cm (2007). IRI_m = 4,4
- 38+0080 – 38+0420: Initial pavement section + LB2 (2009). IRI_m =2,80
- 38+0420 – 38+1020: Initial pavement section. IRI_m =3,46
- 39+0000 – 39+0700: Initial pavement section + LB2 (2009). IRI_m = 3,53

2016:
In stretch 32+0120 – 32+0630, slurry was extended in 30/07/2013. Therefore, IRI values are observed as maintenance work (IRI_m = 2,22) Stretch 51+0300 – 53+0330 is introduced as new outline (IRI_m = 2,19). The stretch 36+0680 – 39+0700 is divided according to performed activities: • 36+0650 – 36+0810: Initial pavement section. IRI_m = 4,92 • 36+0810 – 37+0910: Initial pavement section + LB2 (2009). IRI_m = 3,32 • 37+0910 – 38+0080: Initial pavement section + AC 16 surf S 5 cm (2007). IRI_m = 4,5 • 38+0080 – 38+0420: Initial pavement section + LB2 (2009). IRI_m = 3,33 • 38+0420 – 38+1020: Initial pavement section. IRI_m = 3,83 • 39+0000 – 39+0700: Initial pavement section + LB2 (2009). IRI_m = 3,90 As observed, all sections have worse IRI values than in 2011.
Skid resistance. SFC:
2011:
Data from stretch 31+0830 – 32+0630 cannot be compared to those from 2016, as slurry was extended over it in 2013. Data from stretch 51+0300 – 53+0330 is introduced as new outline, although only 2 years old. Data from the stretch 36+0680 – 39+0700 are classified according to previously divided stretches, identifying each pavement section.
2016:
Stretch 31+0830 – 32+0630 is introduced as maintenance activity (in 2013). Stretch 51+0300 – 53+0330 is introduced as new outline. Data from the stretch 36+0680 – 39+0700 are classified according to previously divided stretches, identifying each pavement section.
Skid resistance of surface layers in 2016
2016
32+0630 – 33+0280: PROJ-575 (30/07/2013) LB2. 33+0280 – 34+0440: PROJ-1228 (01/06/2009) LB2. 34+0440 – 34+0920: PROJ-96 (01/06/1992) AC 16 surf S 5 cm 34+0920 – 35+0160: PROJ-97 (01/02/2007) BBTM 11A 4 cm 35+0160 – 35+0460: PROJ-2014 (01/06/2009) LB2. 35+0460 – 35+0500: PROJ-96 (01/06/1992) AC 16 surf S 5 cm 39+0700 – 39+0720: PROJ-1294 (01/06/2009) LB2. 39+0830 – 40+0990: PROJ-1296 (01/07/2014) LB2. 41+0950 – 43+0150: PROJ-1297 (01/06/2009) LB2. 43+0370 – 44+1020: PROJ-2013 (01/06/2009) LB2. 45+0000 – 47+0610: PROJ-1298 (01/07/2014) LB2. 48+0980 – 50+0630: PROJ-1300 (01/07/2013) LB2. 51+0330 – 51+0700: PROJ-1765 (01/07/2013) LB2.

Road Denomination	**BI-2704**	
Itinerary	Asua - Plentzia	
Marker post (KP)	**From:** 7+0980	**To:** 19+0790
	From: 20+1450	**To:** 24+0200

Transferred stretches

19+0790 – 20+1450: Urduliz

General comments:

The first and final parts of the road are well documented. At the central part of the road, there are some stretches without projects, only recent maintenance works.

The first document recorded is from 1984, in the BOPV (PROJ-302) (without project), which indicates that a regularization bituminous layer (±5 cm) and another one (5 cm) are extended (9+0400 – 14+0000 and 20+0600 – 24+0400).

In the initial part, the corridor Unbe Part IV (PROJ-300) was performed in 1993, extending a new layer over existing one in 7+0970 – 8+0170 and constructing a new pavement structure in 8+0170 – 9+0230. In 1994, the corridor of Unbe part V (PROJ-299) was finished, implying a new complete pavement in stretch 9+0200 – 13+0000. Consequently, stretch from 8+0170 to 13+0000 is introduced as new outline in the PMS. In 1994, the roundabout of Asua (PROJ-301) was carried out, 8+000 – 8+0170, reusing partially existing pavement.

In 2012, the crossway of Arkalanda, with BI-2731 (PROJ-305) was remodelled, constructing additional lanes, and not affecting to existing pavement. In March 2014, maintenance project PROJ-347 is carried out, removing 3 cm of existing pavement and extending BBTM 11A.

In the final part, the bridge of Plentzia (PROJ-346) is built in 1986, affecting the stretch 23+0300 – 24+0200 with new pavement. In 2009, the intersection with BI-2122 was remodelled, adding 5 cm of AC 16 surf S to existing pavement in stretch 23+0350 – 23+0890 (PROJ-293).

Main actuations:

1984: PROJ-302 (01/06/19484)

9+0400 – 14+0000 and 20+0600 – 24+0400: AC 16 surf S 5 cm + S12 5 cm

1986: PROJ-346 Bridge of Plentzia (01/06/1986)

23+0300 – 23+0700 and 23+0900 – 24+0200: AC 16 surf S 4 cm + G12 4 cm + MG (ZA) 20 cm

23+0700 – 23+0900 Bridge: AC 16 surf S 4 cm + G12 4 cm + Structure (concrete).

1993: PROJ-300 (01/05/1993)

7+0970 – 8+0170: AC 16 surf S 5 cm

8+0170 – 9+0230: PA 11 4 cm + S12 3 cm + AC 32 base G 7 cm + GE 25 cm

1994: PROJ-299 (01/06/1994)

9+0200 – 13+0000: PA 11 3 cm + S12/G20 3+7 cm + AC 32 base G 7 cm + MG (ZA) 20 cm

1994: PROJ-301 (01/12/1994)

7+0970 – 8+0170: AC 16 surf S 6 cm

2008: PROJ-1319 (01/06/2008)

8+0500 – 9+0300: LB2

2008: PROJ-1318 (01/01/2008)

8+0010 – 10+0230: Crack repairing

2010: PROJ-293 (01/12/2010)

23+0350 – 23+089: AC 16 surf S 5 cm

2012: PROJ-305 (28/09/2012)

11+0100 – 11+0770: New lanes, BBTM 11A 3 cm + Ac 22 bin S 7 cm + AC 25 base S 10 cm + MG (ZA) 25 cm

2014: PROJ-347 (30/03/2014)

7+0970 – 8+0500: Remove 3 cm + BBTM 11A 3 cm

IRI:

2000:

8+0170 – 9+0230: New outline, from 1993. IRI_m = 1,43.

9+0230 – 13+000: New outline, from 1994. IRI_m = 1,50.

23+0300 – 23+0700: New outline, from 1986. IRI_m = 2,92. Data only from stretch 23+0300 – 23+0700 are considered as bridge joints affect roughness enormously and are repaired independently. Bridge is located in stretch 23+0700 – 23+0900. Data from 23+0900 – 24+0200 are not complete and inconsistent.

2004:
8+0170 – 9+0230: New outline, from 1993. IRI_m = 1,64. 9+0230 – 13+000: New outline, from 1994. IRI_m = 1,52. 23+0300 – 23+0700: New outline, from 1986. IRI_m = 3,02.
2007:
8+0170 – 9+0230: New outline, from 1993. IRI_m = 1,96. 9+0230 – 13+000: New outline, from 1994. IRI_m = 1,50 ?? Better than in 2004. The only work in this area is PROJ-1318, where crack repairing and patching were carried out in 8+0010 – 10+0230. Global and local improvements are observed in all the stretch. Even data from 10+0230 – 12+0000 are better. Data are rejected as spurious. 23+0300 – 23+0700: New outline, from 1986. IRI_m = 3,58.
2011:
8+0500 – 9+0300: Maintenance work (LB2) in 2008. IRI_m = 1,88. (Stretch 8+0170 – 8+0500 shows similar IRI values (1,28) and, hence, these data are discarded as spurious. Patching activity is performed also in 2008 in this stretch. 23+0400 – 23+0700: Maintenance in 2010 (AC 16 surf S 5 cm). IRI_m = 1,77.
2016:
8+0500 – 9+0300: Maintenance work (LB2) in 2008. IRI_m = 2,12. 23+0400 – 23+0700: Rehabilitation in 2010 (AC 16 surf S cm). IRI_m = 2,14.
Skid resistance. SFC:
2011:
Data were collected in summer (04/08/2011) 8+0500 – 9+0300: Maintenance work (LB2) in 2008 23+0400 – 23+0700: Rehabilitation in 2010 (S-12 5 cm).
2016:
8+0500 – 9+0300: Maintenance work (F-10 3 cm) in 2014. 23+0400 – 23+0700: Rehabilitation in 2010 (S-12 5 cm).
Skid resistance of surface layers in 2016
2016
8+0000 – 8+0500: PROJ-347 (30/03/2014) BBTM 11A 3 cm. 13+0120 – 13+0680: PROJ-348 (01/05/2014) AC 16 surf S 5 cm 13+0680 – 13+0900: PROJ-1305 (01/06/2008) LB2. 14+0300 – 14+0820: PROJ-1303 (01/02/2014) AC 16 surf S 5 cm. 14+0820 – 15+0400: PROJ-1311 (01/06/2008) LB2. 15+0500 – 16+0250: PROJ-1311 (01/06/2008) LB2. 17+0050 – 18+0900: PROJ-1311 (01/06/2008) LB2. 16+0250 – 17+0650: PROJ-1310 (01/06/2012) LB2. 18+0900 – 19+0660: PROJ-1312 (01/06/2012) LB2. 19+0660 – 19+0800: PROJ-608 (01/08/2008) LB2. 20+1450 – 20+2040: PROJ-1315 (01/07/2014) LB2. 22+0000 – 23+0350: PROJ-1315 (01/07/2014) LB2.

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-2713</td></tr>
<tr><td>Itinerary</td><td colspan="2">Larrabetzu – Mountain pass of Gerekiz</td></tr>
<tr><td rowspan="2">Marker post (KP)</td><td>From: 12+0180</td><td>To: 21+0500</td></tr>
<tr><td>From: 22+0120</td><td>To: 24+0080</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">21+0500 – 22+0120: Andra Mari</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">The first documented project is under the Pavement Plan of 1990, PROJ-306, and included the extension of 2 bituminous layers (5 cm + 5cm).
In 1999, the stretch 14+0400 – 16+0960 is widened, adding a new pavement structure at the right border of each lane (PROJ-345).
In 2005 it is finished the Bypass of Larrabetzu (PROJ-8), which consists on a new outline (12+0180 – 14+0400) and rehabilitation of the stretch 14+0560 – 16+0960, with BBTM 11A 3 cm and S-20, from 6 to 18 cm.
In 2003, connected to the Bypass of Gernika, a new pavement structure is carried out in 24+0000 – 24+0080.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">1990: PROJ-306 (01/06/1990)
14+0400 – 24+0140: AC 16 surf S 5 cm + S-12 5 cm
1999: PROJ-345 (01/06/1999)
14+0400 – 16+0960: Remove 7 cm + AC 16 surf S 7 cm
AC 16 surf S 7 cm + G-20 11 cm + ZA 25 cm (in borders)
2003: PROJ-27 (01/11/2003)
24+0000 – 24+0080: S-12 4 cm + S-12 4 cm + GE 22 cm
2005: PROJ-8 (03/08/2005)
12+0180 – 14+0400: BBTM 11A 3 cm + AC 22 bin S 12 cm + GC 30 cm
14+0560 – 16+0960: BBTM 11A 3 cm + AC 22 bin S 6 cm</td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">2000:</td></tr>
<tr><td colspan="3">There is not section where all the pavement structure is known.</td></tr>
<tr><td colspan="3">2004:</td></tr>
<tr><td colspan="3">There are not data.</td></tr>
<tr><td colspan="3">2007:</td></tr>
<tr><td colspan="3">Stretch 12+0300 – 14+0400 is introduced as new outline. IRIm = 1,80
There are not data for stretch 24+0000 – 24+0080.</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">12+0300 – 14+0400: There are global and individual improvements in IRI values, although no works are recorded. Hence, data are discarded. IRIm = 1,65
There are not data for stretch 24+0000 – 24+0080.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">12+0300 – 14+0400: IRI values are better than in 2007 (and worse than in 2011), which indicates that a maintenance work was carried out and not recorded. IRIm = 1,72.
There are not data for stretch 24+0000 – 24+0080.</td></tr>
<tr><td colspan="3">Skid resistance. SFC:</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">Stretch 24+0000 – 24+0080 is employed.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">Stretch 24+0000 – 24+0080 is employed.</td></tr>
</table>

Skid resistance of surface layers in 2016
2016
14+0900 – 16+0820: PROY-1320 (01/08/2013) LB2. 16+0820 – 17+0990: PROY-1332 (01/06/2010) LB2. 17+0990 – 18+0700: PROY-1323 (01/06/2006) LB2. 18+0700 – 19+0110: PROJ-1324 (01/06/2010) LB2. 19+0110 – 19+0380: PROJ-1325 (01/06/2006) LB2. 19+0380 – 19+0850: PROJ-1326 (01/06/2010) LB2 19+0850 – 20+0420: PROJ-2011 (01/06/2006) LB2. 20+0420 – 23+0190: PROY-792 (01/08/2010) LB2. 23+0190 – 24+0010: PROY-1329 (01/07/2015) LB2.

Road Denomination	**BI-2731**	
Itinerary	UPV/EHU - Arkalanda	
Marker post (KP)	**From:** 14+0930	**To:** 18+0120
Transferred stretches		
-		

General comments:

In 2012, in the final part of the road, 18+0000 – 18+0120, the crossroad with BI-2704 was rebuilt (PROJ-305), but existing pavement (unknown) was used and a new surface layer was extended.

In 2014, a new outline was carried out, bypassing the campus of the University of the Basque Country, with a double carriageway (14+0960 – 15+0970) (PROJ-178).

The central part of the road, 16+0130 – 18+0050 is unknown, only the surface layer is registered.

Main actuations:

2012: PROJ-305 (28/09/2012)

18+0000 – 18+0120: BBTM 11A 3cm

2014: PROJ-178 (03/03/2014)

14+0960 – 15+0970: Double carriage, AC 16 surf S 6 cm + AC 22 bin S 6 cm + SC 28 cm + SEST 25 cm

15+0970 – 16+0130: AC 16 surf S 6 cm + AC 22 bin S 6 cm

IRI:

2000:

No pavement structure section is known.

2004:

No pavement structure section is known

2007:

No pavement structure section is known

2011:

No pavement structure section is known

2016:

Stretch 14+0960 – 15+0970, double carriageway, is introduced in the calculation.

Skid resistance. SFC:

2011:

No pavement structure section is known.

2016:

Stretch 14+0960 – 15+0970, double carriageway, is introduced in the calculation..

Skid resistance of surface layers in 2016

2016

16+0030 – 16+0130: PROJ-178 (03/03/2014) AC 16 surf S 6 cm.

16+0130 – 16+0680: PROJ-1331 (01/06/2006) LB2.

16+0680 – 18+0050: PROJ-1332 (01/06/2010) LB2.

18+0050 – 18+0120: PROJ-305 (28/09/2012) BBTM 11A 3 cm.

A.2.2. Conservation Area 2

<table>
<tr><td>Road Denomination</td><td colspan="2">N-240</td></tr>
<tr><td>Itinerary</td><td colspan="2">Tarragona – Bilbao (stretch Ubide – El Gallo)</td></tr>
<tr><td>Marker post (KP)</td><td>From: 23+0050</td><td>To: 55+0770</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">-</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">It is a national road, listed in the Plan Peña of 1940, from Tarragona to Bilbao and San Sebastián, with a lot of rehabilitation and maintenance works and high heavy traffic, because it is a not paying route for trucks. It crosses the Barazar mountain pass.
The complete pavement section is known from some stretches, where new outlines (usually bypasses of villages) were carried out
PROJ-330. 3rd lane in the mountain pass (01/06/1992) 28+0710 – 30+0520.
PROJ-36. Bypass of Zeanuri (19/07/2002) 35+0000 – 36+0810.
PROJ-35: Bypass of Igorre (01/11/2009). 43+0180 – 47+0640.
PROJ-1782: Renovation of the intersection of Apario (24/05/2002) 48+0000 – 48+0700.
PROJ-331: A new roundabout in Bedia (20/05/2014). 53+0000 – 53+0060.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">1992: PROJ-330 (01/06/1992). Only right lane according to marker post.
28+0710 – 30+0520: AC 22 surf S 6 cm + AC 22 bin G 6 cm + AC 32 base G 18 cm + MG (ZA) 20 cm
1997: PROJ-214 (01/11/1997)
54+0000 – 54+0260: AC 16 surf D 6 cm + AC 22 bin S 7 cm + AC 32 base G 12 cm + MG (ZA) 25 cm
2002: PROJ-1782 (24/05/2002)
48+0000 – 48+0700: BBTM 11A 3 cm + AC 22 bin S 5 cm + AC 22 base S 12 cm + GC 22 cm
2002: PROJ-211 (01/06/2002)
23+0140 – 30+0800: BBTM 11A 3 cm + AC 22 bin S 5 cm
2002: PROJ-36 (19/07/2002)
35+0000 – 36+0810: BBTM 11A 4 cm + AC 22 bin S 8 cm + GC 30 cm
2009: PROJ-35 (01/11/2009)
43+0180 – 47+04640: BBTM 11A 3 cm + AC 22 bin S 5 cm + AC 22 base G 12 cm + SC 25 cm
2014: PROJ-331 (20/05/2014)
53+0000 – 53+0060: BBTM 11A 3 cm + AC 22 bin S 6 cm + AC 22 base S 9 cm + AC 32 base S 11 cm + MG (ZA) 25 cm</td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">2000:</td></tr>
<tr><td colspan="3">28+0710 – 30+0520. Only right lane, 3rd lane. IRIm = 2,22. New outline from 1992.
54+0000 – 54+0260, very high values, > 6, due to the geometry of the roundabout and the presence of traffic calming measures.</td></tr>
<tr><td colspan="3">2002:</td></tr>
<tr><td colspan="3">28+0710 – 30+0520. Only right lane, 3rd lane. IRIm = 1,97 Rehabilitated in 2002.</td></tr>
<tr><td colspan="3">2004:</td></tr>
<tr><td colspan="3">35+0000 – 36+0810, Bypass of Zeanuri, IRIm = 2,00 New outline</td></tr>
<tr><td colspan="3">2007:</td></tr>
<tr><td colspan="3">28+0710 – 30+0520. Only right lane, 3rd lane. IRIm = 1,72. Local and global improvements are observed and, hence, discarded as spurious. Data from 2011 are even better.
35+0000 – 36+0810, Bypass of Zeanuri, IRIm = 2,42 New outline
48+0000 – 48+0700, Apario, IRIm = 2,27</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">35+0000 – 36+0810, Bypass of Zeanuri, IRIm = 2,32. Data are maintained as the variation can be possible.
43+0180 – 47+0640, Bypass of Igorre, IRIm = 1,47. New outline
48+0000 – 48+0700, Apario, IRI_m = 2,12. Better value, but maintained as it is similar to the previous data</td></tr>
</table>

2016
35+0000 – 36+0810, Bypass of Zeanuri, IRIm = 2,60 43+0180 – 47+0640, Bypass of Igorre, IRIm = 1,60 48+0000 – 48+0700, Apario, IRIm = 2,09 after maintenance. 53+0000 – 53+0060, IRI values very high due to the geometry of the roundabout.
Skid resistance. SFC:
2011:
35+0000 – 36+0810, Bypass of Zeanuri 43+0180 – 47+0640, Bypass of Igorre
2016:
35+0000 – 36+0810, Bypass of Zeanuri 43+0180 – 47+0640, Bypass of Igorre 48+0000 – 48+0700, Apario 53+0000 – 53+0060
Skid resistance of surface layers in 2016
2016
23+0140 – 30+0800: PROJ-211 (01/06/2002) BBTM 11A 3 cm 30+0800 – 34+0600: PROJ-858 (01/06/2012) LB 2 34+0600 – 34+1010: PROJ-330 (01/06/1992) AC 22 surf S 6 cm 36+0810 – 39+0250: PROJ-207 (01/06/2002) BBTM 11A 3 cm 39+0250 – 39+0490: PROJ-1996 (01/06/2015) BBTM 11A 3 cm 39+0490 – 43+0180: PROJ-207 (01/06/2002) BBTM 11A 3 cm 47+0640 – 47+1090: PROJ-857 (01/06/2002) LB 2 48+0700 – 49+0550: PROJ-857 (01/06/2002) LB 2 49+0550 – 50+0070: PROJ-205 (01/06/2002) BBTM 11A 3 cm 50+0070 – 50+0530: PROJ-1977 (01/06/2007) AC 16 surf S 5 cm 50+0530 – 50+0650: PROJ-205 (01/06/2002) BBTM 11A 3 cm 50+0650 – 50+0840: PROJ-1978 (01/06/2007) AC 16 surf 5 cm 50+0840 – 51+0490: PROJ-205 (01/06/2002) BBTM 11A 3 cm 51+0490 – 52+1000: PROJ-331 (20/05/2014) BBTM 11A 3 cm 53+0060 – 53+0960: PROJ-331 (20/05/2014) BBTM 11A 3 cm 54+0260 – 55+0180: PROJ-205 (01/06/2002) BBTM 11A 3 cm 55+0180 – 55+0510: PROJ-1976 (01/06/2009) AC 16 surf S 5 cm 55+0510 – 55+0780: PROJ-205 (01/06/2002) BBTM 11A 3 cm

Road Denomination	**N-636**	
Itinerary	Beasain – Durango (through Kanpazar)	
Marker post (KP)	**From:** 34+0980	**To:** 49+0830
Transferred stretches		
-		
General comments:		
It is national road, which connets Beasain (Gipuzkoa) with Durango through the mountain pass of Kanpazar (in the border of the two territories) In 1999 the bypass of Elorrio (PROJ-353; 39+0650 – 44+00990) was carried out. It has a stretch of double carriageway (41+0600 – 44+0000). Its management was given to a company after a tender. IRI and skid resistance data are available, but the rehabilitation and maintenance works carried out by the company are not registered. In 2016, another stretch of double carriageway was carried out, Gerediaga-Elorrio, linking the bypass of Elorrio with the AP-8 paying motorway and the management of the stretch 40+0520 – 49+0830. In the part managed by the Regional Government of Biscay, there is stretch that was carried out with the Bypass of Elorrio, but the complete pavement section is not registered. Moreover, the works on that stretch are not registered, as improvements are observed. Consequently, no stretch can be analyzed from this road.		
Main actuations:		
1999: PROJ-353 (16/04/1999) 39+0650 – 41+0590:BBTM 11A 4 cm + AC 16 bin S 4 cm + AC 32 base G 6 cm 41+0600 – 44+0000, ascending carriageway: BBTM 11A 4 cm + AC 16 bin S 4 cm + AC 32 base G 6 cm 44+0000 – 41+0600, descending carriageway, BBTM 11A 4 cm + AC 16 bin S 4 cm + AC 32 base G 6 cm 44+0010 – 44+0090:		
IRI:		
2000:		
There is no data in the stretches where the complete pavement structure is known		
2004:		
There is no data in the stretches where the complete pavement structure is known.		
2007:		
There is no data in the stretches where the complete pavement structure is known		
2011:		
There is no data in the stretches where the complete pavement structure is known		
2016:		
There is no data in the stretches where the complete pavement structure is known		
Skid resistance. SFC:		
2011:		
There is no data in the stretches where the complete pavement structure is known		
2016:		
There is no data in the stretches where the complete pavement structure is known		
Skid resistance of surface layers in 2016		
2016		
Recorded surfaces are not sure.		

Road Denomination	BI-623	
Itinerary	Durango – Vitoria/Gasteiz	
Marker post (KP)	**From:** 23+0810	**To:** 48+0840
Transferred stretches		
-		

General comments:

It is a road that connects Durango and Vitoria/Gasteiz through the mountain pass of Urkiola. It was the regional road CC-6211 of the Peña Plan of 1940.

There are some stretches where the complete pavement section is known: the bypass of Durango PROJ-351, 27+0810 – 30+0140, from 1985, which included a double carriageway with two lanes each; and the bypass of Otxandio, PROJ-339, 45+0700 – 48+0200, from 1998.

Moreover, some improvements in the road, like roundabouts, also are introduced as new outlines: PROJ-1773, 29+0000 – 29+0360; PROJ-1880, 30+0000 – 30+0140 in 2016; 30+0700 – 30+0890, PROJ-130 in 2009.

Main actuations:

1983: PROJ-371 (01/12/1983)

- 29+0900 – 33+0200: AC 16 surf S 5 cm + HMA 5 cm

1985: PROJ-351 (01/06/1985)

- Ascending carriage, 27+0810 – 29+0360: Ophite HMA 5 cm + HMA 7 cm + GC 18 cm + Unbound material (ZA) 20 cm
- Ascending carriage, 29+0360 – 30+0140: Ophite HMA 5 cm + HMA 7 cm + GC 18 cm + Unbound material (ZA) 20 cm
- Descending carriage, 29+0360 – 27+0810: Ophite HMA 5 cm + HMA 7 cm + GC 18 cm + Unbound material (ZA) 20 cm
- Descending carriage, 30+0140 – 29+0360: Ophite HMA 5 cm + HMA 7 cm + GC 18 cm + Unbound material (ZA) 20 cm

1988: PROJ-339 (30/08/1988)

- 45+0700 – 48+0200: AC 16 surf D 6 cm + AC 22 bin S 7 cm + GE 25 cm + Unbound material (ZA) 20 cm

1999: PROJ-128 (01/06/1999)

- 30+0150 – 33+0450: AC 22 surf S 6 cm + AC 22 base G 11 cm

2005: PROJ-338 (01/06/2005)

- 33+0970 – 36+0850: AC 16 surf S 5 cm + AC 22 bin S 8 cm
- 36+0850 – 46+0000: AC 16 surf S 5 cm + AC 22 bin S 7 cm

2005: PROJ-840 (01/06/2005)

- 45+0710 – 48+0820: LB2

2006: PROJ-1956 (01/06/2006)

- 33+0900 – 34+0090: BBTM 11A 3 cm

2009: PROJ-130 (01/06/2009)

- Ascending carriage, 27+0810 – 28+150: BBTM 11A 3 cm + AC 22 bin S 4 cm
- Descending carriage, 28+0150 – 27+0810:
- 30+0700 – 30+0890: BBTM 11A 3 cm + AC 22 bin S 6 cm + AC 22 base G 13 cm + Unbound material (ZA) 25 cm

2010: PROJ-1957 (01/06/2010)

- 32+0980 – 33+0230: BBTM 11A 3 cm

2012: PROJ-1773 (01/06/2012)

- Ascending carriage, 28+0920 – 29+0000: BBTM 11A 3 cm + AC 22 bin S 12 cm
- Ascending carriage, 29+0000 – 29+0360, lane 1: BBTM 11A 3 cm + AC 22 bin S 12 cm + AC 22 base G 14 cm
- Ascending carriage, 29+0000 – 29+0360, lane 2: BBTM 11A 3 cm + AC 22 bin S 12 cm + AC 22 base G 14 cm + Unbound material (ZA) 25 cm
- Descending carriage, 29+0360 – 28+0920: BBTM 11A 3 cm + AC 22 bin S 12 cm + AC 22 base G 14 cm

2012: PROJ-2001 (01/06/2012)

- Ascending carriage, 28+0670 – 28+0930: BBTM 11A 3 cm
- Descending carriage, 28+0920 – 28+0660: BBTM 11A 3 cm

2013: PROJ-1952 (08/05/2013)

30+0890 – 32+0980: AC 16 surf S 5 cm

33+0230 – 33+0900: AC 16 surf S 5 cm

2013: PROJ-834 (16/08/2013)

Ascending carriage, 28+0100 – 28+0670: LB2

2013: PROJ-835 (16/08/2013)

Descending carriage, 28+0660 – 28+0100: LB2

2013: PROJ-836 (16/08/2013)

Ascending carriage, 29+0500 – 30+0100: LB2

2013: PROJ-838 (16/09/2013)

34+0080 – 39+0870: LB2

2016: PROJ-1880 (01/04/2016)

Ascending carriage, 30+0000 – 30+0140: BBTM 11A 3 cm + AC 22 bin S 6 cm + AC 32 base S 9 cm + GC 20 cm + SC 20 cm

Ascending carriage, 29+0260 – 30+0000: BBTM 11A 3 cm

Descending carriage, 30+0140 – 29+0960: BBTM 11A 3 cm + AC 22 bin S 6 cm + AC 32 base S 9 cm + GC 20 cm + SC 20 cm

Descending carriage, 29+0960 – 29+0260: BBTM 11A 3 cm

30+0140 – 30+0220: BBTM 11A 3 cm

IRI:

2000:

27+0810 – 30+0140, only data from ascending carriage. IRIm = 2,62

45+0700 – 48+0200: IRIm = 2,35. Data are introduced

2004:

27+0810 – 30+0140, only data from ascending carriage, IRIm = 2,61. Some sections have some important improvements. Data with improvements over 0,40 m/km are discarded as spurious data. Corrected IRIm = 2,70.

45+0700 – 48+0200: IRIm =2,33. Some values are much better than in 2000. Data with improvements higher than 0,50 (after 4 years) are removed as spurious data. Final IRIm=2,46.

2007:

27+0810 – 30+0140, ascending carriage: IRIm = 3,27. Deterioration is observed in all sections from values of 2000. High values (IRI > 6,00 m/km) in some sections, which is reasonable for a 22 year old pavement.

27+0810 – 30+0140, descending carriage: IRIm = 3,98.

45+0700 – 48+0200: Stretch has been rehabilitated in the initial part (+ 12 cm HMA + Slurry) and maintained in the rest (slurry). IRIm,reh = 2,79; IRIm,main = 2,51.

2011:

27+0810 – 28+0150: Rehabilitated in 2009. Improvements are observed.

Ascending carriage: IRIm = 2,92 .Descending carriage: IRIm = 3,31.

28+0150 – 30+0140. General and local improvements are observed in both carriages. Rehabilitation works are performed from 2010 to 2012. Although works finished in 2012, some layers could be extended when data were collected. Hence, these data are not introduced.

45+0700 – 48+0200: IRIm,reh = 2,67; IRIm,main = 2,40. Improvements are observed, but only in some sections, not globally. Considering the data from 2016, data from 2011 represent and improvement between both. Data from 2016 seem reasonably and the continuation of data from 2007. Hence, data from 2011 are discarded as spurious but those from 2016 are introduced.

2016:

27+0810 – 28+0100: Rehabilitated in 2009. Ascending carriage IRIm = 3,22, descending carriage IRIm = 3,39.

28+0150 – 28+0670. Maintained in 2013 (slurry).

Ascending carriage, IRIm = 3,42, Descending carriage IRIm = 3,17.

28+0670 – 28+0920. Maintained in 2012 (+ BBTM 11A 3 cm).

Ascending carriage, IRIm = 2,11. Descending carriage, IRIm = 2,16

28+0920 – 29+0360. Rehabilitated and new pavement structure (in a new lane).

Ascending carriage, IRI values are high due to rough bands to speed down traffic and the presence of the roundabout (29+0200)

Descending carriage, there is stretch, after passing the roundabout, where there are not rough

	bands. 29+0130 – 28+020, IRIm = 1,27.
	29+0260 – 30+0000. Rehabilitation works are finished in 2016. IRI values are not worse than in 2011, but there are over 2 m/km (over the minimum initial IRI). That means that data were collected while works were carrying out. Consequently, data are discarded.
	45+0700 – 48+0200: IRIm,reh = 3,13; IRIm,mant = 2,70.
Skid resistance. SFC:	
2011:	
	Some stretches have the same pavement structure in 2011 and in 2016 and hence, variations from winter to summer can be observed: 27+0810 – 28+0100: Rehabilitated in 2009 30+0700 – 30+0890: New outline in 2009 45+0700 – 46+0000: Rehabilitated in 2005 46+0000 – 48+0200: Maintained in 2005
2016:	
	Stretches where all the pavement structure is known, and it has been in service (works that finished in 2016 are not introduced: 27+0810 – 28+0100, ascending, descending: Rehabilitated in 2009 28+0100 – 28+0670, ascending, descending: Maintained in 2013 28+0670 – 28+0920, ascending, descending: Maintained in 2012 30+0700 – 30+0890: New outline in 2009 45+0700 – 46+0000: Rehabilitated in 2005 46+0000 – 48+0200: Maintained in 2005
Skid resistance of surface layers in 2016	
2016	
	28+0920 – 29+0260, ascending, descending PROJ-1773 (01/06/2012) BBTM 11A 3cm 30+0150 – 30+0220: PROJ-1880 (01/04/2016) BBTM 11A 3 cm 30+0220 – 30+0700: PROJ-128 (01/06/1999) AC 22 surf S 6 cm 30+0890 – 32+0980: PROJ-1952 (08/05/2013) AC 16 surf S 5 cm 32+0980 – 33+0230: PROJ-1957 (01/06/2010) BBTM 11A 3 cm 33+0230 – 33+0900: PROJ-1952 (08/05/2013) AC 16 surf S 5 cm 33+0900 – 34+0080: PROJ-1956 (01/06/2006) BBTM 11A 3 cm 34+0080 – 39+0870: PROJ-838 (16/09/2013) LB2 39+0870 – 45+0710: PROJ-338 (01/06/2005) AC 16 surf S 5 cm 48+0200 – 48+0820: PROJ-840 (01/06/2005) LB2

Road Denomination	**BI-633**	
Itinerary	Durango - Ondarroa	
Marker post (KP)	**From:** 31+0450	**To:** 59+0560
Transferred stretches		
-		

General comments:

It is a road where new stretches have been constructed successively as alternative outline to existing road, which had a lot of curves. It is calledd the Corridor Durango – Ondarroa, linking these villages efficiently.

The complete pavement structure is known in these stretches and subsequently maintenance and rehabilitation works:

Stretch A1: 32+0040 – 33+0600 PROJ 362 (01/06/1991)
PROJ-29 (01/06/2012): 32+0130 – 32+0225.
PROJ-2063 (12/10/2016): 32+0200 – 33+0600
Stretch A2: 33+0600 – 35+0000 PROJ 369 (18/09/2000)
PROJ-2063 (12/10/2016): 34+0565 – 34+0850
PROJ-1967(31/01/2012) 34+0850 – 35+0000
Stretch A3: 35+0000 – 37+0500 PROJ-364 (01/06/1992)
PROJ-843 (16/07/2013): 35+0000 – 36+0770
PROJ-1829 (27/05/2016): 36+0750 – 36+0980
PROJ-380 (01/06/2010): 36+0980 – 37+0500
Stretch B: 37+0500 – 39+0300 PROJ—358 (27/04/1993)
PROJ-380 (01/06/2010): 37+0500 – 39+0300
Stretch C1: 39+0300 – 42+0200 PROJ-360 (01/07/1993)
PROJ-380 (01/06/2010): 39+0300 – 42+0200
Stretch C2.1: 42+0200 – 45+0400 PROJ-366 (01/11/1994)
PROJ-380 (01/06/2010): 42+0200 – 45+0400
PROJ-845 (16/08/2013): 44+0680 – 45+0400
Bypass of Markina: 47+0400 – 49+0200 PROJ-175 (01/12/1995)
PROJ-1966 (01/06/2007) 47+0400 – 47+0650
PROJ-1955 (16/08/2013) 47+0650 – 49+0200
Bypass of Ondarroa: 57+0000 – 59+0550 PROJ-355 (07/08/2002)
PROJ-848 (16/08/2013) 57+0000 – 59+0540
Intersection with N-634: 31+0460 – 32+0120 PROJ-29 (01/06/1992)

Main actuations:

New outline projects
1991: PROJ-362 (01/06/1991). A1
32+0040 – 33+0600: AC 16 surf S 6 cm + G20 11 + MG (ZA) 25 cm
2000: PROJ-369 (18/09/2000) A2
33+0600 – 35+0000: PA 11 4 cm + S12 4 cm + AC 22 base G 11 cm + MG (ZA) 25 cm
1992: PROJ-364 (01/06/1992) A3
35+0000 – 37+0500: AC 22 surf S 6 cm + G20 11 cm + MG (ZA) 25 cm
1993: PROJ-358 (27/04/1993) B
37+0500 – 39+0300: AC 16 surf S 6 cm + G20 11 cm + MG (ZA) 25 cm
1993: PROJ-360 (01/07/1993) C1
39+0300 – 42+0200: AC 16 surf S 6 cm + G20 11 cm + MG (ZA) 25 cm
1994: PROJ-366 (01/11/1994) C2.1
42+0200 – 45+0400: AC 16 surf S 6 cm + G20 11 cm + MG (ZA) 25 cm
1995: PROJ-174 (01/12/1995) Bypass of Markina
47+0400 – 49+0200: AC 22 surf S 6 cm + G25 7 cm + GE 20 cm + ZA 20 cm
2002: PROJ-355 (07/08/2002)
57+0000 – 59+0550: Ac 16 surf S 6 cm + G20 11 cm + ZA 25 cm + Soil CBR > 20
2012: PROJ-29 (01/06/2012) Intersection with N-634
31+0460 – 32+0120: S-12 4 cm + S-12 6 cm + GC 33 cm

Rehabilitation and maintenance projects
2012: PROJ-29 (01/06/2012)
    32+0130 – 32+0225: AC 16 surf S 4 cm
2016: PROJ-2063 (12/10/2016)
    32+0200 – 33+0600: Remove 3 cm + BBTM 11A 3 cm
2008: PROJ-842 (01/06/2008):
    32+0130 – 56+0830: Patching, crack repairing.
2016: PROJ-2063 (12/10/2016)
    34+0565 – 34+0850: Remove 7 cm + BBTM 11A 3 cm + S12 4 cm
2012: PROJ-1967 (31/01/2012):
    34+0850 – 35+0000: Remove 4 cm + BBTM 11A 4 cm
2013: PROJ-843 (16/07/2013)
    35+0000 – 36+0770: LB2
2016: -PROJ-1829 (27/05/2016)
    36+0750 – 36+0980: BBTMA 11A 3 cm
2010: PROJ-380 (01/06/2010)
    36+0980 – 45+0650: BBTMA 11A 3 cm + S12 5 cm
2013: PROJ-845 (16/08/2013)
    44+0680 – 47+0340: LB2
2007: PROJ-1966 (01/06/2007)
    47+0340 – 47+0650: AC 16 surf S-12 5 cm
2013: PROJ-1955 (16/08/2013)
    47+0650 – 49+0200: LB2
2013: PROJ-848 (16/08/2013)
    57+0000 – 59+0540: LB2

IRI:

Introduced IRI data are commented by individual stretches, indicating years that are used.
Stretch A1, 32+0100 – 33+0600, 01/06/1991.
    2000: IRIm = 2,26. 2004: IRIm = 1,83 (generalized improvements are observed).
    2007: IRIm = 2,18. 2011: IRIm = 2,18. And no works are recorded until 2012.
    No data are used from this stretch. Not registered activities must be carried out.
Stretch A2: 33+0600 – 34+1130 (18/09/2000)
    2004: IRIm = 1,79. 2007: IRIm = 2,13. 2011: 2,07 (maintenance, patching in 2008)
    2016: MANT-2008, IRIm = 2,10; MANT-2012, IRIm = 2,02.
    All years' data are included.
Stretch A3: 35+000 – 37+0500 (01/06/1992)
    2000: IRIm = 2,03. 2004: IRIm = 2,13 (eliminating data with improvements > 0,30).
    2007: IRIm = 2,41 (eliminating data with improvements > 0,30).
    2011: New Outline IRIm = 2,55; REHAB IRIm = 1,01.
    2016: Mant-1(2013); IRIm = 2,59; NewOutline 1992: IRIm = 2,58 (vaules are higher than in 2011, so data were collected before PROJ-1829); REHAB 2010, IRI m = 1,03.
    All years' data are included.
Stretch B: 37+0500 – 39+0300 (27/04/1993)
    2000: IRIm = 2,36. 2004: IRIm = 2,41 (eliminating data with improvements > 0,30).
    2007: IRIm = 2,66 (eliminating 3 data with improvements > 0,30).
    2011: REHAB IRIm = 0,97. 2016: REHAB IRIm = 1,06. All years' data are included.
Stretch C1: 39+0300 – 42+0200 (01/07/1993)
    2000: IRIm = 2,09. 2004: IRIm = 2,03 (eliminating data with improvements > 0,30). 2007: IRIm = 2,63. 2011: REHAB, IRIm = 1,20. 2016: REHAB, IRIm = 1,24. All years' data are included.
Stretch C2.1: 42+0200 – 45+0400 (01/11/1994)
    2000: IRIm = 1,76. 2004: Generalized improvements are observed. These data are discarded. 2007: 1,77 (eliminating data with improvements > 0,30). 2011: REHAB, IRIm = 1,52. 2016: REHAB, IRIm = 1,61. Data from 2004 were eliminated, as it seems that in 2004 spurious data were collected if no maintenance activity was performed between 2000 and 2004, as the PMS states.
Bypass of Markina: 47+0400 – 49+0200 (01/12/1995)
    2000: IRIm = 1,80. 2004: No data are recorded. 2007: 2,13. 2011: In the maintained area IRI values get worse, and in non-maintained stretch IRI values improved. 2016: No data are recorded. Values from 2000

and 2007 are only employed. Other data are discarded as spurious.
Bypass of Ondarroa: 57+0000 – 59+0550 (07/08/2002)
2004: IRIm = 2,32 (high in comparison with other stretch older than this). 2007: IRIm = 1,85. 2011: IRIm = 1,73. Values get better and better. Data were revised. Until 58+0550, values seem reasonable: IRI-2004: 1,57, IRI-2007: 1,89, IRI-2011: 1,78. From 58+0550, values in 2004 are very high (some over 6 m/km), and later, they become lower and lower, indicating that works have been done. Consequently, data from 57+0000 – 58+0550 from 2004 and 2007 are only introduced in the calculation.
Intersection with N-634: 31+0460 – 32+0120 (01/06/2012)
2016: IRIm = 2,60, values over 5,00 for a stretch 4 years old. Discarded as spurious data. There are sonorous barriers before roundabouts and they disturb collected data.
Skid resistance. SFC:
2011:
Stretches with complete pavement known and which were not rehabilitated or maintained during 2011-2016 are included to compare data with 2016: 33+0600 – 34+0850 36+0750 – 36+0980 36+0980 – 37+0500 37+0500 – 39+0300 39+0300 – 42+0200 42+0200 – 44+0690.
2016:
Analyzed stretches, with known complete pavement structure: 33+0600 – 34+0850: PA 11 (initial) 34+0850 – 35+0000: BBTM 11 A, some stretches are not introduced as experimental micro-removing is applied in April 2016 35+0000 – 36+0750: LB2 36+0750 – 36+0980: AC 22 surf S (initial) 36+0980 – 37+0500: BBTM 11A 37+0500 – 39+0300 BBTM 11A, some stretches are not introduced as experimental micro-removing is applied in April 2016. 39+0300 – 42+0200 BBTM 11A, some stretches are not introduced as experimental micro-removing is applied in April 2016. 42+0200 – 44+0680: BBTM 11A 44+0680 – 45+0400: LB2, some stretches are not introduced as experimental micro-removing is applied in April 2016.
Skid resistance of surface layers in 2016
2016
32+0130 – 32+0200: PROJ-29 (01/06/2012) AC 16 surf S 4 cm 34+0940 – 35+0100: PROJ-1855 (25/04/2016) Micro-milling 38+0140 – 38+0740: PROJ-1856 (25/04/2016) Micro-milling 38+0870 – 39+0130: PROJ-1857 (25/04/2016) Micro-milling 42+0130 – 42+0620: PROJ-1858 (25/04/2016) Micro-milling 43+0060 – 43+0210: PROJ-1859 (25/04/2016) Micro-milling 45+0400 – 47+0340: PROJ-845 (16/08/2013) LB2 47+0340 – 47+0650: PROJ-842 (01/06/2007) AC 16 surf S 5 cm 47+0650 – 50+0500: PROJ-1955 (16/08/2013) LB2 52+0400 – 52+0870: PROJ-847 (21/08/2014) LB2 56+0830 – 59+0540: PROJ-848 (16/08/2013) LB2

Road Denomination	**BI-638**	
Itinerary	Deba - Ondarroa	
Marker post (KP)	**From:** 8+0000	**To:** 8+0900
Transferred stretches		
-		
General comments:		
The first documented project, from 1982, included a regularizing layer over existing pavement (5 cm) and slurry, in all the stretch (PROJ-359). In 1993, a layer of AC 16 surf S was extended to regularize (4 cm) and over it, 5 cm of AC 16 surf S (8+0000 – 8+0500) (PROJ-337). In 2010, a roundabout was built in the final part of the road (PROJ-336). Existing pavement was used and an AC 16 surf S layer (5 cm) was extended over it. No complete pavement section is known.		
Main actuations:		
1982: PROJ-359 (01/06/1982) 8+0000 – 8+0900: Bituminous layer to regularize (5 cm) and slurry 1993: PROJ-337 (01/06/1993) 8+0000 – 8+05000: AC 16 surf S 5 cm + Regularization layer 4 cm 2010: PROJ-336 (01/06/2010) 8+0800 – 8+0900: AC 16 surf S 5 cm		
IRI:		
2000:		
No pavement structure section is known.		
2004:		
No pavement structure section is known		
2007:		
No pavement structure section is known		
2011:		
No pavement structure section is known		
2016:		
No pavement structure section is known.		
Skid resistance. SFC:		
2011:		
No pavement structure section is known.		
2016:		
No pavement structure section is known.		
Skid resistance of surface layers in 2016		
2016		
8+0000 – 8+0870: PROJ-850 (16/08/2013) LB2. 8+0870 – 8+0900: PROJ-336 (01/06/2013) AC 16 surf S 5 cm.		

Road Denomination	**BI-732**	
Itinerary	Montefuerte – San Fausto	
Marker post (KP)	**From:** 49+0130	**To:** 51+0390
Transferred stretches		
-		

General comments:
This short road can be divided in 3 parts according to its construction dates. The first part, 49+0130 – 50+0210, was constructed between 1991 and 1995, but the project is not available. Consequently, PROJ-1985 (49+0600 – 49+0900) were introduced in the PMS, to register the initial projects of the road, the construction data are supposed to be in 1994 and the surface layers, the only layer identified, is recorded in the project data. The pavement section is unknown. The central part was built between 1991 and 2001. The construction project is not available and hence, PROJ-1984, is introduced in the PMS, to record the surface layer, the only part that has been identified. It is supposed that it was concluded in 2000. Later, it was used as lateral road of the Bypass road that was performed in 2005. This new stretch, 50+0390 – 51+0100, consists of two parts: a two-lane road, 50+0390 – 50+0940 and a double carriageway part, 50+0940 – 51+0100 (PROJ-343) and the complete pavement section is known. The final part, 51+0100 – 51+0390 is also a double carriageway road and was made between 1991 and 2001 but the project is not conserved. Once again, PROJ-1983 is introduced in the PMS, indicating that it was finished in 2000 and recording only the surface layer. It was the initial pavement, but the complete pavement structure is unknown. Apart from these projects, maintenance works have been performed, which are registered. In 51+0010, there are not working railroads, which modify IRI values
Main actuations:
1994: PROJ-1985 (01/01/1994) 49+0600 – 49+0900: AC 16 surf S 2000: PROJ-1984 (01/01/2000) 50+0200 – 50+0420: AC 16 surf S 5 cm 2000: PROJ-1983 (01/01/2000) 51+0100 – 51+0390 Double carriage: BBTM 11 A 3 cm 2005: PROJ-343 (01/11/2005) 50+0390 – 50+0940: BBTM 11A 3 cm + AC 22 bin S 12 cm + GE 30 cm + MG (ZA) 20 cm 50+0940 – 51+0100: Double carriage, BBTM 11A 3 cm + AC 22 bin S 12 cm + GE 30 cm + MG (ZA) 20 cm 51+0100 – 51+0200: Double carriage, BBTM 11 A 3 cm. 2009: PROJ-1982 (01/06/2009) 49+0900 – 50+0210 : AC 16 surf S 5 cm. 2013: PROJ-851 (18/07/2013) 49+0130 – 50+0940: Crack repairing, patching. 2013: PROJ-852 (08/05/2013) 50+0940 – 51+0380 ascending: Crack repairing, patching. 2013: PROJ-853 (08/05/2013) 50+0940 – 51+0380 descending: Crack repairing, patching.
IRI:
2000:
No pavement structure section is known.
2004:
No pavement structure section is known
2007:
50+0390 – 50+0940. IRIm = 1,52
2011:
50+0390 – 50+0940. IRIm = 1,52
2016:
50+0390 – 50+0940, with crack repairing. IRIm =1,60

Skid resistance. SFC:
2011:
50+0390 – 50+0940, in winter
2016:
50+0390 – 50+0940, in summer
Skid resistance of surface layers in 2016
2016
49+0600 – 49+0900: PROJ-1985 (01/01/1994) AC 16 surf S 5 cm 49+0900 – 50+0210: PROJ-1982 (01/06/2009) AC 16 surf S 5 cm 50+0210 – 50+0390: PROJ-1984 (01/01/2000) AC 16 surf S 5 cm 51+0200 – 51+0340: PROJ-1983 (01/01/2000) BBTM 11 A 3 cm

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-2224</td></tr>
<tr><td>Itinerary</td><td colspan="2">Gernika - Markina</td></tr>
<tr><td rowspan="4">Marker post (KP)</td><td>From: 33+0830</td><td>To: 37+0200</td></tr>
<tr><td>From: 37+0810</td><td>To: 45+0090</td></tr>
<tr><td>From: 47+0220</td><td>To: 52+0520</td></tr>
<tr><td>From: 53+0030</td><td>To: 54+0640</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">37+0200 – 37+0810: Loyola
45+0090 – 47+0220: Munitibar
52+0520 – 53+0030: Ziortza-Bolivar</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">In the historic archive it is registered that this road was constructed in 1883. In 1984, it is documented that 8000 t of aggregates were employed.
There were some plan projects, PROJ-76 (01/07/1993) and PROJ-77 (01/06/1993). They were plan project, not construction projects.
There are some stretches were complete pavement structure is known due to a complete renovation of the pavement of new outline:
33+0830 – 34+0100, related to the Bypass of Gernika (PROJ-30), in 2003.
47+0230 – 47+0300, a new outline for a curve was performed in 2007 under PROJ-80.
47+0300 – 52+0510, a complete new pavement was carried out. Existing pavement was used as subbase layer in 1999 (PROJ-79)
53+0040 – 54+0620, a complete new pavement structure was performed in 2011. Some parts of existing pavement, but all the stretch is computed as new pavement structure (PROJ-341) in the PMS.
PROJ-78 indicates that some repairs were made in some localized points in 37+0000 – 45+0150 in 2006, but it is not included as new pavement in any place.
PROJ-378 states that some rehabilitation works were carried out in 33+0920 – 34+0070 and 35+0090 – 35+0370 in 2013.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">1999: PROY-79 (01/06/1999) (Data is not real, it has estimated 2 years after project)
47-0300 – 52+0510: AC 16 surf S 5 cm + AC 22 bin S 6 cm + MG (ZA) 20 cm
2003: PROJ-30 (01/11/2003)
33+0830 – 34+0100: AC 16 surf S 4 cm + S12 4 cm + S12 4 cm+ GE 24 cm
2007: PROY-80 (01/06/2007)
47+0230 – 47+0300: AC 16 surf S 5 cm + S12 5 cm + MG (ZA) 30 cm
2008: PROY-78 (01/06/2008)
37+0000 – 45+1500: Localized repairs, patching
2009: PROJ-1552 (01/06/2009)
34+0230 – 35+0100: LB2 + S12 5 cm
2009: PROJ-1988 (01/08/09)
35+0180 – 38+0550: LB2 + S12 5 cm
2011: PROY-341 (01/06/2011)
53+0040 – 54+0620: AC 16 surf S 5 cm + AC 22 bin S 9 cm + MG (ZA) 25 cm
2013: PROY-378 (03/04/2013)
33+0920 – 34+0070: AC 16 surf S 5 cm + AC 32 bin S 9 cm
35+0090 – 35+0370: AC 16 surf S 5 cm + AC 32 bin S 9 cm
2013: PROJ-794 (30/09/13)
35+0040 – 35+0170: LB2
2013: PROJ-800 (30/09/13)
34+0070 – 35+0040: LB2
2016: PROY-1828 (03/06/2016)
49+0000 – 52+0520: Remove 5 cm, AC 16 surf S 5 cm + AC 22 base G 15 cm
2016: PROJ-1876 (12/09/2016)
47+0220 – 49+0000: Remove 5 cm, AC 16 surf S 5 cm</td></tr>
</table>

IRI:	
2000:	
	47+0300 – 52+0700: IRIm = 2,77 (47+0300 – 50+0950). It seems high for a one-year old road. Perhaps project finished after data collection of 2000. Data are not introduced in the calculation.
2004:	
	No data from 2004
2007:	
	33+0830 – 34+0100: IRIm = 2,89 New outline 47+0230 – 47+0300: IRIm = 1,89 New outline. 47+0300 – 52+0520: IRIm = 3,52 New outline.
2012:	
	33+0830 – 34+0100: IRIm = 3,17. New outline. 47+0230 – 47+0300: IRIm = 2,56. New outline. 47+0300 – 52+0520: IRIm = 2,59 (all stretch). General and local improvements are observed. All data are rejected as spurious due to a possible non recorded maintenance work. 53+0040 – 54+0020: IRIm = 1,75 New outline.
2016:	
	33+0920 – 34+0070: IRIm = 4,17. After rehabilitation (+ 14 cm HMA). It seems high but it is introduced in the calculation. 47+0230 – 47+0300: IRIm = 3,44. It seems that the maintenance work in 2016 was performed after data collection. Hence, it remains as new outline for the calculation. 53+0040 – 54+0020: IRIm = 1,77. New outline
Skid resistance. SFC:	
2011:	
	33+0830 – 33+0920 New outline from 2003. 47+0230 – 47+0300. Data in 2016 was collected before finishing rehabilitation. It is possible to compare. 53+0040 – 54+0020: Data were collected before the construction of the new outline. Data cannot be compared with .data from 2016
2016:	
	33+0830 – 33+0920 New outline from 2003. 33+0920 – 34+0070, rehabilitation in 2013. 34+0070 – 34+0100: Maintenance in 2013 47+0230 – 47+0300: Data is collected 1 month before the rehabilitation. 53+0040 – 54+0020:
Skid resistance of surface layers in 2016	
2016	
	34+0100 – 35+0040: PROY-800 (30/09/2013) LB2. 35+0040 – 35+0170: PROY-794 (30/09/2013) LB2. 35+0170 – 35+0370: PROY-378 (03/04/2013) AC 16 surf S 5 cm. 35+0370 – 38+0550: PROY-1988 (01/08/2009) LB2

Road Denomination	**BI-2301**	
Itinerary	Ermua – Arrangiznaga	
Marker post (KP)	**From:** 41+0650	**To:** 48+0050

Transferred stretches

41+0370 – 41+0640: Ermua.

General comments:

There are two projects, which detailed complete pavement structure, PROJ-340, 42+0200 – 42+0450 (01/06/1995) and PROJ-360, 47+0300 – 48+0050 related to BI-633 Stretch C1 (01/07/1993).

Moreover, there is project for rehabilitation of almost all the road, PROJ-99 (01/11/2006) and for maintenance, PROJ-822.

Main actuations:

1993: PROJ-360 (01/07/1993)

47+0300 – 48+0050: AC 16 surf S 5 cm + S12 7 cm + MG (ZA) 20 cm + E2 60 cm

1995: PROJ-340 (01/06/1995)

42+0200 – 42+0450: AC 16 surf S 5 cm + G20 7 cm + MG (ZA) 40 cm (in 2 layers)

2006: PROJ-99 (01/06/2006)

42+0000 – 43+0375: AC 16 surf S 5 cm + S12 4 cm

43+0375 – 43+0825: AC 16 surf S 5 cm + S12 5 cm

43+0825 – 44+0512: AC 16 surf S 6 cm

44+0512 + 44+0938: AC 16 surf S 5 cm + S12 5 cm

44+0938 – 46+0077: AC 16 surf S 6 cm

46+0077 – 48+0000: AC 16 surf S 5 cm + S12 4 cm

2013: PROJ-822 (16/08/2013)

42+0010 – 48+0000: LB2

IRI:

2000:

42+0200 – 42+0450: IRIm = 4,24

47+0300 – 48+0050: IRIm = 3,07

2004:

No data from 2004

2007:

42+0200 – 42+0450: IRIm = 1,88, after rehabilitation.

47+0300 – 48+0050: IRIm =2,01

2011:

42+0200 – 42+0450: IRIm = 1,37. Improvements are observed. Data are discarded as spurious

47+0300 – 48+0050: IRIm = 2,09. Some local improvements (3 sections) are eliminated. Rest of data introduced

2016:

42+0200 – 42+0450: IRIm = 1,39, after maintenance in 2013.

47+0300 – 48+0050: IRIm = 2,00, after maintenance in 2013.

Skid resistance. SFC:

2011:

Pavement structure in both stretches with known section is modified in 2013

2016:

Analyzed stretches: 42+0200 – 42+0450 and 47+0300 – 48+0050.

Skid resistance of surface layers in 2016

2016

42+0010 – 42+0200: PROJ-822 (16/08/2013) LB2.

42+0450 – 47+0300: PROJ-822 (16/08/2013) LB2.

48+0000 – 48+0030: PROJ-822 (16/08/2013) LB2.

Road Denomination	**BI-2405**	
Itinerary	Plazakola - Lekeitio	
Marker post (KP)	**From:** 55+0120	**To:** 63+0350
	From: 64+0000	**To:** 67+0090

Transferred stretches

63+0350 – 64+0000: Oleta

General comments:

There are 2 project which establish complete pavement structure, as initial pavement, PROJ-123 (20/10/2003), in stretch 59+0520 – 63+0350, and the Bypass of Lekeitio, PROJ-7, in stretch 65+0500 – 67+0090 (23/10/2009).

In 2008, PROJ-5 is carried out, where new bituminous layers were extended (64+0000 – 65+0500), with varying thickness, from 13 to 18 cm of HMA.

Main actuations:

2003: PROJ-123 (20/10/2003)

59+0520 – 63+0350: AC 16 surf S 5 cm + G20 10 cm + AC 22 base G 5 cm + MG (ZA) 25 cm

2008: PROJ-5 (01/05/2008)

64+0000 – 65+0500: Ac 16 surf S 5 cm + 8 or 13 HMA

2009: PROJ-3 (23/10/2009)

65+0500 – 67+0090: AC 16 surf 5 cm + AC 22 bin S 8 cm + GC 30 cm

2012: PROJ-824 (01/06/2012)

58+0840 – 63+0350: LB2

IRI:

2000:

No pavement structure section is known.

2004:

Stretch 59+0520 – 63+0350 is employed as new outline. IRI_m = 1,89.

2007:

Stretch 59+0520 – 63+0350 is employed as new outline. IRI_m = 2,14.

2011:

Stretch 59+0520 – 63+0350 is employed as new outline. IRI_m = 2,31. Improvements are observed, improvements over 0,3 are discarded as spurious. IRIm=2,51.

Stretch 65+0500 – 67+0090 is employed as new outline. IRI_m = 1,46.

2016:

Stretch 59+0520 – 63+0350 is employed as maintenance. IRI_m = 2,10.

Stretch 65+0500 – 67+0090 is employed as new outline. IRI_m = 1,55.

Skid resistance. SFC:

2011:

Stretch 65+0500 – 67+0090 is employed as new outline.

2016:

Stretch 59+0520 – 63+0350 is employed as maintenance.

Stretch 65+0500 – 67+0090 is employed as new outline

Skid resistance of surface layers in 2016

2016

55+0120 – 56+0390: PROJ-823 (01/06/2011) LB2.

56+0520 – 58+0840: PROJ-823 (01/06/2011) LB2.

58+0840 – 59+0520: PROJ-825 (01/06/2012) LB2.

64+0000 – 65+0500: PROJ-5 (01/05/2008) AC 16 surf S 5 cm.

Road Denomination	**BI-2543**	
Itinerary	Igorre –- Otxandio (through Dima)	
Marker post (KP)	**From:** 22+0600	**To:** 25+0000
	From: 26+0000	**To:** 41+0150
Transferred stretches		
25+0000 – 26+0000: Dima		

General comments:

The complete pavement structure of the vast majority of the road outline is known.

In 1989 it is finished the reconstruction of the pavement of the stretch 26+000 – 36+0700 (PROJ-334). In 1991 the reconstruction of the entire pavement of the stretch 36+0700 – 41+0150 (PROJ-335) is concluded. These 2 stretches are rehabilitated in 2008 with bituminous layers of varying thickness (PROJ-112).

In 2009 a new outline is carried out in the initial part as a consequence of the Bypass of Igorre (PROJ-35).

There are two maintenance projects, PROJ-827 and PROJ-828, where slurries are applied.

Main actuations:

1989: PROJ-334 (10/10/1989)

25+0600 – 36+0700: AC 16 surf S 4 cm + Gravaemulsion 8 cm + MG (ZA) 15 cm

1991: PROJ-335 (01/06/1991)

36+0700 – 41+0160: AC 16 surf S 5 cm + AC 22 bin S 7 cm + MG (ZA) 12 cm

2008: PROJ-112 (01/06/2008)

26+0000 – 27+0018: AC 16 surf S 5 cm + AC 22 bin S 4 cm
27+0018 – 27+0509: AC 16 surf S 6 cm
27+0509 – 27+0809: AC 16 surf S 5 cm + AC 22 bin S 7 cm
27+0809 – 28+0858: AC 16 surf S 5 cm + AC 22 bin S 4 cm
28+0858 – 29+0600: AC 16 surf S 5 cm + AC 22 bin S 5 cm
29+0600 – 31+0050: AC 16 surf S 5 cm + AC 22 bin S 4 cm
31+0050 – 31+0887: AC 16 surf S 5 cm + AC 22 bin S 5 cm
31+0887 – 32+0192: AC 16 surf S 6 cm
32+0192 – 32+0342: AC 16 surf S 5 cm + AC 22 bin S 7 cm
32+0342 – 33+0200: AC 16 surf S 5 cm + S12 4 cm
33+0200 – 36+0400: AC 16 surf S 5 cm + AC 22 bin S 5 cm
36+0400 – 38+0010: AC 16 surf S 5 cm + S12 4 cm
38+0010 – 39+0925: AC 16 surf S 5 cm + AC 22 bin S 7 cm
39+0925 – 41+0107: AC 16 surf S 5 cm + AC 22 bin S 5 cm
41+0107 – 41+0160: AC 16 surf S 5 cm + AC 22 bin S 7 cm

2009: PROJ-35 (01/11/2009)

22+0660 – 22+0750: AC 16 surf S 5 cm + AC 22 bin S 7 cm + SC 33 cm

2013: PROJ-827 (16/09/2013)

22+0760 – 23+0170: LB2

2013: PROJ-828 (16/09/2013)

23+0390 – 23+0480: LB2

IRI:

2000:

26+0000 – 36+0700: IRIm = 2,88 (11 years) New outline
36+0700 – 41+0150: IRIm = 2,28 (9 years) New outline

2004:

No data from 2004

2007:

26+0000 – 36+0700: IRIm = 4,08 (18 years) New outline
36+0700 – 41+0150: IRIm = 3,07 (16 years) New outline

2011:
22+0660 – 22+0750: IRIm = 4,81 (Values are conditioned by bridge joints) No introduced. 26+0000 – 36+0700: IRIm = 1,43 Rehabilitation 36+0700 – 41+0150: IRIm = 0,9 Rehabilitation
2016:
26+0000 – 36+0700: IRIm = 1,52 Rehabilitation 36+0700 – 41+0150: IRIm = 1,10 Rehabilitation
Skid resistance. SFC:
2011:
Data collected in winter. 26+0000 – 36+0700 Rehabilitated 36+0700 – 41+0150 Rehabilitated
2016:
26+0000 – 36+0700 Rehabilitated 36+0700 – 41+0150 Rehabilitated
Skid resistance of surface layers in 2016
2016
22+0660 – 22+0750: PROJ-35 (01/11/2009) AC 16 surf S 5 cm. 22+0760 – 23+0170: PROJ-827 (16/09/2013) LB2 23+0170 – 23+0390: PROJ-1987 (01/06/2007) AC 16 surf S 5 cm 23+0390 – 23+0480: PROJ-828 (16/09/2013) LB2 23+0480 – 24+0870: PROJ-1995 (01/01/2000) AC 16 surf S 5 cm

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-2632</td></tr>
<tr><td>Itinerary</td><td colspan="2">Elorrio - Bergara</td></tr>
<tr><td rowspan="2">Marker post (KP)</td><td>From: 39++0450</td><td>To: 43+0220</td></tr>
<tr><td></td><td></td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">-</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">It is a short road, less than 4 km, which links Elorrio and the border of the province with Gipuzkoa, through a mountain pass. It has low traffic.
In 1987, the entire road was rehabilitated by extending two bituminous layers, 5 cm of AC 16 surf S after 5 cm of G20 (PROJ-342).
Next activity was a maintenance work, the extension of slurry (LB2) all road long in 2013 (PROJ-829).
The complete pavement section is unknown.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">1987: PROJ-342 (01/06/1987)
39+0450 – 43+0220: AC 16 surf S 5 cm + G20 5 cm.
2013: PROJ-829 (16/09/2013)
39+0450 – 43+0220: LB2</td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">2000:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known. IRI_m = 3,87.</td></tr>
<tr><td colspan="3">2004:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known. IRI_m = 3,25.</td></tr>
<tr><td colspan="3">2007:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known. IRI_m = 3,46.</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known. IRI_m = 3,31.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known. IRI_m = 3,33.
IRI improvements are observed (global) and the only maintenance work registered was conducted in 2013.</td></tr>
<tr><td colspan="3">Skid resistance. SFC:</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">No complete pavement structure section is known.</td></tr>
<tr><td colspan="3">Skid resistance of surface layers in 2016</td></tr>
<tr><td colspan="3">2016</td></tr>
<tr><td colspan="3">39+0450 – 43+0220: PROJ-829 (16/09/2013) LB2.
.</td></tr>
</table>

Road Denomination	**BI-2636**	
Itinerary	Markina – Elgoibar	
Marker post (KP)	**From:** 50+0860	**To:** 52+0300
	From: 53+0000	**To:** 58+0070
Transferred stretches		
52+0300 – 53+0000: Etxebarria		
General comments:		
There are not documented projects about pavement structure from this road. The project PROJ-113, from 1999, which included a new pavement structure, was not carried out. The only projects related to this road are 2 maintenance activities from 2009, where slurry was extended in both stretches of the road.		
Main actuations:		
2013: PROJ-831 (01/06/2009) 50+0860 – 52+0300: LB2. 2013: PROJ-833 (01/06/2009) 53+0000 – 58+0070: LB2		
IRI:		
2000:		
No complete pavement structure section is known.		
2004:		
No complete pavement structure section is known.		
2007:		
No complete pavement structure section is known.		
2011:		
No complete pavement structure section is known.		
2016:		
No complete pavement structure section is known.		
Skid resistance. SFC:		
2011:		
No complete pavement structure section is known.		
2016:		
No complete pavement structure section is known.		
Skid resistance of surface layers in 2016		
2016		
50+0860 – 52+0300: PROJ-831 (01/06/2009) LB2. 53+0000 – 58+0070: PROJ-833 (01/06/2009) LB2.		

A.2.3. Conservation Area 3

Road Denomination	**N-639**	
Itinerary	Access to the Port of Bilbao through Zierbena	
Marker post (KP)	**From:** 15+0880	**To:** 24+0170
Transferred stretches		
-		

General comments:

It is a road that gives an alternative access to the Port of Bilbao through Zierbena.

There are some stretches were the complete pavement section is known: PROJ-224 (21+0300 – 22+0800) from 1995, PROJ-399 (22+0800 – 23+0750) from 1992; PROJ-447 (23+0750 – 24+0170) from 1987.

In 2008, 2012 and 2016, there are some reconstruction and rehabilitation works in these stretches.

Main actuations:

1987: PROJ-447 (20/11/1987)

23+0750 – 24+0170: HMA Ophite 5 cm + HMA 10 cm + Gravel cement 20 cm + MG (ZA) 20 cm

1992: PROJ-399 (28/12/1992)

22+0800 – 23+0750: PA 11 4 cm + AC 22 bin D 5 cm + AC 22 bin S/G 5+7 cm + MG (ZA) 20 cm

1995: PROJ-224 (01/06/1995)

21+0300 – 22+0800: AC 16 surf S 6 cm + AC 22 base G 11 cm + MG (ZA) 25 cm

2008; PROJ-12 (18/06/2008)

23+0870 – 24+0170: AC 16 surf S 6 cm

2012: PORJ-1599 (01/06/2012)

21+0200 – 21+0360: A 16 surf S 5 cm

2016: PROJ-1885 (17/11/2016)

21+0370 – 22+0750: AC 16 surf S 5 cm

IRI:

2000:

21+0300 – 22+0800: IRIm = 1,57

22+0800 – 23+0750: IRIm = 1,85

23+0750 – 24+0170: IRIm = 2,10

2004:

There is no data in the stretches where the complete pavement structure is known.

2007:

21+0300 – 22+0800: IRIm = 1,62

22+0800 – 23+0750: IRIm = 1,89 (some improvements, over 0,20, are discarded

23+0750 – 24+0170: IRIm = 2,45

2011:

21+0300 – 22+0800: IRIm = 1,58. Some improvements are observed. Sections with an improvement > 0,30 after 4 year is discarded as spurious. New IRIm = 1,63.

22+0800 – 23+0750: IRIm = 1,76. Some improvements are observed. Sections with an improvement > 0,30 after 4 year is discarded as spurious. New IRIm = 1,84.

23+0750 – 24+0170: Not maintained, IRIm = 2,19

23+0870 – 24+0130, Rehabilitated, IRIm = 2,21

2016:

21+0300 – 22+0800: IRIm = 1,97.

22+0800 – 23+0750: IRIm = 2,04

23+0750 – 24+0170: Not maintained, IRIm = 2,30

23+0870 – 24+0130, Rehabilitated, IRIm = 2,23

Skid resistance. SFC:
2011:
Stretches that maintain the same pavement structure are introduced: 21+0360 – 22+0800 22+0800 – 23+0750 23+0750 – 23+0870 23+0870 – 24+0170
2016:
Stretches in which the entire section is known are introduced: 21+0300 – 21+0360 21+0360 – 22+0800 22+0800 – 23+0750 23+0750 – 23+0870 23+0870 – 24+0170
Skid resistance of surface layers in 2016
2016
15+0880 – 18+0870: PROJ-1018 (01/06/2005) LB2 18+0870 – 19+0390: PROJ-1202 (26/05/2015) BBTM 11 A 3 cm 19+0390 – 21+0200: PROJ-1020 (01/06/1993) AC 16 surf S 5 cm 21+0200 – 21+0300: PROJ-1599 (01/06/2012) AC 16 surf S 5 cm

Road Denomination	**BI-624**	
Itinerary	Altube - Balmaseda	
Marker post (KP)	**From:** 64+0440	**To:** 67+0070
Transferred stretches		
-		
General comments:		
It is a part of a regional road in the Plan Peña of 1940, C6210. It connects Balmaseda with Vitoria through the port of Altube, but only a short stretch is located in the province of Biscay. No complete pavement section is known in the entire road. Only two projects are registered: the extension of 5 cm of AC 16 surf S in 2011 under PROJ-134 in the final part (66+0850 – 67+0070), and in the rest of the stretch, the extension of slurry (LB2) under PROJ-939 in 2007.		
Main actuations:		
2007: PROJ-939 (01/06/2007) 64+0440 – 66+0850: LB2 2011: PROJ-134 (01/06/2011) 66+0850 – 67+0070: AC 16 surf S 5 cm		
IRI:		
2000:		
No complete pavement structure is known.		
2004:		
No complete pavement structure is known.		
2007:		
No complete pavement structure is known.		
2011:		
No complete pavement structure is known.		
2016:		
No complete pavement structure is known.		
Skid resistance. SFC:		
2011:		
No complete pavement structure is known.		
2016:		
No complete pavement structure is known.		
Skid resistance of surface layers in 2016		
2016		
64+0440 – 66+0850: PROJ-939 (01/06/2007) LB2. 66+0850 – 67+0070: PROJ-134 (01/06/2011) AC 16 surf S 5 cm.		

Road Denomination	BI-625	
Itinerary	Orduña - Bilbao	
Marker post (KP)	**From:** 351+0390	**To:** 354+0210
	From: 372+0560	**To:** 387+0310
	From: 391+0030	**To:** 392+0440
	From: 395+0000	**To:** 396+0120

Transferred stretches

354+0210 – 372+0560: Province of Álava

387+0310 – 391+0030: Road shared with N-634 (Irubide – Bolueta)

392+0440 – 395+0000: Bilbao (Avda. Zumalakarregi)

General comments:

This road was a National Road, N-625, in the "Plan Peña" of 1940 and linked Bilbao with Madrid through the port of Orduña and Pancorbo. At present, this Preference road links Bilbao with Orduña along the valley of Nervión. The marker posts have high values as the original ones were maintained, indicating the distance to Madrid, since the km 0 is in Madrid.

Orduña is a city of Biscay with is separated from the territory of Biscay, and the territory of the province of Álava must be crossed to reach Orduña from Bilbao.

Some complete pavement sections are known as by-pass of some villages were conducted. The new outlines that were constructed and subsequent rehabilitation and maintenance works are listed below.

- By-pass of Orduña, in 1994: PROJ-314: 351+0390 – 352+0750.
- Improvements and new outlines in some parts in the stretch Areta–Bakiola in 2001: PROJ-144: 373+0480 – 377+0380.
- Access to Martiartu industrial area, in 1992: 381+0300 – 382+0000
- Roundabout of Rezola Cements, in 2015: 382+0280 – 382+0350
- Connection BI-625 – AP-68, in 2014: 382+0500 – 382+0800
- New access to Zaratamo, in 2002: 384+0050 – 384+0800
- Junction of Basauri, BI-625 – A-8, in 1996: Ascending and descending carriageway, 386+0050 – 387+0000.
- Stretch Junction of Zaratamo – Junction of Basauri, in 2016, double carriage

Main actuations:

Bypass of Orduña.

1994: PROJ-314 (01/10/1994)

351+0390 – 352+0750: HMA ophite 6 cm + HMA calcareous 11 cm + MG (ZA) 25 cm

2012: PROJ-316 (01/06/2012)

352+0000 – 352+0120: AC 22 surf S 6 cm + AC 32 bin S 14 cm + MG (ZA) 25 cm

352+0600 – 352+0750:

Improvement on some parts in the stretch Areta – Bakiola

2001: PROJ-144 (01/06/2001)

373+0480 – 374+0050: AC 22 surf D 6 cm + AC 22 bin S 10 cm + GE 30 cm + vitrified slug 15 cm.

374+0350 – 374+0860: AC 22 surf D 6 cm + AC 22 bin S 10 cm + GE 30 cm + vitrified slug 15 cm

375+0350 – 375+0580: AC 22 surf D 6 cm + AC 22 bin S 10 cm + GE 30 cm + vitrified slug 15 cm

375+0710 – 376+0020: AC 22 surf D 6 cm + AC 22 bin S 10 cm + GE 30 cm + vitrified slug 15 cm

377+0150 – 377+0380: AC 22 surf D 5 cm + AC 22 bin S 10 cm + GE 30 cm + vitrified slug 15 cm

2006: PROJ-1968 (01/06/2006)

372+0480 – 382+0180: LB2

2012: PROJ-404 (01/06/2012)

377+0130 – 378+0300: AC 22 surf S 6 cm

Access to Martiartu Industrial Area

1992: PROJ-156 (01/06/1992)

381+0300 – 382+0000: AC 16 surf S 5 cm + AC 22 base G 7 cm + GE 20 cm + MG 30 cm

2006: PROJ-1968 (01/06/2006)

372+0480 – 382+0180: LB2

Roundabout of Rezola Cements 2015: PROJ-412 (13/04/2015) 382+0280 – 382+0350: Ac 22 surf S 6 cm + AC 22 bin S 7 cm + AC 32 base S 16 cm + MG (ZA) 25 cm Connection BI-625 – AP-68 paying motorway 2014: PROJ-401 (16/06/2014) 382+0500 – 382+0800: BBTM 11A 3 cm + AC 22 bin S 10 m + AC 32 base S 20 cm + MG (ZA) 25 cm New access to Zaratamo. 2002: PROJ-410 (01/12/2002) 384+0050 – 384+0800: PA 11 4 cm + AC 22 bin S 13 cm + GE (Gravel-slag) 45 cm 2015: PROJ-2015 (29/09/2015) 383+0900 – 384+0100: BBTM 11A 3 cm + AC 22 bin S 7 cm + AC 22 base S 7 cm + GC 22 cm + SC 22 cm 384+0100 – 384+0900: BBTM 11A 3 cm + AC 22 bin S 7 cm Junction of Basauri (link BI-625 and A-8) 1996: PROJ-411 (31/12/1996) Ascending carriageway, 386+0050 – 387+0000: PA 11 4 cm + AC 16 bin S 6 cm + AC 22 base S 7 cm + Gravel-slag 45 cm Descending carriageway, 387+0000 – 386+0050: PA 11 4 cm + AC 16 bin S 6 cm + AC 22 base S 7 cm + Gravel-slag 45 cm 2006: PROJ-158 (01/06/2006) Ascending carriageway 385+0970 – 387+0240: Remove 4 cm + BBTM 11A 4 cm Descending carriageway 387+0240 – 385+0970: Remove 4 cm + BBTM 11A 4 cm Stretch Junction of Zaratamo – Junction of Basauri, double carriageway. Existing carriageway is the descending one and the new one is the ascending carriageway 2016: PROJ-403 (30/01/2016) Ascending carriage 384+0750 – 386+0040: BBTM 11A 3 cm + AC 22 bin S 7 cm + GC 22 cm + SC 22 cm Descending carriageway 386+0040 – 384+0750: BBTM 11A 3 cm
IRI:
2000:
351+0390 – 352+0750: By-pass of Orduña. IRIm = 1,97. Stretch Areta-Bakiola. Still no new outlines conducted 381+0300 – 382+0000: Martiartu, from 1992, IRIm = 2,02 386+0050 – 387+0000 Ascending carriageway Junction of Basauri, IRIm = 1,87 387+0000 – 386+0050: Descending carriageway Junction of Basauri, IRIm = 2,36
2004:
351+0390 – 352+0750: By-pass of Orduña. No data are available from 2004 Stretch Areta-Bakiola. For each stretch, IRIm = 2,26 ; 2,13 ; 2,61 ; 2,40 ; 2,33. 381+0300 – 382+0000: Martiartu, from 1992, IRIm = 2,11 384+0050 – 384+0800: Access to Zaratamo. IRIm = 1,64 386+0050 – 387+0000 Ascending carriageway Junction of Basauri, IRIm = 2,00 387+0000 – 386+0050: Descending carriageway Junction of Basauri, no data
2007:
351+0390 – 352+0750: By-pass of Orduña. IRIm = 2,47 Stretch Areta-Bakiola. After Maintenance, for each stretch, IRIm = 2,39 ; 2,19 ; 2,42 ; 2,74 ; 2,73. 381+0300 – 382+0000: Martiartu, maintained in 2006, IRIm = 2,13 384+0050 – 384+0800: Access to Zaratamo. IRIm = 1,77 386+0050 – 387+0000 Ascending carriageway Junction of Basauri, Maint IRIm = 1,73 387+0000 – 386+0050: Descending carriageway Junction Basauri, Maint IRIm = 1,77
2011:
351+0390 – 352+0750: By-pass of Orduña. IRIm = 3,77. Stretch Areta-Bakiola. After Maintenance, for each stretch, IRIm = 2,54 ; 2,33 ; 2,53 ; 2,70 ; 3,56. 381+0300 – 382+0000: Martiartu, maintained in 2006, IRIm = 2,20 384+0050 – 384+0800: Access to Zaratamo. IRIm = 1,81 386+0050 – 387+0000 Ascending carriageway Junction of Basauri, Maint IRIm = 1,90 387+0000 – 386+0050: Descending carriageway Junction Basauri, Maint IRIm = 1,87

2016:
351+0390 – 352+0750: By-pass of Orduña. When new pavement structure is carried out, very high values are collected. It corresponds to the two roundabouts, and problems must have occurred when measuring. On the contrary, in the rest of the by-pass, when no works were performed, according to PROJ-316, better values are obtained. An activity was carried out and it was not registered. Therefore, all the data from the bypass are discarded as spurious. Stretch Areta-Bakiola. After Maintenance, for each stretch, IRIm = 2,40 ; 2,57 ; 2,85 ; 3,03 ; and after new maintenance work, IRIm = 2,24. 381+0300 – 382+0000: Martiartu, maintained in 2006, IRIm = 2,60 382+0280 – 382+0350: Roundabout of Rezola Cements. High values after 1 year. Influence of the roundabout geometry. 382+0500 – 382+0800: Connection with AP-68: Some stretches are influenced by the presence of a high roundabout. From 382+0700 is considered. IRIm = 1,66. 384+0050 – 384+0800: Access to Zaratamo. New outline IRIm = 1,41, Rehabilitated IRIm = 1,28 386+0050 – 387+0000 Ascending carriageway Junction of Basauri, Maint IRIm = 1,93 387+0000 – 386+0050: Descending carriageway Junction Basauri, Maint IRIm = 1,94 384+0750 – 386+0040: Ascending carriageway, Stretch junction Zaratamo – Junction Basauri, IRIm = 0,82.
Skid resistance. SFC:
2011:
Stretches that the complete pavement structure is known in 2016 and in 2011 and remain the same. Stretch Areta – Bakiola: 373+0480 – 374+0050 374+0350 – 374+0860 375+0350 – 375+0580 375+0710 – 376+0020 381+0300 – 382+0000: Martiartu 386+0050 – 387+0000 Ascending carriageway Junction of Basauri 387+0000 – 386+0050: Descending carriageway Junction Basauri
2016:
Stretches that the complete pavement structure is known in 2016: Stretch Areta – Bakiola: 373+0480 – 374+0050 374+0350 – 374+0860 375+0350 – 375+0580 375+0710 – 376+0020 377+0150 – 377+0380 381+0300 – 382+0000: Martiartu 382+0280 – 382+0350: Roundabout of Rezola Cements 382+0500 – 382+0800: Connection with AP-68 384+0050 – 384+0800: Access to Zaratamo 386+0050 – 387+0000 Ascending carriageway Junction of Basauri 387+0000 – 386+0050: Descending carriageway Junction Basauri 384+0750 – 386+0040: Ascending carriageway Stretch junction Zaratamo – Junction Basauri,
Skid resistance of surface layers in 2016
2016
352+0190 – 353+0410: PROJ-793 (15/08/2013) AC 16 surf S 5 cm 372+0800 – 383+0480: PROJ-1968 (01/06/2006) LB2 372+0800 – 373+0480: PROJ-404 (01/06/2012) AC 22 surf S 6 cm 374+0050 – 374+0350: PROJ-1968 (01/06/2006) LB2 374+0860 – 375+0350: PROJ-1968 (01/06/2006) LB2 375+0580 – 375+0740: PROJ-1968 (01/06/2006) LB2 376+0020 – 376+0560: PROJ-1968 (01/06/2006) LB2

376+0560 – 376+0720: PROJ-1958 (01/06/2012) AC 16 surf S 5 cm
376+0720 – 377+0130: PROJ-1959 (20/10/2014) BBTM 11A 3 cm
377+0380 – 378+0030: PROJ-404 (01/06/2012) AC 22 surf S 6 cm
378+0030 – 378+0400: PROJ-1960 (21/10/2014) BBTM 11A 3 cm
378+0400 – 378+0920: PROJ-1961 (01/06/2010) AC 16 surf S 5 cm
378+0920 – 379+0050: PROJ-1567 (15/08/2013) AC 16 surf S 5 cm
379+0050 – 379+0310: PROJ-1961 (01/06/2010) AC 16 surf S 5 cm
379+0310 – 379+0360: PROJ-1962 (01/05/2011) AC 16 surf S 5 cm
379+0360 – 379+0720: PROJ-1963 (01/10/2014) AC 16 surf S 5 cm
379+0720 – 379+0880: PROJ- 1964 (22/10/2014) BBTM 11 A 3 cm
379+0880 – 381+0300: PROJ-1968 (01/06/2006) LB2
382+0000 – 382+0100: PROJ-1968 (01/06/2012) LB2
382+0100 – 382+0280: PROJ-412 (13/04/2015) AC 22 surf S 6 cm
382+0350 – 382+0500: PROJ-412 (13/04/2015) AC 22 surf S 6 cm
382+0800 – 383+0900: PROJ-402 (29/04/2015) BBTM 11A 3 cm
387+0000 – 387+0240 Asc. carriageway: PROJ-158 (01/06/2006) BBTM 11A 4 cm
391+0030 – 391+0500 Asc carriageway: PROJ-428 (17/07/2000) BBTM 11A 4 cm
391+0500 - 391+0030: Desc carriageway: PROJ-428 (17/07/2000) BBTM 11A 4 cm
387+0240 – 387+0000 Desc. carriageway: PROJ-158 (01/06/2006) BBTM 11A 4 cm
386+0040 – 384+0350: Desc. Carriageway: PROJ-407 (30/01/2016) BBTM 11A 3 cm

Road Denomination	BI-630	
Itinerary	Balmaseda - Karrantza	
Marker post (KP)	**From:** 30+0270	**To:** 42+0420
	From: 48+0970	**To:** 60+0190

Transferred stretches

29+0000 – 29+0640: Balmaseda

29+0580 – 30+0270: Balmaseda

42+0420 – 48+0970: Cantabria (the road crosses the territory of the Self-governing communityof Cantabria (Trucios)

General comments:

It is another stretch of a regional road in the Plan Peña of 1940, C-6210, which connected Vitoria (Alava) with Ramales de la Victoria (Cantabria). At present this road goes from Balmased to Karrantza and it crosses also part of the territory of Cantabria, and therefore, that part is not managed by the Regional Government of Biscay.

With regard to the pavements, no complete pavement section is known in the entire road. Rehabilitation and maintenance works are recorded to have been performed, but no new outlines have been carried out. It serves areas with low density population and the original outline of the old road remains, with many curves through two mountain passes. Only surface layer are known, but not in the entire length of the road. Consequently, a fiction project is introduced in the PMS (PROJ-2066). As the road was paved before being managed by the Regional Government of Biscay, it is indicated that 5 cm of AC 16 surf S was extended over the entire length of the road. Thus, a minimum surface layer is supposed. Visual observations of existing pavement in that time suggested that AC 16 surf S was on the surface. However, samples obtained in 2000 showed wider bituminous layers, but as for a minimum, it serves.

.

Main actuations:

1987: PROJ-164 (01/01/1987)

56+0300 – 60+0190: AC 16 surf S 5 cm + AC 22 base G 5 cm (regulation)

2011: PROJ-1573 (01/01/2011)

37+0520 – 38+0050: AC 16 surf S 5 cm

IRI:

2000:

No complete pavement structure is known.

2004:

No complete pavement structure is known.

2007:

No complete pavement structure is known.

2011:

No complete pavement structure is known.

2016:

No complete pavement structure is known.

Skid resistance. SFC:

2011:

No complete pavement structure is known.

2016:

No complete pavement structure is known.

Skid resistance of surface layers in 2016

2016

Only stretches were rehabilitation or maintenance works have been carried out are introduced in the calculation. Where no works were carried out, it has the fiction project as the only data about pavement, but it is not used.

30+0270-30+0520: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm

30+0520 – 30+0650: PROJ-952 (01/06/2005) LB2

30+0350 - 30+0720: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm

30+0720 – 30+0860: PROJ-953 (01/06/2005) LB2
30+0860 – 31+0140: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
31+0140 – 31+0310: PROJ-954 (01/06/2005) LB2
31+0310 – 33+0120: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
33+0120 – 33+0340: PROJ-955 (01/06/2005) LB2
33+0340 – 35+0120: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
35+0120 – 35+0340: PROJ-957 (01/06/2005) LB2
35+0340 – 35+0500: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
35+0500 – 35+0590: PROJ-958 (01/06/2005) LB2
35+0590 – 35+0900: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
35+0900 – 35+1000: PROJ-960 (01/06/2005) LB2
35+1000 – 36+0190: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
36+0190 – 36+0230: PROJ-964 (01/06/2005) LB2
36+0230 – 36+0280: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
36+0280 – 36+0350: PROJ-967 (01/06/2005) LB2
36+0350 – 36+0910: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
36+0910 – 36+0990: PROJ-971 (01/06/2005) LB2
36+0990 – 37+0200: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
37+0200 – 37+0250: PROJ-973 (01/06/2005) LB2
37+0250 – 37+0480: PROJ-1572 (01/06/2012) AC 16 surf S 5 cm
37+0480 – 37+0520: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
37+0520 – 38+0050: PROJ-1573 (01/06/2011) AC 16 surf S 5 cm
38+0050 – 38+0520: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
38+0520 – 38+0620: PROJ-1574 (01/06/2011) AC 16 surf S 5 cm
38+0620 – 38+0860: PROJ-978 (01/06/2005) LB2
38+0860 – 38+0950: PROJ-1575 (01/06/2011) AC 16 surf S 5 cm
38+0970 – 39+0060: PROJ-980 (01/06/2005) LB2
39+0060 – 39+0220: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
39+0220 – 36+ 0360: PROJ-981 (01/06/2005) LB2
39+0360 – 40+0000: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
40+0000 – 40+0110: PROJ-982 (01/06/2005) LB2
40+0110 – 42+0420: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
48+0970 – 49+0880: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
49+0880 – 50+0080: PROJ-1785 (30/10/2015) AC 16 surf S 6 cm
50+0080 – 50+0660: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
50+0660 – 51+0900: PROJ-984 (15/08/2016) AC 16 surf S 5 cm
51+0900 – 52+0500: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
52+0500 – 53+0210: PROJ-985 (01/01/2000) LB2
53+0210 – 53+0400: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
53+0400 – 53+0730: PROJ-986 (01/01/2000) LB2
53+0730 – 54+0100: PROJ-987 (15/08/2014) AC 16 surf S
54+0100 – 56+0010: PROJ-988 (01/01/2000) LB2
56+0010 – 56+0300: PROJ-2066 (01/01/2000) AC 16 surf S 5 cm
56+0300 – 60+0190: PROJ-164 (01/06/1989) AC 16 surf S

Road Denomination	**BI-712**	
Itinerary	Basauri - Bolueta	
Marker post (KP)	**From:** 387+0680	**To:** 388+0550

Transferred stretches

382+0530 – 384+0350: Intersection with BI-625 – Basauri
384+0350 – 387+0680: San Miguel and Basauri
388+0550 – 389+0050: Bolueta

General comments:

This road was a part of the N-625. Nowadays it is complementary road, with the main part crosses urban areas and it is transferred to municipalities.

In the part managed by the Regional Government of Biscay, the oldest project conserved is from 1993 and detailed the different bituminous layers extended over existing pavements (PROJ-400).

There is a maintenance work registered in 2007 in the entire road, where slurry was extended (PROJ-2008). Later, in 2008 a layer of 5 cm of S-12 was extended in 388+0240 – 388+0420 (PROJ-2009) and in 2015. A layer of 6 cm of S-12 was extended in 388+0100 – 388+0240 (PROJ-1756).

Main actuations:

1993: PROJ-400 (01/06/1993)
387+0680 – 388+0100: AC 16 surf S 5 cm + AC 22 bin S 7 cm + AC 22 base G 12 cm + MG (ZA) 25 cm
388+0100 – 388+0550: AC 16 surf S2 5 cm + G20 7 cm
2007: PROY-2008 (01/06/2007)
387+0690 – 388+0550: LB2
2008: PROY-2009 (01/06/2008)
388+0240 – 388+0480: AC 16 surf S 5 cm
2015: PROY-1756 (30/10/2015)
388+0110 – 388+0240: AC 16 surf S 6 cm

IRI:

2000:
Stretch 387+0680 – 388+0100: IRIm = 2,47.
2004:
Stretch 387+0680 – 388+0100: IRIm = 2,06. Sections with improvements are discarded. New IRIm = 2,59
2007:
Stretch 387+0680 – 388+0100: IRIm = 2,14 (After maintenance work)
2011:
Stretch 387+0680 – 388+0100: IRIm = 2,16
2016:
Stretch 387+0680 – 388+0100: IRIm = 2,62

Skid resistance. SFC:

2011:
Stretch 387+0680 – 388+0100.
2016:
Stretch 387+0680 – 388+0100.

Skid resistance of surface layers in 2016

2016
388+0100 – 388+0240: PROJ-1756 (30/10/2015) AC 16 surf S 6 cm
388+0240 – 388+0480: PROJ-2009 (01/06/2008) AC 16 surf S 5 cm
388+0480 – 388+0550: PROJ-2008 (01/06/2007) LB2

Road Denomination	**BI-745**	
Itinerary	Barakaldo - Trapagaran	
Marker post (KP)	**From:** 9+0090	**To:** 9+0130
	From: 9+0180	**To:** 11+0580

Transferred stretches

8+0470 – 9+0030: San Vicente (Barakaldo)

9+0130 – 9+0180: Intersection with BI-644 (roundabout)

General comments:

This road links two important industrial cities near Bilbao and it is registered when it was constructed. The first part, from San Vicente (Barakaldo) to the crossroad of Babcok Wilkox, was conducted in 1932, and the second part from that crossroad to Trapagaran in 1941, when important factories, mainlty Babco Wilcox, were installed in this area.

No completed pavement section is known. There are only 3 projects registered. The only one which is not a maintenance work, PROJ-1892, is related to the part 3A of the Metropolitan South Bypass of Bilbao, and a new roundabout was conducted. In small parts new pavement section was performed, buy so short, that only the extension of two bituminous layers are considered in the PMS.

Main actuations:

2005: PROJ-2004 (01/06/2005)

9+0210 – 11+0580: LB2

2008: PROJ-2010 (01/06/2008)

9+0090 – 9+0210: Remove 5 cm + AC 16 surf S 5 cm

2010: PROJ-1892 (30/09/2010)

10+0400 – 11+0580: AC 16 surf S 4 cm + S12 5 cm

IRI:

2000:

No complete pavement structure is known.

2004:

No complete pavement structure is known.

2007:

No complete pavement structure is known.

2011:

No complete pavement structure is known.

2016:

No complete pavement structure is known.

Skid resistance. SFC:

2011:

No complete pavement structure is known.

2016:

No complete pavement structure is known.

Skid resistance of surface layers in 2016

2016

9+0090 – 9+0210: PROJ-2010 (01/06/2008) AC 16 surf S 5 cm

9+0210 – 10+0400: PROJ-2004 (01/06/2005) LB2.

10+0400 – 10+0550: PROJ-1892 (30/09/2010) AC 16 surf 4 cm.

10+0550 – 11+0580: PROJ-2004 (01/06/2005) LB2.

Road Denomination	**BI-2521**	
Itinerary	Orduña – Altube	
Marker post (KP)	**From:** 40+0060	**To:** 44+0520
Transferred stretches		
-		
General comments:		
In the historic library, it is registered that this road was constructed around 1920 to communicate Orduña with Altube through the mountain pass of La Barrerrilla. There are a project of 1956 about the road reparation and another project from 1967 about the pavement reparation and road cleaning. In 2010 the part corresponding to the port (inside the province of Biscay) is rehabilitated, 41+0650 – 44+0520 (PROJ-309). There are a rehabilitation project from 2012 in the stretch 40+0060 – 40+0180 (PROJ-316) and another one from 2016 (PROJ-1757).		
Main actuations:		
2010: PROJ-309 (01/06/2010) 41+0650 – 41+01000: LB2 42+0000 – 44+0520: Remove 5 cm + AC 16 surf S 5 cm 2012: PROJ-316 (01/06/2012) 40+0060 – 40+0180: AC 22 surf S 6 cm 2016: PROJ-1757 (28/04/2016) 40+0180 – 41+0650: AC 16 surf S 6 cm		
IRI:		
2000:		
The complete pavement section is unknown in the entire road.		
2004:		
The complete pavement section is unknown in the entire road.		
2007:		
The complete pavement section is unknown in the entire road.		
2011:		
The complete pavement section is unknown in the entire road.		
2016:		
The complete pavement section is unknown in the entire road.		
Skid resistance. SFC:		
2011:		
The complete pavement section is unknown in the entire road.		
2016:		
The complete pavement section is unknown in the entire road.		
Skid resistance of surface layers in 2016		
2016		
40+0060 – 40+0180: PROJ-316 (01/06/2012) AC 22 surf S 6 cm 40+0180 – 41+0650: PROJ-1757 (28/04/2016) AC 16 surf S 6 cm 41+0650 – 41+0100: PROJ-309 (01/06/2010) LB2 42+0000 – 44+0520: PROJ-309 (01/06/2010) AC 16 surf S 5 cm		

Road Denomination	**BI-2522**	
Itinerary	Areta – Ziorraga (through Orozko)	
Marker post (KP)	**From:** 20+0320	**To:** 30+0030
Transferred stretches		
-		

General comments:
The first documents about works in this road were some repairs due to the floods in 1983 (PROJ-395 and PROJ-108). The only project that provides a complete pavement section is PROJ-310 (01/06/1997), the bypass of Orozko. Other projects are rehabilitation and maintenance works.
Main actuations:
1984: PROJ-395 (01/06/1984) 20+0320 – 23+0800: Repairs to damages due to floods. Real works are unknown 1985: PROJ-108 (01/12/1985) 25+0100 – 28+0900: AC 16 surf S + AC 22 base G 6 cm 1997: PROJ-310 (01/06/1997) 23+0800 – 25+0180: AC 16 surf S 6 cm + G20 7 cm + GEscoria 25 cm + Unbound material (ZA) 20 cm 2005: PROJ-109 (01/06/2005) 20+0320 – 22+0000: AC 16 surf S 6 cm + AC 25 base G 7 cm 22+0220 – 22+0500: AC 16 surf S 6 cm + AC 25 base G 7 cm 2007: PROJ-110 (01/06/2007) 21+0900 – 22+0220: AC 16 surf S 5 cm 22+0500 – 23+0920: AC 16 surf S 5 cm 2008: PROJ-2005 (01/06/2008) 25+0180 – 30+0020: AC 16 surf S 5 cm
IRI:
2000:
23+0800 – 25+0180: Data from bypass of Orozko are introduced. IRIm = 2,18
2004:
23+0800 – 25+0180:IRIm = 2,02. Global improvement is observed and almost all the section decrease IRI values. Data are discarded as spurious.
2007:
23+0800 – 25+0180: IRIm = 2,20. Data is worse than in 2004, but almost similar to 2000, after 7 years. Data are discarded as spurious.
2011:
23+0800 – 25+0180: IRIm = 1,87. Values are even better than at 2000. Rehabilitation or maintenance works might have performed and they have not been registered. The rehabilitation work registered in PROJ-110, which takes the first 120 m are better, but other improvements are observed along the stretch. Data are not used.
2016:
23+0800 – 25+0180: IRIm = 2,18. Data are worse than in 2011 but similar to 2000. This indicates that rehabilitation works were carried out before 2011 but they were not recorded. Data are not used.
Skid resistance. SFC:
2011:
Works were performed in the stretch in which the complete section was known. Data are not used as it is unknown the real pavement structure..
2016:
Works were performed in the stretch in which the complete section was known. Data are not used as it is unknown the real pavement structure.
Skid resistance of surface layers in 2016
2016
20+0320 – 21+0900: PROJ-109 (01/06/2005) AC 16 surf S 6 cm 22+0000 – 22+0220: PROJ-110 (01/06/2007) AC 16 surf S 5 cm

22+0220 – 22+0500: PROJ-109 (01/06/2005) AC 16 surf S 6 cm
22+0500 – 23+0920: PROJ-110 (01/06/2007) AC 16 surf S 5 cm
23+0920 – 25+0180: PROJ-310 (01/06/1997) AC 16 surf S 5 cm
25+0180 – 30+0020: PROJ-2005 (01/06/2008) AC 16 surf S 5 cm

Road Denomination	**BI-2617**	
Itinerary	Villaverde de Trucios - Trucios	
Marker post (KP)	**From:** 43+0000	**To:** 45+0400
Transferred stretches		
41+0630 – 41+1990: Border with Cantabria - Trucios		
General comments:		
It is a road in a remote rural area. There is not project recorded at least from 2000. It is supposed that some layer extension could have been conducted during the “Pavement Plan” (Plan de Firmes) in 1990. It the PMS, it is introduced a fiction project in 1990 to reflect that possible work, but it needs to be confirmed. Therefore, no exact project is known in this road.		
Main actuations:		
IRI:		
2000:		
No pavement section is known..		
2004:		
No pavement section is known..		
2007:		
No pavement section is known..		
2011:		
No pavement section is known..		
2016:		
No pavement section is known..		
Skid resistance. SFC:		
2011:		
No pavement section is known..		
2016:		
No pavement section is known..		
Skid resistance of surface layers in 2016		
2016		
No real project in this road is recorded in the PMS. Consequently, no surface layer is known.		

Road Denomination	**BI-2625**	
Itinerary	Montain pass of Orduña - Orduña	
Marker post (KP)	**From:** 350+0580	**To:** 351+0370

Transferred stretches

-

General comments:

It is a short road, which was a part of a national road, N-625. Nowadays, it links the final part of the BI-625 Bilbao-Orduña with the border with the province of Araba.

In 1994 the Bypass of Orduña was carried out and related to the final roundabout, a new pavement section was performed in the stretch 351+0240 – 351+0370 (PROJ-314).

In 2008 there was a rehabilitation of the stretch 350+0580 – 351-0240, with 5 cm of AC 16 surf S (PROJ-1559). In 2010, there was an additional rehabilitation in the stretch 350+0580 – 350+0690 with 5 cm of AC 16 surf S (PROJ-1974).

In 2012 the rehabilitation of the roundabouts of the Bypass of Orduña is finished, and additional layers are added to stretch 351+0240 – 351+0370 (PROJ-316)

Main actuations:

1994: PROJ-314 (01/10/1994)
- 351+0240 – 351+0370: AC 16 surf S 6 cm + AC 22 bin S 11 cm + MG (ZA) 25 cm

2008: PROJ-1559 (01/06/2008)
- 350+0580 – 351+0240: AC 16 surf S 5 cm

2010: PROJ-1974 (01/06/2010)
- 350+0580 – 351+0690: AC 16 surf S 5 cm

2012: PROJ-316 (01/06/2012)
- 351+0240 – 351+0370: AC 22 surf S 6 cm + AC 32 bin S 14 cm

IRI:

2000:
- 351+0240 – 351+0370, as new outline

2004:
- No data from 2004

2007:
- 351+0240 – 351+0370, as new outline

2011:
- 351+0240 – 351+0370, as new outline

2016:
- No IRI data were collected in stretch 351+0240 – 351+0370.

Skid resistance. SFC:

2011:
- 351+0240 – 351+0370, different pavement to the one of 2016.

2016:
- 351+0240 – 351+0370, as rehabilitated pavement.

Skid resistance of surface layers in 2016

2016
- 350+0580 – 350+0690: PROJ-1974 (01/06/2010) AC 16 surf S 5 cm
- 350+0690 – 351+0240: PROJ-1559 (01/06/2008) AC 16 surf S 5 cm

Road Denomination	**BI-2701**	
Network		
Itinerary	Muskiz – Malabrigo (through Sopuerta)	
Marker post (KP)	**From:** 20+0120	**To:** 35+0700
Transferred stretches		
-		

General comments:

The road goes along the valley of the river Barbadun. There are some stretches in which the initial pavement structure is known, after a total reconstruction of new alignments or new crossroads were made.

The first project in which a complete section is known is PROJ-115, in 1988. After that, PROJ-117 the rehabilitation of many stretches and a new complete pavement structure in other stretches.

More recently, two projects related with the construction of two roundabouts implied new pavement structures: PROJ-312 (30+0450 – 30+0550) in 2010 and PROJ-313 (32+0800 – 33+0290) in 2011.

The rest of works are rehabilitation or maintenance works.

In 1984 there is a project that takes into the account the works carried out after the floods in 1983, but there is no information about it, and it is only mentioned in the database (PROJ-395).

Main actuations:

1988: PROJ-115 (01/06/1988)
- 27+0500 – 27+0900: Ophite HMA 5 cm + HMA 10 cm + GEs 30 cm + MG (ZA) 15cm

1994: PROJ-117 (01/06/1994)
- 27+0580 – 27+0960: AC 16 surf S 6 cm
- 27+0000 – 27+0580: AC 16 surf S 5 cm + AC 22 bin G 7 cm + AC 22 base G 7 cm + MG (ZA) 25 cm
- 26+0200 – 27+0000: AC 16 surf S 5 cm
- 25+0900 – 26+0200 AC 16 surf S 5 cm + AC 22 bin G 7 cm + AC 22 base G 7 cm + MG (ZA) 25 cm
- 25+0400 – 25+0900: AC 16 surf S 5 cm
- 23+0600 – 25+0400: AC 16 surf S 5 cm + AC 22 bin G 7 cm + AC 22 base G 7 cm + MG (ZA) 25 cm
- 23+0150 – 23+0600: AC 16 surf S 5 cm
- 22+0600 – 23+0150: AC 16 surf S 5 cm + AC 22 bin G 7 cm + AC 22 base G 7 cm + MG (ZA) 25 cm
- 21+0100 – 22+0600: AC 16 surf S 5 cm

2004: PROJ-116 (01/12/2004)
- 28+0050 – 28+0500: LB2
- 24+0300 – 27+0550: LB2
- 27+0750 – 28+0050: AC 16 surf S 5 cm + AC 16 bin S 4 cm
- 24+0000 – 24+0300: AC 16 surf S 5 cm + AC 16 bin S 4 cm
- 21+0640 – 23+0000: AC 16 surf S 5 cm + AC 16 bin S 4 cm
- 27+0550 – 27+0750: AC 16 surf S 5 cm
- 23+0000 – 24+0000: AC 16 surf S 5 cm
- 21+0000 – 23+0000: AC 16 surf S 5 cm + AC 16 bin S 4 cm

2010: PROJ-1561 (01/06/2010)
- 21+0050 – 21+0190: AC 16 surf S cm

2010: PROJ-312 (19/07/2010)
- 30+0450 – 30+0550: AC 16 surf S 6 cm + AC 22 bin S 10 cm + MG (ZA) 40 cm

2011: PROJ-313 (01/06/2011)
- 32+0800 – 33+0290: AC 16 surf 4 cm AC 22 bin S 6 cm + AC 22 base G 7 cm + MG (ZA) 25 cm

2012: PROJ-1560 (01/06/2012)
- 20+0120 – 21+0050: BBTM 11A 3 cm

2012: PROJ-1562 (01/06/2012)
- 28+0260 – 29+0280: BBTM 11A 3 cm

2014: PROJ-1563 (15/06/2014)
- 29+0850 – 30+0280

IRI:

2000:

22+0600 – 23+0150: New pavement structure from 1994, 19 cm HMA. IRIm = 1,76

23+0600 – 25+0400: New pavement structure from 1994, 19 cm HMA. IRIm = 1,76

25+0900 – 26+0200: New pavement structure from 1994, 19 cm HMA. IRIm = 1,78

27+0000 – 27+0550. New pavement structure from 1994, 19 cm HMA. IRIm = 1,49

2004:

22+0600 – 23+0150: Rehabilitation in 2004. However, it seems that the correct works are not introduced, as in the stretch 22+0600 – 23+0000, two rehabilitation works are registered, in which 9 cm of AC 16 surf S are extended. This stretch is discarded as spurious.

23+0600 – 24+0000: Rehabilitated with 5 cm of AC 16 surf S in 2004. Improvements are shown, so maintenance works were performed before data collection. IRIm = 1,58

24+0000 – 24+0300: Rehabilitated with 9 cm of AC 16 surf S in 2004. Improvements are shown, so maintenance works were performed before data collection. IRIm = 1,65

24+0300 – 25+0400. Slurry in 2004. Improvements are shown, so maintenance works were performed before data collection. IRIm = 1,84

25+0900 – 26+0200: Slurry in 2004. Improvements are shown, so maintenance works were performed before data collection. IRIm = 1,73

27+0000 – 27+0550. Slurry in 2004. Improvements are shown, so maintenance works were performed before data collection. IRIm = 1,45.

2007:

23+0600 – 24+0000: Rehabilitated with 5 cm of AC 16 surf S in 2004. IRIm = 1,65

24+0000 – 24+0300: Rehabilitated with 9 cm of AC 16 surf S in 2004. IRIm = 1,67

24+0300 – 25+0400. Maintained in 2004. IRIm = 2,45

25+0900 – 26+0200: Maintained in 2004. IRIm = 1,93

27+0000 – 27+0550. Maintained in 2004. IRIm = 1,92

2011:

23+0600 – 24+0000: Rehabilitated with 5 cm of AC 16 surf S in 2004. IRIm = 1,43

24+0000 – 24+0300: Rehabilitated with 9 cm of AC 16 surf S in 2004. IRIm = 1,13

24+0300 – 25+0400. Maintained in 2004. IRIm = 2,14

As seen, in the 3 previous stretches individual and global improvements are recorded and only slurry is applied in stretch 24+0310 – 24+0880 in 2012. These data and following data are not introduced in the modelling.

25+0900 – 26+0200: Maintained in 2004. IRIm = 2,01

27+0000 – 27+0550. Maintained in 2004. IRIm = 1,96

30+0450 – 30+0550. New pavement section. IRI m = 4,02 due to roundabout geometry. Discarded.

32+0800 – 33+0290. New pavement section in 2011. Data from the length of the roundabout are discared. In the rest, IRIm = 1,87

2016:

25+0900 – 26+0200: Maintained in 2004. IRIm = 2, 18.

27+0000 – 27+0550. Maintained in 2012. IRIm = 2,11.

32+0800 – 33+0290. New pavement section in 2011. Data from the length of the roundabout are discarded. In the rest, IRIm = 1,77

Skid resistance. SFC:

2011:

25+0900 – 26+0200 (data collected in summer)

2016:

Stretches that are included in the modelling with data from 2016 are introduced.

25+0900 – 26+0200.

27+0000 – 27+0550.

32+0800 – 33+0290.

Skid resistance of surface layers in 2016
2016
20+0120 – 21+0050: PROJ-1560 (01/06/2012) BBTM 11A 3 cm 21+0050 – 21+0190: PROJ-1561 (01/06/2010) AC 16 surf S 5 cm 21+0190 – 22+0200: PROJ-921 (01/07/2012) LB2 22+0200 – 23+0200: PROJ-166 (01/07/2004) AC 16 surf S 5 cm 23+0200 – 23+0510: PROJ-922 (01/07/2012) LB2 23+0600 – 24+0300: PROJ-116 (01/07/2014) AC 16 surf S 5 cm 24+0310 – 24+0890: PROJ-923 (01/07/2012) LB2 24+0890 – 25+0900: PROJ-116 (01/07/2004) LB2 26+0200 – 26+0740: PROJ-116 (01/07/2004) LB2 26+0740 – 26+1020: PROJ-926 (01/07/2012) LB2 27+0550 – 27+0840: PROJ-926 (01/07/2012) LB2 27+0840 – 28+0050: PROJ-116 (01/07/2004) AC 16 surf S 28+0050 – 28+0160: PROJ-116 (01/07/2004) LB2 28+0160 – 28+0260: PROJ-927 (001/06/2009) AC 16 surf S 28+0260 – 29+0280: PROJ-1562 (01/06/2009) BBTM 11A 3 cm 29+0280 – 29+0850: PROJ-1864)15/06/2014) AC 16 surf S 5 cm 29+0850 – 30+0280: PROJ-1563 (15/06/2014) BBTM 11A 3 cm 30+0280 – 30+0450: PROJ-1865 (15/06/2014) AC 16 surf S 5 cm 30+0450 – 30+0550: PROJ-312 (19/07/2010) AC 16 surf S 6 cm 30+0550 – 32+0800: PROJ-931 (01/07/2012) LB2 33+0290 – 33+0380: PROJ-932 (01/07/2012) LB2 33+0380 – 33+0480: PROJ-2006 (20/03/2014) AC 16 surf S 6 cm 33+0480 – 34+0640: PROJ-934 (01/07/2012) LB2 34+0640 – 35+0340: PROJ-395 (01/06/2003) LB2 35+0370 – 35+0490: PROJ-936 (01/06/2003) LB2

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-2757</td></tr>
<tr><td>Itinerary</td><td colspan="2">Gallarta – La Arboleta</td></tr>
<tr><td rowspan="2">Marker post (KP)</td><td>From: 0+0000</td><td>To: 1+0120</td></tr>
<tr><td>From: 2+0820</td><td>To: 6+0830</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">1+0120 – 2+0820: El Campillo</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">It is a recently constructed road, carried out in different stretches, 3, from 2004. The 3 construction projects of the 3 stretches are registered (PROJ-11, PROJ-12 and PROJ-14).
There are no rehabilitation projects.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3">2004: PROJ-11 (07/09/2004)
4+0700 – 6+0820: AC 16 surf S 5 cm + AC 22 bin S 5 cm + Gravaescoria 25 cm
2008: PROJ-12 (18/06/2008)
0+0000 – 1+0120: AC 16 surf S 6 cm + AC 22 bin S 6 cm + GC 33 cm
2008: PROJ-14 (01/11/2008)
2+0820 – 4+0700: AC 16 surf S 5 cm + AC 22 bin S 5 cm + GC 30 cm</td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">2000:</td></tr>
<tr><td colspan="3">The road was not constructed and, hence, there are no data.</td></tr>
<tr><td colspan="3">2004:</td></tr>
<tr><td colspan="3">The road was not constructed and, hence, there are no data.</td></tr>
<tr><td colspan="3">2007:</td></tr>
<tr><td colspan="3">4+0700 – 6+0830: IRIm = 2,50 / 2,58</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">0+0000 – 1+0120: IRIm = 2,05
2+0820 – 4+0700: IRIm = 1,36 (some sections were removed, as sonorous band were displayed to reduce speed in the way down).
4+0700 – 6+0830: IRIm = 2,31</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">0+0000 – 1+0120: IRIm = 2,16
2+0820 – 4+0700: IRIm = 1,43
4+0700 – 6+0830: IRIm = 2,50 (3 sections are removed as they have better values).</td></tr>
<tr><td colspan="3">Skid resistance. SFC:</td></tr>
<tr><td colspan="3">2011:</td></tr>
<tr><td colspan="3">The three stretches are introduced.</td></tr>
<tr><td colspan="3">2016:</td></tr>
<tr><td colspan="3">The three stretches are introduced.</td></tr>
<tr><td colspan="3">Skid resistance of surface layers in 2016</td></tr>
<tr><td colspan="3">2016</td></tr>
<tr><td colspan="3">The entire road is introduced in the calculation with complete pavement section. There is no unknown pavement section.</td></tr>
</table>

Road Denomination	**BI-2794**	
Itinerary	Junction A-8 – La Arena	
Marker post (KP)	**From:** 23+0050	**To:** 24+0768
Transferred stretches		
-		

General comments:
It is a short road that links a junction of the A-8 freeway with a local road to a part of the Port of Bilbao, bypassing an area related to a big beach. The first part of the road (23+0050 – 23+0850) existed for long and the second part, the bypass of the La Arena beach was conducted under PROJ-329 in 2004. The first part was reinforced in the same project. Later, in 2013, there were some repairs due to bridge joints.
Main actuations:
2004: PROJ-329 (01/06/2004) 23+0050 – 23+0850: Remove 4 cm + AC 16 surf S 6 cm + AC 22 bin S 6 cm + AC 22 base G 8 cm 23+0850 – 24+0768: AC 16 surf S 6 cm + AC 22 bin S 6 cm + AC 22 base G 13 cm + MG (ZA) 25 cm 2013: PROJ-937 (15/07/2013) 24+0050 – 24+0080: AC 16 surf S 5 cm 2013: PROJ-1564 (15/07/2013) 24+0550 – 24+0590: AC 16 surf S 5 cm
IRI:
2000:
There are no data from 2000.
2004:
Some months after the construction of the second part of the road, data are collected. Stretch 23+0850 – 24+0768 is introduced in the deterioration modelling as new outline, but eliminating stretches that have bridge joints (23+00950 – 24+0060 and 24+0560 – 24+0660), which were repaired in 2013, because their performance is not related to pavement structure deterioration, but to the joint evolution.
2007:
Data in 2007 show high values. Subsequent years do not show this progression, and values are lower and no maintenance or rehabilitation works are recorded to be conducted. Values in 2011 are lower, and in 2016 higher than in 2011, but still lower than in 2007. Therefore, if it is admitted that no works were carried out since the construction in 2004, data from 2007 must be discarded as spurious, in order to underline that values from 2011 and 2016 are correct.
2011:
Data of stretch 23+0850 – 24+0768 from 2011 are introduced in the modelling.
2016:
Data of stretch 23+0850 – 24+0768 from 2016 are introduced in the modelling.
Skid resistance. SFC:
2011:
Data of stretch 23+0850 – 24+0768 from 2011 are introduced in the modelling. Data were collected in summer.
2016:
Data of stretch 23+0850 – 24+0768 from 2016 are introduced in the modelling. Stretches 24+0050 – 24+0080 and 24+0550 + 24+0590 are introduced as maintained.
Skid resistance of surface layers in 2016
2016
23+0050 – 23+0850: PROJ-329 (01/06/2004) AC 16 surf S 6 cm.

A.2.4. Conservation Area 4

Road Denomination	**A-8**	
Itinerary	Motorway of the Bay of Biscay	
Marker post (KP)	**From:** 106+0040	**To:** 139+0220 (ascending carriageway)
	From: 139+0220	**To:** 106+0040 (descending carriageway)
Transferred stretches		
-		
General comments:		
It is the main motorway of Biscay. It was constructed in 70s. Not all the length is managed directly by the Regional Government of Biscay. The paying section is managed by Interbiak, a public company, dependent on the RGB.		
Main actuations:		
Skid resistance of surface layers in 2016		
2016		

Ascending carriageway:
106+0040 – 107+0100: PROJ-505 (01/06/2008) PA11 4 cm
107+0100 – 108+0210: PROJ-44 (16/04/2007) PA11 4 cm
108+0210 – 108+0720: PROJ-2037 (01/06/2014) BBTM 11A 4 cm
108+0720 – 109+0400: PROJ-44 (16/04/2007) PA11 4 cm
109+0400 – 109+0900: PROJ-37 (01/06/2007) BBTM 11A 4 cm
109+0900 – 110+0290: PROJ-1804 (26/05/2015) BBTM 11A 4 cm
110+0290 – 110+0970: PROJ-1888 (01/06/2006) BBTM 11A 3 cm
111+0000 – 111+0570: PROJ-411 (31/12/1996) PA11 4 cm
111+0570 – 112+1000: PROJ-1836 (24/02/2007) BBTM 11A 3 cm
113+0000 – 113+0900: PROJ-1889 (01/06/2006) BBRM 11A 3 cm
113+0900 – 113+0990: PROJ-458 (01/06/1990) AC surf 16 D 5 cm
113+0990 – 114+0400: PROJ-503 (15/09/2010) BBTM 11A 3 cm
114+0400 – 114+0790: PROJ-1908 (15/04/2016) Micro-milling from BBTM 11A 3 cm
114+0790 – 115+0600: PROJ-1922 (20/10/2016) BBTM 11A 3 cm
115+0600 – 115+0800: PROJ-436 (29/04/1995) PA 11 4 cm
115+0800 – 116+0600: PROJ-1338 (17/01/2012) BBTM 11A 3 cm
116+0600 – 117+0800: PROJ-1841 (18/03/2013) BBTM 11A 3 cm
117+0800 – 119+0400: PROJ-456 (18/05/2013) BBTM 11A 3 cm
119+0400 – 120+0540: PROJ-1783 (18/05/2013) BBTM 11A 4 cm
120+0540 – 120+0800: PROJ-374 (20/12/2004) BBTM 11A 3 cm
120+0800 – 121+0210: PROJ-1726 (12/01/2012) BBTM 11A 3 cm
121+0210 – 122+0200: PROJ-374 (20/12/2004) BBTM 11A 3 cm
122+0200 – 122+0750: PROJ-1783 (18/05/2013) BBTM 11A 4 cm
122+0750 – 124+0890: PROJ-374 (20/12/2004) BBTM 11A 3 cm
124+0890 – 125+0530: PROJ-2058 (01/01/2001) BBTM 11A 4 cm
125+0530 – 125+0740: PROJ-419 (25/06/1984) HMA Ophitic 5 cm
125+0740 – 126+0050: PROJ-455 (15/09/2010) BBTM 11A 4 cm
126+0050 – 126+0500: PROJ-455 (15/09/2010) BBTM 11A 3 cm
129+0500 – 129+0780: PROJ-453 (23/07/2010) BBTM 11A 3 cm
129+0780 – 130+0190: PROJ-447 (20/11/1987) AC 22 surf S 6 cm
130+0190 – 130+0890: PROJ-1891 (23/09/2016) BBTM 11A 4 cm
130+0890 – 131+0600: PROJ-1728 (20/11/1987) AC 22 surf S 6 cm
131+0600 – 134+0990: PROJ-1843 (01/08/2015) BBTM 11A 4 cm
135+0000 – 136+0390: PROJ-1824 (01/04/2014) BBTM 11A 4 cm
136+0390 – 136+0850: PROJ-1752 (27/10/2014) BBTM 11A 3 cm

136+0850 – 138+0320: PROJ-468 (10/09/2000) BBTM 11A 4 cm
138+0320 – 139+0220: PROJ-1845 (15/09/2015) BBTM 11A 3 cm

Descending carriageway
139+0220 – 138+0140: PROJ-1823 (24/10/2014) BBTM 11A 3 cm
138+0140 – 136+0390: PROJ-1796A (10/09/2000) BBTM 11A 4 cm
136+0390 – 135+0000: PROJ-1796B (01/04/2014) BBTM 11A 4 cm
135+0000 – 134+0100: PROJ-1753 (30/10/2015) BBTM 11A 3 cm
134+0100 – 133+0070: PROJ-1847 (31/07/2014) BBTM 11A 4 cm
133+0070 – 131+0600: PROJ-1753 (30/10/2015) BBTM 11A 3 cm
131+0600 – 130+0960: PROJ-1729 (30/05/2001) LB2
130+0960 – 130+0800: PROJ-447 (20/11/1987) AC 22 surf S 6 cm
130+0800 – 127+0520: PROJ-453 (23/07/2010) BBTM 11A 3 cm
127+0520 – 127+0300: PROJ-454 (23/07/2010) BBTM 11A 3 cm
127+0300 – 127+0000: PROJ-455 (15/09/2010) BBTM 11A 3 cm
127+0000 – 126+0050: PROJ-1852 (03/07/2015) BBTM 11A 3 cm
126+0050 – 125+0630: PROJ-455 (15/09/2010) BBTM 11A 3 cm
125+0630 – 123+0190: PROJ-2057 (01/01/2001) BBTM 11A 4 cm
123+0190 – 122+0700: PROJ-1734 (15/08/2012) BBTM 11A 3 cm
122+0700 – 122+0640: PROJ-2056 (01/01/2001) BBTM 11A 4 cm
122+0640 – 122+0130: PROJ-1749 (24/10/2014) BBTM 11A 3 cm
122+0130 – 121+0310: PROJ-374 (20/12/2004) BBTM 11A 3 cm
121+0310 – 120+0770: PROJ-1909 (15/04/2016) Micromilling
120+0770 – 119+0690: PROJ-1827 (01/05/2010) BBTM 11A 3 cm
119+0690 – 117+0800: PROJ-456 (18/05/2013) BBTM 11A 3 cm
117+0800 – 116+0900: PROJ-1841 (18/03/2013) BBTM 11A 3 cm
116+0600 – 115+0800: PROJ-1839 (10/01/2012) BBTM 11A 3 cm
115+0800 – 115+0500: PROJ-436 (29/04/1995) PA 11 4 cm
115+0500 – 115+0200: PROJ-1914 (15/04/2015) Micromilling
115+0200 – 115+0110: PROJ-503 (15/09/2010) BBTM 11A 3 cm
115+0110 – 115+0050: PROJ-1910 (15/04/2015) Micromilling
115+0050 – 114+0900: PROJ-2007 (06/04/2006) BBTM 11A 3 cm
114+0900 – 113+0900: PROJ-503 (15/09/2010) BBTM 11A 3 cm
113+0900 – 113+0700: PROJ-458 (01/06/1990) AC 16 surf D 5 cm
113+0700 + 113+0200: PROJ-1724 (15/08/2014) BBTM 11A 3 cm
113+0200 – 112+0970: PROJ-2059 (01/01/2000) BBTM 11A 5 cm
112+0970 – 111+0600: PROJ-1836 (24/02/2007) BBTM 11A 3 cm
111+0600 – 111+0500: PROJ-411 (31/12/1996) PA 11 4 cm
111+0500 – 111+0000: PROJ-493 (01/06/2007) BBTM 11A 7 cm
111+0000 – 110+0000: PROJ-1888 (01/06/2006) BBTM 11A 3 cm
110+0000 – 109+0420: PROJ-37 (01/06/2007) BBTM 11A 4 cm
109+0420 – 108+0180: PROJ-44 (16/04/2007) PA 11 4 cm
108+0180 – 108+0100: PROJ-2036 (01/06/2010) BBTM 11A 4 cm
108+0100 – 107+0100: PROJ-44 (16/04/2007) PA 11 4 cm
107+0100 – 106+0740: PROJ-505 (01/06/2008) PA 11 4 cm
106+0740 – 106+0300: PROJ-2035 (01/06/2014) BBTM 11A 4 cm
106+0300 – 106+0040: PROJ-505 (01/06/2008) PA 11 4 cm

Road Denomination	**N-633**	
Itinerary	Access to the Airport of Loiu (by Aldekone)	
Marker post (KP)	**From:** 9+0310	**To:** 13+0730 (double carriageway)
Transferred stretches		
-		
General comments:		
It is the new access to the airport. In the initial part, 9+0310 – 11+0750 shares the same road with the motorway BI-631, but the network level of the road N-633 prevails.		
Main actuations:		
Skid resistance of surface layers in 2016		
2016		
Ascending carriageway 9+0310 – 9+0800: PROJ-350 (16/05/2010) PA11 4 cm 9+0800 – 10+0400: PROJ-1851 (03/04/2014) BBTM 11A 3 cm 10+0400 – 11+0800: PROJ-350 (16/05/2010) PA11 4 cm 11+0800 – 12+0700: PROJ-387 (01/08/1995) PA11 3 cm 12+0700 – 13+0740: PROJ-1549 (07/07/2015) BBTM 11A 3 cm Descending 13+0720 – 12+0700: PROJ-1549 (07/07/2015) BBTM 11A 3 cm 12+0700 – 11+0950: PROJ-387 (01/08/1995) PA11 3 cm 11+0950 – 10+0350: PROJ-350 (16/05/2010) PA11 4 cm 10+0350 – 9+0740: PROJ-385 (01/06/2003) BBTM 11A 4 cm 9+0740 – 9+0310: PROJ-349 (01/03/2010) BBTM 11B 4 cm		

Road Denomination	**N-634**	
Itinerary	Donostia/San Sebastian – Santander and A Coruña	
Marker post (KP)	**From:** 65+0630	**To:** 97+0690 (unique carriageway)
	From: 97+0690	**To:** 108+0300 (double carriageway)
	From: 108+0810	**To:** 110+0040 (unique carriageway)
	From: 116+0990	**To:** 118+0020 (double carriageway)
	From: 118+0020	**To:** 119+0960 (unique carriageway)
	From: 119+0060	**To:** 120+0770 (double carriageway)
	From: 120+0770	**To:** 122+0370 (unique carriageway)
	From: 123+0870	**To:** 136+0130 (double carriageway)
Transferred stretches		
108+0810 – 110+0040 Bilbao (Bolueta) 110+0040 – 116+0990: Bilbao(Atxuri-Zorroza) 122+0370 – 123+0870: Trapagaran		
General comments:		
It is the main two-lane road of the territory of Biscay. It has existed from long ago and many projects have been conducted on it. The RGB has mainly introduced by-passes of main villages and city to make the traffic more fluid.		
Main actuations:		
Skid resistance of surface layers in 2016		
2016		
Unique carriageway 65+0630 – 66+0060: PROJ-1965 (01/06/2012) BBTM 11B 3 cm 66+0060 – 67+0400: PROJ-1975 (01/01/1990) AC 16 surf S 5 cm 67+0400 – 70+0080: PROJ-861 (30/05/2009) LB2 70+0080 – 75+0030: PROJ-862 (16/09/2013) LB2 75+0030 – 77+0370: PROJ-515 (31/10/2013) BBTM 11A 3 cm 77+0370 – 77+0600: PROJ-1970 (01/06/2012) BBTM 11A 3 cm 77+0610 – 77+0960: PROJ-864 (01/07/2009) LB2 77+0960 – 78+0200: PROJ-1970 (01/06/2012) BBTM 11A 3 cm 78+0200 – 78+0650: PROJ-29 (01/06/2012) AC 16 surf S 4 cm 78+0650 – 78+0850: PROJ-864 (01/07/2009) LB2 78+0850 – 78+0870: PROJ-29 (01/06/2012) AC 16 surf S 4 cm 78+0870 – 78+0950: PROJ-1743 (01/06/2012) AC 16 surf S 4 cm 79+0020 – 79+0220: PROJ-2101 (08/06/2017) AC 16 surf S 3 cm 80+0700 – 81+0080: PROJ-1743A (01/06/2011) BBTM 11A 3 cm 81+0400 – 82+0300: PROJ-1743A (01/06/2011) BBTM 11A 3 cm 83+0000 – 83+0080: PROJ-1743B (01/06/2006) BBTM 11A 3 cm 83+0080 – 83+0170: PROJ-1743C (01/06/2011) BBTM 11A 3 cm 83+0170 – 84+0400: PROJ-1743B (01/06/2006) BBTM 11A 3 cm 87+0050 – 87+0550: PROJ-867 (01/06/2012) LB2 87+0700 – 89+0800: PROJ-867 (01/06/2012) LB2 89+0800 – 93+0780: PROJ-2045 (01/06/2004) LB2 93+0780 – 93+0900: PROJ-507 (01/08/1995) AC 16 surf S 5 cm 93+0930 – 95+0670: PROJ-511 (30/06/2013) BBTM 11A 3 cm 95+0670 – 95+0820: PROJ-2044 (01/06/2004) AC 16 surf S 5 cm 95+0820 – 96+0130: PROJ-2043 (01/06/2013) AC 16 surf S 5 cm 107+0890 – 108+0030: PROJ-428 (17/07/2000) BBTM 11A 3 cm 109+0120 – 109+0400: PROJ-434 (01/03/2007) BBTM 11A 3 cm		

118+0030 – 118+0830: PROJ-482 (01/06/1989) AC 16 surf S 6 cm
119+0000 – 119+0950: PROJ-2047 (01/06/2005) BBTM 11A 4 cm

120+0780 – 121+0630: PROJ-1022 (01/06/2005) LB2
121+0630 – 122+0190: PROJ-1892 (30/09/2010) AC 16 surf S 5 cm
122+0190 – 122+0370: PROJ-1022 (001/06/2005) LB2
122+0370 – 124+0110: PROJ-1030 (01/01/2010) AC 16 surf S 5 cm
124+0110 – 124+0700: PROJ-1034 (01/06/2005) LB2
124+0700 – 124+0950: PROJ-1032 (01/01/2005) AC 16 surf S 5 cm
124+0950 – 125+0900: PROJ-1034 (01/06/2005) LB2
125+0900 – 125+0980: PROJ-1032 (01/01/2005) AC 16 surf S 5 cm
126+0130 – 126+0490: PROJ-416 (28/12/2005) BBTM 11A 3 cm
126+0490 – 126+0660: PROJ-1606 (01/06/2010) AC 16 surf S 5 cm
126+0660 – 126+0800: PROJ-483 (01/06/1990) AC 16 surf S 5 cm
126+0800 – 127+0160: PROJ-1039 (01/06/2005) LB2
127+0160 – 127+0320: PROJ-1040 (01/01/2008) AC 16 surf S 5 cm
127+0320 – 127+0410: PROJ-1039 (01/06/2005) LB2
127+0410 – 127+0560: PROJ-1042 (01/01/2005) AC 16 surf S 5 cm
127+0560 – 127+0850: PROJ-483 (01/06/1990) AC 16 surf S 5 cm
127+0850 – 128+0420: PROJ-12 (18/06/2008) AC 16 surf S 6 cm
128+0420 – 128+0520: PROJ-479 (01/06/1990) AC 16 surf S 6 cm
128+0420 – 129+0100: PROJ-1045 (01/06/2005) LB2
129+0100 – 129+0330: PROJ-479 (01/06/1990) AC 16 surf S 5 cm
129+0330 – 129+0420: PROJ-1608 (01/01/2006) AC 16 surf S 5 cm
129+0420 + 130+0060: PROJ-1048 (01/06/2005) LB2
130+0060 – 130+0560: PROJ-475 (01/06/2010) AC 16 surf S 5 cm
130+0560 – 130+0900: PROJ-475 (01/06/1993) AC 16 surf S 6 cm
130+0900 – 131+0100: PROJ-1044 (01/06/2005) LB2
131+0100 – 131+0380: PROJ-1610 (01/06/2008) AC 16 surf S 5 cm
131+0380 – 131+0630: PROJ-2068 (01/01/1980) AC 16 surf S 5 cm
131+0630 – 132+0340: PROJ-1813 (01/06/2015)AC 16 surf S 5 cm
132+0340 – 132+0550: PROJ-2068 (01/01/1980) AC 16 surf S 5 cm
132+0550 – 134+0010: PROJ-1611 (01/06/2006) AC 16 surf S 5 cm
134+0010 – 135+0730: PROJ-2068 (01/01/1980) AC 16 surf S 5 cm
135+0730 – 135+0830: PROJ-399 (28/12/1992) AC 16 surf S 4 cm
135+0830 + 136+0110: PROJ-2068 (01/01/1980) AC 16 surf S 5 cm

Ascending carriageway
97+0690 – 97+0740: PROJ-499 (31/08/1994) PA 11 4 cm
97+0470 – 97+0800: PROJ-2029 (01/06/2009) AC 16 surf S 5 cm
97+0840 – 99+0150: PROJ-490 (01/06/2006) BBTM 11A 4 cm
99+0150 – 100+0430: PROJ-498 (01/06/2001) BBTM 11A 3 cm
100+0430 – 100+0900: PROJ-2042 (01/06/2014) AC 16 surf S 5 cm
100+0900 – 101-1240: PROJ-2032 (01/06/2000) BBTM 11A 4 cm
102+0000 – 103+0090: PROJ-498 (01/06/2001) BBTM 11A 3 cm
103+0090 – 103+0240: PROJ-2039 (01/06/2009) BBTM 11A 4 cm
103+0240 – 103+0770: PROJ-498 (01/06/2001) BBTM 11A 3 cm
104+0540 – 104+0750: PROJ-2041 (01/06/2005) AC 16 surf S 5 cm
105+0500 – 105+0670: PROJ-496 (01/06/2013) BBTM 11A 3 cm
106+0250 – 106+0840: PROJ-2038 (01/06/2014) AC 16 surf S 5 cm
106+0840 – 107+0150: PROJ-1810 (20/11/2015) BBTM 11A 3 cm
107+0370 – 107+0870: PROJ-428 (17/07/2000) BBTM 11A 4 cm
119+0980 – 120+0760: PROJ-2048 (01/06/2005) BBTM 11A 4 cm

Descending carriageway
119+1010 – 119+0970: PROJ-2047 (01/06/2005) BBTM 11A 4 cm

107+0800 – 107+0370: PROJ-428 (17/07/2000) BBTM 11A 4 cm
107+0180 – 106+0840: PROJ-1810 (20/11/2015) BBTM 11A 3 cm
106+0840 – 106+0370: PROJ-2038 (01/06/2014) AC 16 surf S 5 cm
105+0680 – 105+0490: PROJ-496 (01/06/2013) BBTM 11A 3 cm
103+0760 – 103+0260: PROJ-498 (01/06/2001) BBTM 11A 3 cm
103+0260 – 103+0130: PROJ-2039 (01/06/2009) BBTM 11A 4 cm
103+0130 – 102+0000: PROJ-498 (01/06/2001) BBTM 11A 3 cm
102+0000 – 101+0980: PROJ-2032 (01/06/2000) BBTM 11A 4 cm
100+0460 – 99+0150: PROJ-498 (01/06/2001) BBTM 11A 3 cm
99+0150 – 97+0800: PROJ-490 (01/06/2006) BBTM 11A 4 cm
97+0800 – 97+0760: PROJ-2029 (01/06/2009) AC 16 surf S 5 cm
97+0760 – 97+0690: PROJ-499 (31/08/1994) PA 11 4 cm

Road Denomination	**N-637**	
Itinerary	Cruces – Erletxes (by Rontegi bridge)	
Marker post (KP)	**From:** 129+0510	**To:** 132+0200 (double carriageway)
Transferred stretches		
-		
General comments:		
It is one of the main corridors around Bilbao. It is the north itinerary in the metropolitan ring around Bilbao. It is double carriageway motorway constructed entirely by the RGB		
Main actuations:		
IRI:		
Skid resistance. SFC:		
Skid resistance of surface layers in 2016		
2016		
Ascending carriageway 8+0000 – 9+0760: PROJ-374 (20/12/2004) BBTM 11A 3 cm 9+0760 – 11+0130: PROJ-383 (01/06/1997) BBTM 11A 3 cm 11+0130 – 11+0450: PROJ-2061 (01/10/2011) BBT 11A 4 cm 11+0450 – 12+0200: PROJ-1994 (01/06/2002) BBT 11A 4 cm 12+0200 – 13+0900: PROJ-390 (26/12/2005) PA11 4 cm 13+0900 – 17+0970: PROJ-389 (28/07/2009) PA11 4 cm 17+0970 – 18+0100: PROJ-391a (30/05/2014) BBT 11A 3 cm 18+0100 – 18+0260: PROJ-391b (30/05/2014) BBT 11A 4 cm 18+0260 – 18+0750: PROJ-391a (30/05/2014) BBT 11A 3 cm 18+0750 – 18+0880: PROJ-391b (30/05/2014) BBT 11A 4 cm 18+0880 – 25+0990: PROJ-391a (30/05/2014) BBT 11A 3 cm 26+0000 – 26+0100: PROJ-391b (30/05/2014) BBT 11A 4 cm 26+0100 – 26+0500: PROJ-391a (30/05/2014) BBT 11A 3 cm 26+0500 – 27+0450: PROJ-2051 (01/06/2014) BBT 11A 4 cm 27+0450 – 27+0740: PROJ-1978 (28/05/2015) BBT 11A 4 cm 27+0740 – 28+0060: PROJ-1938 (08/08/2015) BBT 11A 4 cm 28+0060 – 28+0180: PROJ-2050 (31/08/2016) BBTM 11A 3 cm Descending carriageway 28+0140 – 28+0060: PROJ-2054 (01/06/2007) PA11 4 cm 28+0060 – 26+0460: PRPJ-1723 (10/07/2015) BBT 11A 3 cm 26+0460 – 18+0880: PROJ-391a (30/05/2014) BBT 11A 3 cm 18+0880 – 18+0750: PROJ-391b (30/05/2014) BBT 11A 4 cm 18+0750 – 18+0260: PROJ-391a (30/05/2014) BBT 11A 3 cm 18+0260 – 18+0100: PROJ-391b (30/05/2014) BBT 11A 4 cm 18+0100 – 17+0080: PROJ-1754 (27/07/2015) BBT 11A 3 cm 17+0080 – 15+0620: PROJ-396 (01/06/2010) PA11 4 cm 15+0620 – 15+0450: PROJ-1720 (28/06/2013) BBT 11A 3 cm 15+0450 – 12+0200: PROJ-390 (26/12/2005) PA11 4 cm 12+0200 – 11+0420: PROJ-384 (09/01/2006) PA11 4 cm 11+0420 – 10+0800: PROJ-1722 (05/06/2015) BBT 11A 3 cm 10+0800 – 10+0330: PROJ-1751 (31/07/2014) BBT 11A 3 cm 10+0330 – 9+0670: PROJ-383 (01/06/1997) BBTM 11A 3 cm 9+0670 – 8+0430: PROJ-374 (20/12/2004) BBTM 11A 3 cm 8+0430 – 8+0100: PROJ-1721 (05/01/2012) BBTM 11A 3 cm 8+0100 – 7+0940: PROJ-374 (20/12/2004) BBTM 11A 3 cm		

Road Denomination	**N-644**	
Itinerary	Access to the Port	
Marker post (KP)	**From:** 129+0510	**To:** 132+0200 (double carriageway)
Transferred stretches		
-		
General comments:		
This motorway is the access from the motorway A-8 to the Port of Bilbao, by means of a direct connection through the tunnel of Serantes. It was conducted in 1991 (PROJ-423). The main rehabilitation was carried out in 1998 (PROJ-425) with an additional 3 cm of BBTM 11 A in the majority of both carriageways. The initial part of the road was modified due to the project of the By-pass of Bilba (VSM) (PROJ-453), milling 3 cm and extending new 3cm of BBTM 11A.		
Main actuations:		
Skid resistance of surface layers in 2016		
2016		
Ascending carriageway 129+0510 – 129+0680: PROJ-453 (23/07/2010) BBTM 11A 3 cm 129+0680 – 132+0220: PROJ-425 (01/09/1998) BBTM 11A 3 cm Descending carriageway 132+0220 – 129+0810: PROJ-425 (01/09/1998) BBTM 11A 3 cm 129+0810 – 129+0320: PROJ-453 (23/07/2010) BBTM 11A 3 cm		

Road Denomination	**BI-604**	
Itinerary	Bilbao – Asua (through Enekuri)	
Marker post (KP)	**From:** 2+0740	**To:** 6+0960 (double carriageway)
	From: 6+0960	**To:** 7+0440
Transferred stretches		
-		

General comments:

It is one of the itineraries to go into Bilbao, the capital of the province. There was a road that the Regional Government of Biscay has converted into a double carriageway highway after successive works to increase the capacity.

Main actuations:

Skid resistance of surface layers in 2016

2016

Ascending carriageway:
2+0740 – 4+0050: PROJ-406 (07/07/2003) PA 11 4cm
4+0050 – 4+0780: PROJ-407 (01/06/2009) PA 11 4 cm
4+0780 – 5+0020: PROJ-1787 (12/08/2015) BBTM 11A 4 cm
5+0020 – 5+0610: PROJ-1553 (01/04/2011) PA 11 4 cm
5+0610 – 5+0650: PROJ-2033 (10/09/2013) PA 11 4 cm
5+0650 – 5+0690: PROJ-405 (27/01/1998) PA 11 5 cm
5+0690 – 5+0760: PROJ-2033 (10/09/2013) PA 11 4 cm
5+0760 – 5+0790: PROJ-405 (27/01/1998) PA 11 5 cm
5+0790 – 5+0840: PROJ-2033 (10/09/2013) PA 11 4 cm
5+0850 – 5+0930: PROJ-1930 (12/08/2015) BBTM 11 A 4 cm
5+0930 – 5+1000: PROJ-405 (27/01/1998) PA 11 5 cm
5+1000 – 6+0230: PROJ-2019 (19/12/2007) PA 11 4 cm
6+0230 – 6+0450: PROJ-405 (27/01/1998) PA 11 5 cm
6+0450 – 6+0800: PROJ-421 (01/06/1987) HMA Ophitic 5 cm
6+0800 – 6+0960: PROJ-420 (26/02/1995) PA 11 6 cm

Descending carriageway
6+0960 – 6+0800: PROJ-420 (26/02/1995) PA 11 6 cm
6+0800 – 6+0600: PROJ-1912 (01/04/2016) Micromilling (from PA 11 6 cm)
6+0600 – 6 +0530: PROJ-2017 (06/02/2008) PA 11 4 cm
6+0530 – 5+0760: PROJ-1913 (07/04/2016) Micromilling (from PA 11 4 cm)
5+0760 – 5+0510: PROJ-2016 (07/02/2008) PA 11 4 cm
5+0510 – 5+0210: PROJ-405 (27/01/1998) PA 11 5 cm
5+0210 – 5+0110: PROJ-2034 (07/09/2013) PA 11 4 cm
5+0110 – 5+0050: PROJ-405 (27/01/1998) PA 11 5 cm
5+0050 – 4+0950: PROJ-2034 (07/09/2013) PA 11 4 cm
4+0950 – 4+0780: PROJ-405 (27/01/1998) PA 11 5 cm
4+0780 – 2+0740: PROJ-406 (07/07/2003) PA 11 4cm

Road Denomination	**BI-628**	
Itinerary	Axle of Ballonti	
Marker post (KP)	**From:** 9+0790	**To:** 10+0560 (double carriageway)
	From: 10+0560	**To:** 11+0380 (single carriageway)
	From: 11+0380	**To:** 14+0200 (double carriageway)
	From: 15+0830	**To:** 16+0410 (unique carriageway)
	From: 16+0410	**To:** 17+0040 (double carriageway)
	From: 17+0040	**To:** 17+0460 (single carriageway)
Transferred stretches		
14+0200 – 15+0830: The stretch is still not constructed		

General comments:

A short stretch (10+0670 – 11+0450) has existed from 1986. From 2000, the RGB started constructing new stretches to provide a parallel solution to the motorway A-8, which had capacity problems in that years. It was aimed to provide an alternative solution for the moments when a road accident happened and the motorway was stopped. There is still a stretch (14+0200 – 15+0830) which has not been still constructed.

Main actuations:

Skid resistance of surface layers in 2016

2016

Ascending carriageway
9+0790 – 10+0680: PROJ-418 (16/04/2007) BBTM 11A 3 cm
11+0450 – 11+0920: PROJ-417 (24/06/2007) BBTM 11A 3 cm
11+0920 – 12+0090: PROJ-1920 (31/05/2014) BBTM 11A 3 cm
12+0090 – 12+0700: PROJ-417 (24/06/2007) BBTM 11A 3 cm
12+0700 – 12+0800: PROJ-1920 (31/05/2014) BBTM 11A 3 cm
12+0800 – 14+0160: PROJ-417 (24/06/2007) BBTM 11A 3 cm
16+0410 – 16+0480: PROJ-415 (07/11/2005) BBTM 11A 3 cm
16+0480 – 17+0010: PROJ-416 (28/12/2005) BBTM 11A 3 cm

Descending carriageway
17+0010 – 16+0450: PROJ-416 (28/12/2005) BBTM 11A 3 cm
16+0450 – 16+0410: PROJ-415 (07/11/2005) BBTM 11A 3 cm
14+0160 – 12+0800: PROJ-417 (24/06/2007) BBTM 11A 3 cm
12+0800 – 12+0700: PROJ-1920 (31/05/2014) BBTM 11A 3 cm
14+0700 – 12+0110: PROJ-417 (24/06/2007) BBTM 11A 3 cm
12+0110 – 12+0020: PROJ-1920 (31/05/2014) BBTM 11A 3 cm
12+0020 – 11+0450: PROJ-417 (24/06/2007) BBTM 11A 3 cm
10+0520 – 9+0790: PROJ-418 (16/04/2007) BBTM 11A 3 cm

Unique carriageway
10+0680 – 10+0780: PROJ-418 (16/04/2007) BBTM 11A 3 cm
10+0780 – 10+1200: PROJ-419 (25/06/1984) HMA Ophitic 5 cm
10+1200 – 10+1340: PROJ-2090 (15/02/2008) AC 16 surf S 6 cm
10+1340 – 10+1440: PROJ-419 (25/06/1984) HMA Ophitic 5 cm
11+0000 – 11+0200: PROJ-2090 (15/02/2008) AC 16 surf S 6 cm
11+0200 – 11+0450: PROJ-419 (25/06/1984) HMA Ophitic 5 cm
15+0950 – 16+0400: PROJ-415 (07/11/2005) BBTM 11A 3 cm
17+0020 – 17+0470: PROJ-416 (28/12/2005) BBTM 11A 3 cm

Road Denomination	**BI-631**	
Itinerary	A-8 – Bermeo (Bilbao - Mungia)	
Marker post (KP)	**From:** 0+0000	**To:** 1+0440 (double carriageway)
	From: 3+0240	**To:** 9+0370 (double carriageway)
	From: 11+0820	**To:** 19+0910 (double carriageway)

Transferred stretches

1+0350 – 3+0240 : Shared with N-634 (Miraflores – Bolueta)
9+0350 – 11+0820: shared with N-633 (Junction of Derio – Deviation to the airport)

General comments:

This part of the road, which is in the Area 4 as it provides high capacity road to the metropolitan area of Bilbao, was almost completely constructed by the Regional Government of Biscay. The only part that exists previously was the stretch La Herradura – Galbarritu, but it was rehabilitated entirely in 1984 (PROJ-441 and PROJ-386), being incorporated as new outline in the database.

Moreover, this part is the only one that has not a minimum of two lanes per direction.

Main actuations:

Skid resistance of surface layers in 2016

2016

Ascending carriageway

0+0000 – 0+0950: PROJ-1915 (18/03/2014) BBTM 11A 4 cm
0+0950 – 1+0260: PROJ-434 (01/03/2007) PA 11 4 cm
1+0260 – 1+0350: PROJ-434 (01/03/2007) BBTM 11A 4 cm
3+0240 – 5+0250: PROJ-428 (17/07/2000) BBTM 11A 4 cm
5+0250 – 5+0850: PROJ-1919 (31/03/2014) BBTM 11A 3 cm
5+0850 – 6+0740: PROJ-1556 (02/08/2012) BBTM 11A 3 cm
6+0740 – 6+0950: PROJ-429 (01/06/1997) PA 11 5 cm
7+0000 – 7+0340: PROJ-1558 (01/05/2014) BBTM 11A 3 cm
7+0340 – 7+0850: PROJ-1848 (18/08/2016) BBTM 11A 4 cm
7+0850 – 8+0430: PROJ-1710 (30/06/2013) BBTM 11A 3 cm
8+0430 – 8+0750: PROJ-350 (16/05/2010) BBTM 11A 3 cm
8+0750 – 9+0350: PROJ-385 (01/06/2003) BBTM 11A 4 cm
11+0820 – 11+0890: PROJ-350 (16/05/2010) BBTM 11A 4 cm
11+0890 – 19+0840: PROJ-432 (01/06/2009) BBTM 11A 4 cm

Descending carriageway

19+0840 – 12+0070: PROJ-432 (01/06/2009) BBTM 11A 4 cm
9+0350 – 8+0440: PROJ-385 (01/06/2003) BBTM 11A 4 cm
8+0430 – 7+0850: PROJ-1710 (30/06/2013) BBTM 11A 3 cm
7+0850 – 7+0340: PROJ-1848 (18/08/2016) BBTM 11A 3 cm
7+0340 – 7+0220: PROJ-429 (01/06/1997) PA-11 5 cm
7+0220 – 7+0180: PROJ-1991 (08/04/2016) Micro-milling PA-11 5 cm
7+0180 – 6+0750: PROJ-1991 (08/04/2016) Micro-milling BBTM 11A 3 cm
6+0750 – 5+0850: PROJ-1556 (02/08/2012) BBTM 11A 3 cm
5+0850 – 5+0250: PROJ-1919 (31/03/2014) BBTM 11A 3 cm
5+0250 – 3+0240: PROJ-428 (17/07/2000) BBTM 11A 4 cm
1+0350 – 1+0260: PROJ-434 (01/03/2007) BBTM 11A 4 cm
1+0260 – 0+0950: PROJ-434 (01/03/2007) PA 11 4 cm
0+0950 – 0+0300: PROJ-1916 (23/04/2014) BBTM 11A 4 cm
0+0300 – 0+0200: PROJ-436 (24/04/1995) PA-11 4 cm
0+0200 – 0+0000: PROJ-503)15/09/2010) BBTM 11A 4 cm

<table>
<tr><td>Road Denomination</td><td colspan="2">BI-636</td></tr>
<tr><td>Itinerary</td><td colspan="2">Cadagua Itinerary</td></tr>
<tr><td>Marker post (KP)</td><td>From: 4+0170</td><td>To: 34+0450</td></tr>
<tr><td colspan="3">Transferred stretches</td></tr>
<tr><td colspan="3">-</td></tr>
<tr><td colspan="3">General comments:</td></tr>
<tr><td colspan="3">The original road went along the valley of the Cadagua, following the outline of the Cadagua river. In order to improve the accessibility of the valley, a new road with bigger radii was constructed. It has two parts: the part nearer to Bilbao, until Aranguren (22+0260), is a motorway of double carriageway of two lanes each and from there, it is two-lane road but with an improved outline.
The complete new outline was performed by the Regional Government of Biscay, so the complete pavement structure is known in the entire road. The construction of the road started in the furthest point, in Balmaseda, in 1992, and from there it went towards Bilbao, until the connection with the A-8 motorway.</td></tr>
<tr><td colspan="3">Main actuations:</td></tr>
<tr><td colspan="3"></td></tr>
<tr><td colspan="3">IRI:</td></tr>
<tr><td colspan="3">.</td></tr>
<tr><td colspan="3">Skid resistance. SFC:</td></tr>
<tr><td colspan="3"></td></tr>
<tr><td colspan="3">Skid resistance of surface layers in 2016</td></tr>
<tr><td colspan="3">2016</td></tr>
<tr><td colspan="3">Ascending carriageway
4+0170 – 6+0010: PROJ-1586 (30/10/2013) BBTM 11A 3 cm
6+0010 – 9+0250: PROJ-321A (01/04/2007) BBTM 11A 4 cm
9+0250 – 10+0300: PROJ-321B (01/04/2007) BBTM 11A 3 cm
10+0300 – 14+0600: PROJ-320 (30/05/2008) BBTM 11A 3 cm
14+0600 – 15+0300: PROJ-1925 (01/06/2008) BBTM 11A 4 cm
15+0300 – 15+0420: PROJ-319 (24/07/2001) PA 11 4 cm
15+0420 – 15+0540: PROJ-1578 (01/01/2011) BBTM 11A 4 cm
15+0540 – 16+0530: PROJ-1579 (15/09/2013) BBTM 11B 4 cm
16+0530 – 22+0080: PROJ-318 (01/06/2006) PA 11 4 cm

Descending carriageway
22+0080 – 21+0400: PROJ-317 (29/07/1998) PA 11 4 cm
21+0400 – 21+0290: PROJ-1580 (15/04/2013) BBTM 11B 4 cm
21+0290 – 21+0220: PROJ-1193 (15/06/2015) BBTM 11B 4 cm
21+0220 – 20+0990: PROJ-1580 (15/04/2013) BBTM 11B 4 cm
20+0960 – 20+0270: PROJ-317 (29/07/1998) PA 11 4 cm
20+0270 – 16+0650: PROJ-1581 (01/01/2012) BBTM 11B 4 cm
16+0650 – 15+0560: PROJ-1582 (15/09/2013) BBTM 11B 4 cm
15+0560 – 15+0280: PROJ-319 (24/07/2001) PA 11 4 cm
15+0280 – 14+0620: PROJ-1583 (01/01/2008) BBTM 11A 4 cm
14+0620 – 14+0390: PROJ-1584 (01/01/2011) BBTM 11A 4 cm
14+0390 – 14+0200: PROJ-320 (30/05/2008) BBTM 11A 3 cm
14+0200 – 14+0100: PROJ-1758 (30/10/2015) BBTM 11A 3 cm
14+0100 – 10+0300: PROJ-320 (30/05/2008) BBTM 11A 3 cm
10+0300 – 8+0670: PROJ-321A (01/04/2007) BBTM 11A 4 cm
8+0670 – 8+0250: PROJ-321B (01/04/2007) BBTM 11A 3 cm
8+0250 – 7+0770: PROJ-1585 (15/05/2014) BBTM 11B 3 cm
7+0770 – 6+0700: PROJ-321B (01/04/2007) BBTM 11A 3 cm
6+0700 – 5+0000: PROJ-1586 (30/10/2013) BBTM 11A 3 cm
5+0000 – 4+0170: PROJ-413 (01/08/1997) PA 11 5 cm</td></tr>
</table>

Unique carriageway
22+0090 – 22+0280: PROJ-1587 (01/01/2007) PA 11 4 cm
22+0280 – 22+0300: PROJ-1001 (01/01/2005) LB2
22+0300 – 24+0940: PROJ-1002 (15/07/2013) LB2
24+0940 – 26+0160: PROJ-1588 (15/10/2014) BBTM 11A 3 cm
26+0160 – 27+0200: PROJ-398 (20/01/1995) HMA Ophitic 6 cm
27+0200 – 27+0910: PROJ-1005 (01/01/2005) LB2
27+0910 – 28+0400: PROJ-1598 (01/01/2009) BBTM 11A 3 cm
28+0400 – 28+1030: PROJ-1007 (01/01/2005) LB2
29+0000 – 31+0870: PROJ-791 (18/08/2015) BBTM 11B 3 cm
31+0870 – 31+1060: PROJ-1595 (15/04/2013) BBTM 11A 4 cm
31+1060 – 32+0220: PROJ-1596 (01/01/2012) BBTM 11A 4 cm
32+0220 – 33+0350: PROJ-791 (18/08/2015) BBTM 11B 3 cm
33+0350 – 33+0650: PROJ-2002 (01/06/2013) BBTM 11B 3 cm
33+0650 – 34+0080: PROJ-791 (18/08/2015) BBTM 11B 3 cm
34+0080 – 34+0420: PROJ-1809 (24/04/2015) AC 16 surf S 5 cm

Road Denomination	**BI-637**	
Itinerary	Kukularra - Sopelana	
Marker post (KP)	**From:** 7+0970	**To:** 18+0370 (Ascending carriageway)
	From: 18+0470	**To:** 7+0610 (Descending carriageway)
	From: 18+0370	**To:** 18+0680 (Unique carriageway)
Transferred stretches		
-		
General comments:		
It runs on the right margin of the estuary of Nervion, proving a high capacity road to this area of the Metropolitan Area of Bilbao..		
Main actuations:		
Skid resistance of surface layers in 2016		
2016		
Ascending carriageway 7+0970 – 8+0270: PROJ-2060 (01/01/2000) BBTM 11A 4 cm 8+0270 – 9+0250: PROJ-1713 (19/09/2012) BBTM 11A 3 cm 9+0250 – 9+0400: PROJ-2084 (15/02/2008) PA 11 4 cm 9+0400 – 9+0550: PROJ-449 (01/10/1996) BBTM 11A 3 cm 9+0550 – 9+0650: PROJ-2084 (15/02/2008) PA 11 4 cm 9+0650 – 9+0900: PROJ-449 (01/10/1996) BBTM 11A 3 cm 9+0900 – 11+0700: PROJ-789A (01/06/2002) BBTM 11A 3 cm 11+0700 – 13+0050: PROJ-789B (01/06/2002) BBTM 11A 4 cm 13+0050 – 13+0700: PROJ-1711 (15/08/2014) BBTM 11A 3 cm 13+0700 – 13+0840: PROJ-437A (01/06/2007) BBTM 11A 4 cm 13+0840 – 14+0750: PROJ-437B (01/06/2007) BBTM 11A 3 cm 14+0750 – 15+0510: PROJ-1712 (03/06/2015) BBTM 11A 3 cm 14+0510 – 15+0670: PROJ-437B (01/06/2007) BBTM 11A 3 cm 15+0670 – 15+0900: PROJ-433 (16/06/1999) PA 11 6 cm 15+0900 – 16+0500: PROJ-437B (01/06/2007) BBTM 11A 3 cm 16+0500 – 18+0350: PROJ-435 (21/08/2006) PA 11 4 cm Descending carriageway 18+0350 – 17+0700: PROJ-435 (21/06/2006) PA 11 4 cm 17+0700 – 17+0470: PROJ-2085 (06/04/2011) PA 11 4 cm 17+0470 – 16+0500: PROJ-435 (21/08/2006) PA 11 4 cm 16+0500 – 15+0290: PROJ-433 (16/06/1999) PA 11 6 cm 15+0290 – 13+0750: PROJ-437A (01/06/2007) BBTM 11A 4 cm 13+0750 – 13+0610: PROJ-1718 (15/04/2011) BBTM 11A 3 cm 16+0610 – 13+0410: PROJ-1923 (13/04/2016) Micromilling, from BBTM 11A 3 cm 13+0410 – 13+0310: PROJ-1718 (15/04/2011) BBTM 11A 3 cm 13+0310 – 12+0600: PROJ-789B (01/06/2002) BBTM 11A 4 cm 12+0600 – 11+0700: PROJ-789A (01/06/2002) BBTM 11A 3 cm 11+0700 – 9+0900: PROJ-1917 (01/06/1998) BBTM 11A 3 cm 9+0900 – 9+0400: PROJ-1714 (16/04/2015) BBTM 11A 3 cm 9+0400 – 9+0160: PROJ-384 (09/01/2006) PA 11 4 cm 9+0160 – 8+0360: PROJ-1717 (29/09/2013) BBTM 11A 3 cm 8+0360 – 7+0630: PROJ-2060 (01/01/2000) PA 11 4 cm Unique carriageway 18+0370 – 18+0680: PROJ-435 (21/08/2006)		

Road Denomination	**BI-644**	
Itinerary	Kareaga – Sestao (Vega Vieja)	
Marker post (KP)	**From:** 9+0480	**To:** 11+0550 (double carriageway)
Transferred stretches		
-		
General comments:		
It is a double carriageway highway placed near big commercial areas, which connects important residential areas. The first project was carried out to connect the road N-634 in Kareaga to the A-8 motorway. Later, the last part was modified to connect with the BI-628, providing a double carriageway highway.		
Main actuations:		
New outline projects (ordered according to Kilometre Post)		
Skid resistance of surface layers in 2016		
2016		
Ascending carriageway 9+0480 – 10+0130: PROJ-2015 (01/06/2004) BBTM 11A 3 cm 10+0130 – 10+0800: PROJ-1548 (15/05/2015) BBTM 11A 3 cm 10+0800 – 11+0550: PROJ-417 (24/06/2007) AC 16 surf S 4 cm Descending carriageway 11+0530 – 10+0770: PROJ-417 (24/06/2007) AC 16 surf S 4 cm 10+0770 – 10+0130: PROJ-1548 (15/05/2015) BBTM 11A 3 cm 10+0130 – 9+0480: PROJ-2015 (01/06/2004) BBTM 11A 3 cm		

Road Denomination	**BI-647**	
Itinerary	Access to the University	
Marker post (KP)	**From:** 0+0000	**To:** 2+0180 (unique carriageway)
	From: 2+0180	**To:** 4+0550 (double carriageway)
Transferred stretches		
-		
General comments:		
Main actuations:		
Skid resistance of surface layers in 2016		
2016		
Unique carriageway 0+0000 – 0+0300: PROJ-444 (01/08/1991) AC 22 surf S 5 cm 0+0300 – 1+0850: PROJ-443 (01/06/2007) AC 16 surf S 6 cm 1+0850 – 1+0900: PROJ-442 (01/06/1997) AC 16 surf D 6 cm 1+0900 – 2+0180: PROJ-2018 (01/06/2007) LB2 Ascending carriageway 2+0180 – 2+0230: PROJ-442 (01/06/1997) AC 16 surf D 6 cm 2+0230 – 2+0260: PROJ-426 (14/07/1989) AC 22 surf D 5 cm 2+0260 – 2+0650: PROJ-305 (28/09/2012) BBTM 11A 3 cm 2+0650 – 4+0420: PROJ-445 (01/06/2006) BBTM 11A 3 cm Descending carriageway 4+0420 – 2+0650: PROJ-445 (01/06/2006) BBTM 11A 3 cm 2+0650 – 2+0300: PROJ-305 (28/09/2012) BBTM 11A 3 cm 2+0300 – 2+0180: PROJ-2018 (01/06/2007) LB2		

Road Denomination	**BI-738**	
Itinerary	Junction of Erandio - Artaza	
Marker post (KP)	**From:** 8+0900	**To:** 9+2190 (ascending carriageway)
	From: 9+2190	**To:** 9+0640 (descending carriageway)
Transferred stretches		
-		
General comments:		
This road consists of the lateral carriageways of the motorway BI-637. It serves the local traffic which enters or goes out from the BI-637.		
Main actuations:		
IRI:		
Skid resistance of surface layers in 2016		
2016		
Ascending carriageway 8+0910 – 9+0260: PROJ-2091 (12/09/2013) BBTM 11A 3 cm 9+0260 – 9+0330: PROJ-384 (09/01/2006) PA11 4 cm 9+0330 – 9+0900: PROJ-449 (001/10/1996) PA11 6 cm 9+0900 – 9+2180: PROJ-426 (14/07/1989) AC 22 surf D 5 cm Descending carriageway 9+2190 – 9+1100: PROJ-426 (14/07/1989) AC 22 surf D 5 cm 9+1100 – 9+0900: PROJ-446 (30/03/1994) PA 11 6 cm 9+0900 – 9+0770: PROJ-426 (14/07/1989) AC 22 surf D 5 cm		

Annex III

Chapter 6. Pavement Management System of the Regional Government of Biscay

6.1. Introduction

In this chapter, the characteristics of the Pavement Management System implemented by the Regional Government of Biscay are described. The Regional Government of Biscay (RGB), *Diputación Foral de Bizkaia (DFB)*, in Spanish; *Bizkaiko Foru Aldundia (BFA)*, in Basque, is the road agency responsible of almost the entire interurban road network in the province of Biscay and has developed a pavement management system, called State Agenda (Agenda de Estado) to predict future condition of the road network and better allocate available funds.

First of all, a brief historical review of the road management in Spain and in Biscay is exposed. Secondly, a complete overview of the competences that the Regional Government of Biscay has at present with regard to the road network in the territory is presented. These competences are assigned by means of the various national and regional laws. Once the competences are assigned, a law came into effect, in which how the RGB develops its competences is explained. Additionally, the road classification included in that law is described.

Finally, the characteristics of the pavement management system itself are commented. The PMS employed by the RGB is compared with the ideas commented in Chapter 2, where essential information of any PMS was exposed. Therefore, it is possible to indicate if the usual requirements for the PMS are fulfilled in State Agenda.

6.2. Historical review of the road network in Biscay

Although some roads could exist before the Roman invasion of the Iberian Peninsula, it can be stated that the first complete road network was developed in the age of the Roman Empire. The document that best exposes the existing itineraries through the Iberian Peninsula is the Antonine Itinerary (Itinerarium Antonini Augusti, in Latin; literally, The Itinerary of the Emperor Antoninus). It is a famous *itinerarium*, a register of the stations and distances along various roads. It seems to be based on official documents, possibly from a survey carried out under the emperor Augustus and it describes the roads of the Roman Empire. Due to the scarcity of other records of this type, it has great historical value. Little information is known about the date and author. It is considered to be likely that the original edition was prepared at the beginning of the 3rd century, although it is traditionally ascribed to the patronage of the 2nd century emperor Antoninus Pius, the oldest extant copy was assigned to the time of Diocletian.

With regard to the Hispania, the name of the Roman provinces of the Iberian Peninsula, 34 routes of the 372 mentioned in the document are placed in Hispania. The routes described in the Antonine Itinerary are shown in Fig. 6.1. As observed, no Roman roads are mentioned in the area of the territory of Biscay at present. Nevertheless, the Antonine Itinerary only mentioned the main roads included in the Praetor’s registration, whose equivalent at present would be the main roads of a country. Consequently, local roads, which are known from other authors (Estrabon, Pliny the Elder, etc.) are not included (Pérez-Acebo, 2018).

Fig. 6.1. Main Roman roads of Hispania, recompiled in the Antonine Itinerary. Adapted from Blázquez (1892).

After the Roman Empire fall, Roman roads were abandoned. No rehabilitation or maintenance works were performed and they become the unique existing road network for 10 centuries, during Middle Age.

During the XVI century, the Austrian dynasty performed some reconstruction activities on the road network but the base was the Roman Empire roads (Pérez-Acebo, 2018). Fig. 6.2 shows the existing road network in 1546.

Under the dynasty of Bourbons (from 1714), with the capital established in Madrid by Felipe V, a modern road network was aimed to be constructed. It was carried out by means of the document entitled “*Reglamento General para la Dirección y Gobierno de los Oficios de Correo Mayor y Postas de Españas*” [Regulations for the management and government of the post works] (Ariztia, 1720), where a road network centralised in Madrid was proposed. The proposed main itineraries went from Madrid to:

1. Bayonne (France) through Irun and Pamplona
2. Barcelona and French border
3. Valence
4. Murcia and Cartagena
5. Seville and Cadiz
6. Badajoz
7. Galicia through Medina del Campo

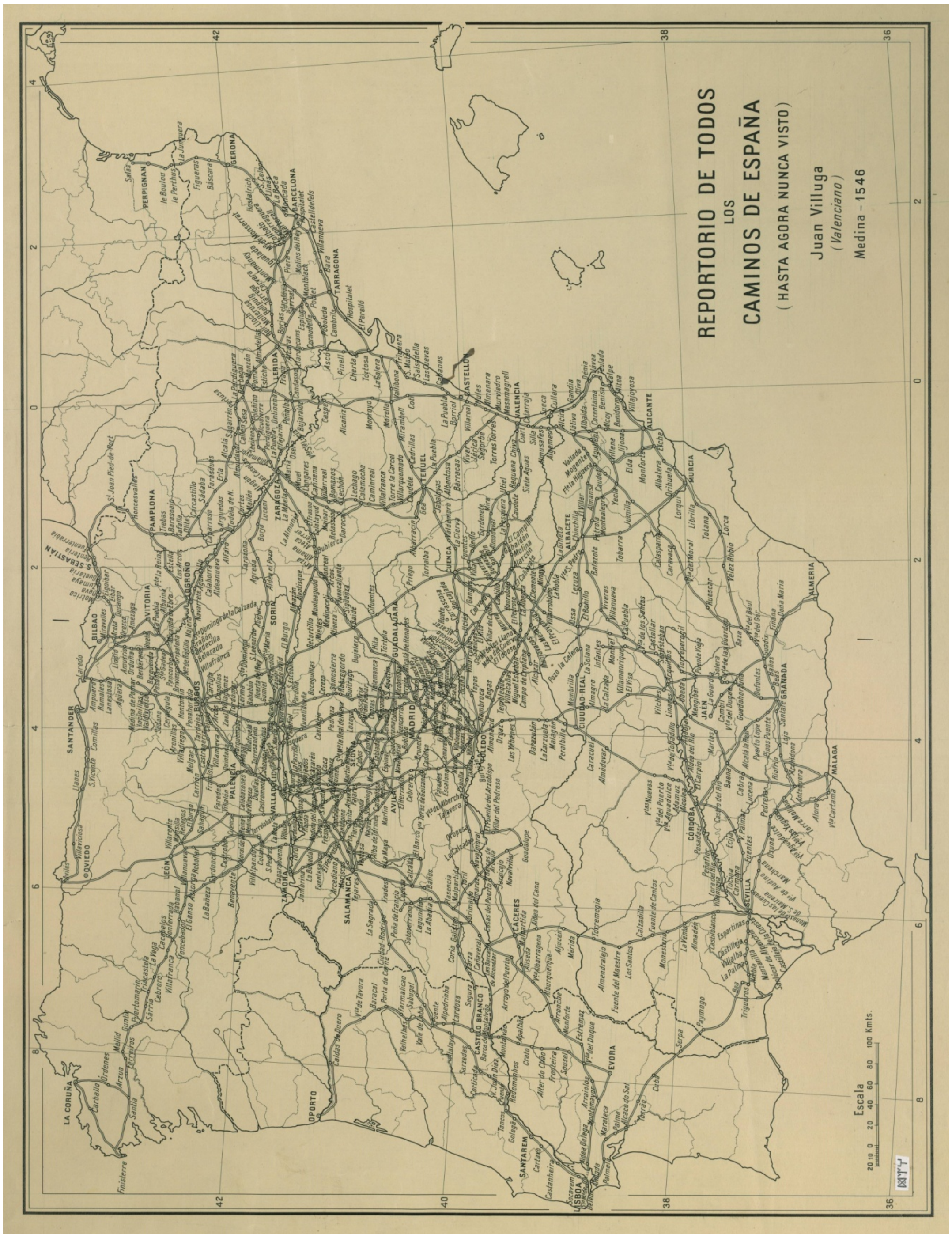


Fig. 6.2. Existing roads in 1546 in Spain (Villuga, 1546).

The network was completed with a road parallel to the Mediterranean Sea from Barcelona to Alicante, another one parallel to Portugal from Benavente to Seville and the connexion between Burgos and Medina del Campo through Valladolid.

This post network was used by the monarchy to send order from the court and to receive information. In 1761 a general plan of roads was performed from a research financed by the King Carlos III, where six radial roads were recommended from Madrid, the capital of the kingdom to La Coruña, Badajoz, Cadiz, Alicante, French border through Bayonne and Perpignan (Ward, 1779; 1787). Ward's proposal was carried out by means of a specific law in June, 10, 1761, which included funding from the Central Budget for the roads to Andalucía, Catalonia, Galicia and Valence (Carlos III, 1761). In order to finance the construction a new tax was imposed to the commerce of salt for the following 20 years, which was prolonged until 1801 (Bel, 2010). This way, the road network planned by Ward was financed except the roads in the provinces of Biscay, Gipuzkoa, Alava and Navarra, which was financed by their own Regional Treasuries (Hacienda Foral) (Wais San Martín, 1963).

Since them, different plans for development of the Spanish road network have been carried out, and the Basque provinces have maintained their autonomy with regard to the road management with varying success according to the different governments in Spain.

In the XX century, during the Franco's dictatorship (1939-1975) all the privileges of the provinces of the Basque Country were eliminated and the centralist government assumed all the competences.

With the arrival of the democracy, those privileges over roads in the territory were recovered based on the Autonomous State of the Region (BOCG del País Vasco, 1980). The current situation is further commented in section 6.3.

6.3. Competences of the Regional Government of Biscay at present

6.3.1. Process for the competences about roads to the Regional Government of Biscay

In the law about the self-governing organization of the Basque Country, the *Estatuto de Autonomía del País Vasco* [Statute of Autonomy of the Basque Country] (BOCG del País Vasco, 1980), in the article 10 it is said that all the exclusive competences of the Basque Country will be maintained and, with regard to highways and roads, apart from the competences mentioned in section 148 of the Spanish Constitution (Cortes Generales, 1978), the Regional Government of each of the 3 provinces of the Basque Country will conserve the judicial status and competences that had in the past, according to the article 3. In the article 3 it is said that each province will be able to conserve, re-establish and update its organization.

Artículo 10.

La Comunidad Autónoma del País Vasco tiene competencia exclusiva en las siguientes materias:

34. En materia de carreteras y caminos, además de las competencias contenidas en el apartado 5, número 1, del artículo 148 de la Constitución, las Diputaciones Forales de los Territorios Históricos conservarán íntegramente el régimen jurídico y competencias que ostentan o que, en su caso, hayan de recobrar a tenor del artículo 3.º de este Estatuto.

Artículo 3.

Cada uno de los Territorios Históricos que integran el País Vasco podrán, en el seno del mismo, conservar o, en su caso, restablecer y actualizar su organización e instituciones privativas de autogobierno.

Source: Statue of Autonomy of the Basque Country (BOCG del País Vasco, 1980)

The section 148 of the Spanish Constitution (Cortes Generales, 1978) indicates that the self-governing Communities may assume competences over railways and roads whose routes lie exclusively within the territory of the Self-governing Community.

Section 148

Matters devolved to self-governing Communities

1. The Self-governing Communities may assume competences over the following matters:

1. Organization of their institutions of self-government.
2. Changes in municipal boundaries within their territory and, in general, functions appertaining to the State Administration regarding local Corporations, whose transfer may be authorised by legislation on local governement.
3. Town and country planning and housing.
4. Public works of interest to the Self-governing Community, within its own territory.
5. Railways and roads whose routes lie exclusively within the territory of the Self-governing Community and transport by the above means or by cable fulfilling the same conditions.

Source: Spanish Constitution (Cortes Generales, 1978)

Moreover, the law *Ley 27/1983, de 25 de noviembre, de las relaciones entre las Instituciones Comunes de la Comunidad Autónoma y los Órganos Forales de sus Territorios Históricos* [Law 27/1983, of November 25, about the relationships between the common institutions of the Self-governing Community of the Basque Country and the Institutions of the provinces] (BOPV, 1983) recognises expressly the exclusive competence for roads of each of the Regional Governments of the Basque Provinces. However, it indicates the necessity of coordination between the 3 regional governments and other administrations.

CAPÍTULO II

De las competencias de los Territorios Históricos

Artículo 7.

a) Los Órganos Forales de los Territorios Históricos tienen competencia exclusiva, que ejercitarán de acuerdo con el régimen jurídico privativo de cada uno de ellos, en las siguientes materias:

8. Planificación, proyecto, construcción, conservación, modificación, financiación, uso y explicitación de carreteras y caminos.

Al objeto de asegurar la debida coordinación de las redes de carreteras de la Comunidad Autónoma, los Territorios Históricos pondrán en vigor para sus redes las normas técnicas y de señalización que se establezcan en el Plan General de Carreteras aprobado por las Instituciones Comunes de la Comunidad Autónoma, y en aquellas carreteras que sean prolongación de las de la Red estatal o que enlacen con las de otros Entes Públicos extracomunitarios o entre los propios Territorios Históricos, realizarán, como mínimo, aquellas previsiones, objetivos, prioridades y mejoras que se establezcan en dicho Plan General de Carreteras.

Cuando en los planes de la Comunidad Autónoma, del Estado, de otros Entes Públicos extracomunitarios o de los Territorios Históricos, se contemple el establecimiento de nuevas vías de comunicación cuyo trazado incida respectivamente en los Territorios Históricos o en los limítrofes a éstos, se procederá a coordinar dichos planes sobre la base de las facultades y atribuciones respectivas.

Todo ello sin merma de lo dispuesto en el artículo 10.34 del Estatuto de Autonomía.

Source: Ley 27/1983, de 25 de noviembre, de las relaciones entre las Instituciones Comunes de la Comunidad Autónoma y los Órganos Forales de sus Territorios Históricos (BOPV, 1983)

Consequently, in the Basque Country, the Regional Governments of each of the provinces (Alava, Biscay and Gipuzkoa) have exclusive competences over the roads and freeways that exist over their territory, although they are a part of long itinerary roads that also cross other Self-governing Communities. The only exceptions are two paying freeways which are owned by the Government of Spain (Central Government) and that are exploited by means of a concession. These motorways cannot be transferred to the regional governments until the concession ends. The paying motorways not managed by the Regional Governments of the Basque Country are:

- Motorway AP-68, which links Bilbao (the capital of Biscay) with Zaragoza through Logroño. The concession ends on November 10, 2026 (Ministerio de Fomento, 2000). It crosses the provinces of Biscay and Alava.
- Motorway AP-1, which links Burgos with Armiñon (Álava). The concession ends on November 30, 2018 (Ministerio de Fomento, 2005). It has only 5,7 km in the territory of Alava.

Table 6. and Table 6. show the kilometres of the road owned by the Government of Spain in Alava and in Biscay, respectively. Gipuzkoa does not have any road managed by the Government of Spain. Furthermore, local roads within the municipalities are owned and managed by the city councils (Pérez-Acebo, 2018).

Table 6.1. Road network owned by the Government of Spain in Alava. Source: Provincial catalogue of roads of the RCE (Red de Carreteras del Estado [Road Network of the State] (Ministerio de Fomento, 2017)

Road	Initial Milestone	Final Mileston	Starting point	Finishing point	Total Length	Road type			
						Paying freeway	Freeway	Multilane highway	Two-lane road
AP-1	77+0300	82+106(	P.L. Burgos-Álava	Junction of Armiñón	5,74	5,74			
AP-68	22+0390	77+047	P.L. Bizkaia-Álava	P.L. Álava-Burgos	55,15	55,15			
Number of roads :			**2**		**60,89**	**60,89**			

Note: P.L: provincial limit

Table 6.2. Road network owned by the Government of Spain in Biscay. Source: Provincial catalogue of roads of the RCE (Red de Carreteras del Estado [Road Network of the State] (Ministerio de Fomento, 2017)

Road	Initial Milestone	Final Milestone	Starting point	Finishing point	Total Length	Road type			
						Paying freeway	Freeway	Multilane highway	Two-lane road
AP-68	0+0000	22+039C	Bilbao	P.L.. Bizkaia-Álava	22,40	22,40			
Number of roads:			**1**		**22,40**	**22,40**			

Note: P.L: provincial limit

6.3.2. Development of the competences about roads by the Regional Government of Biscay

As it was exposed in previous section, all the interurban roads of the province of Biscay are owned and managed by the Regional Government of Biscay, except the paying motorway AP-68 and the municipal roads and streets. According to the competences indicated in the Statute of Basque Country, the previously commented Law 27/1983, of November 25 (BOPV, 1983) and the *Decreto Foral 17/85, de 5 marzo* [Law of Biscay 17/85, from March 5] which developed the previous law, the administration of the Regional

Government of Biscay has the exclusive competence of plan, project, construction, modification, funding, use and operation of the roads. Making use of this competence, it was approved in the Regional Parliament of Biscay [*Juntas Generales*] *the Norma Foral 2/1993, de 18 de febrero* [Law of Biscay 2/1993, from February 18] (BOB, 1993), which regulated the planning, project, modification, construction, conservation, funding, and use of the roads in Biscay and hence, the Regional Government had a law to managed its competences over roads.

15 years after the approval of that law, some aspects were revised with the aim of improving its content and adapt to the new law, the necessities of the territory and the citizens. Therefore, a new law was approved, *Norma Foral 2/2011, de 24 de marzo, de carreteras de Bizkaia* [Law of Biscay 2/2011, from March 24, about roads in Biscay] (BOB, 2011).

This law states the regulations about the planning, project, modification, construction, conservation, funding, use and operation of the roads of the Regional Government of Biscay. It classifies the roads according to their functionality and their spatial location in the following groups:

a) Preferential Interest network (Red network)
b) Basic network (Orange network)
c) Complementary network (Blue network)
d) Provincial network (Green network)
e) Local network (Yellow network)

The characteristics that each road must fulfil to be included in each category according to its functionality and its location are described in Table 6..

The road network managed by the RGB has a total length of more than 1.200 km, considering only main carriageway, without including ramps, connections and junctions. The length of each of the categories described in Table 6. is presented in Table 6.. However, it includes all the interurban roads existing in Biscay. Some of them are not owned by the RGB and other roads, although being owned and not maintained by the RGB. These roads are roads that are under concession, like the AP-8 motorway, managed by Interbiak, a public society depending of the RGB. The National Road N-629, which links Burgos with Colindres, is owned by the RGB but it is maintained by the Ministry of Public Works of Spain as it is located in an extreme part of the territory and the stretch in Biscay is less than 4 km long. Apart from the previously mentioned paying motorway AP-68 owned by the Government of Spain and managed under a concession, there is another road, BI-711, which is owned by the Port Authority, depending on the State Ports, a society of the Ministry of Public Works of the Spanish Government. All this information is exposed in Table 6.1.

The complete road network of Biscay managed by the Regional Government of Biscay is shown in Annex 1 (Pérez-Acebo, 2018). Roads are classified by network types, those exposed previously, and the different stretches that compose each road are identified. The initial and final milepost and the length of each stretch are included. Moreover, the type of road is also shown. The road types are those established by the law *Ley 37/2015, de 29 de septiembre, de carreteras* [Law 37/2015, of 29 of September, about Roads] (BOE, 2015), which is the law of road of the Spanish Government. The road types established in the law are listed below:

- ***Autopistas*** (Motorways). It is described as a road specifically planned and constructed for automobile, with different carriageways in each sense of circulation, separated by a zone that cannot be used for circulating, nor crossing neither crossed by other roads or lines, and with not access to adjacent properties or services roads.
- ***Autovías*** (Motorways). Similar to "*autopistas*" but they can be used by any motor vehicle and they do not have access to adjacent properties but can be connected to service roads.
- ***Carreteras multicarril*** (Multilane highways). Roads with more than 2 lanes per sense of circulation, separated or delimited, that can cross or be crossed by other roads or paths. There is no direct access to adjacent properties.
- ***Carreteras convencionales*** (Conventional roads). Roads that do not fulfil the requirements of previous road types.

Table 6.3. Road network levels of the roads of Biscay, according to their function and their place. Source: Norma Foral 2/2011, de 24 de marzo, de Carreteras de Bizkaia (BOB, 2011)

Metropolitan area of Bilbao	Rest of the territory
a) Preferential interest network	
• Serves for passing long distance journeys that pass through Biscay or the ones that have the Metropolitan Areas as start or finish point. • Serves to access to the transport terminals (port, airport, intermodal and logistic zones)	• Serves for passing long distance journeys, that pass the territory or that have the start or finishing point in the Metropolitan Area or the transport terminals.
b) Basic network	
• Channel the streams between orbital and annular zones of the Metropolitan Area • Serve for complementing the high capacity itineraries, allowing the network management	• Makes easier the accessibility of non Metropolitan areas to the Preferential Interest Network • Helps organizing the territory
c) Complementary network	
• Connects the high capacity itineraries of the Basic Network and the Preferential Interest Network with the main urban streets • Contributes creating a Metropolitan Area integrating unconnected land and areas. • Gives accessibility to opportunity lands • Gives access to huge mobility generators.	• Does not exit
d) Provincial	
• Improves the accessibility between orbital areas in less urbanised areas. • Is an alternative to the Basic Network • Gives access to supra-municipal facilities • Improves the accessibility to all types of lands • The urban development will transform them in streets.	• Connect adjacent regions • Access to fishing ports. • Contributes to create a road net • Improves the accessibility to lands • Improves the interconnection between near council halls. •
e) Local Network	
• Integrated by roads of the RGB that are not included in the previous networks	• Integrated by roads of the RGB which are not included in the previous network.

Table 6.4. Length of the road network of Biscay by network type and road type in 2015, in km (1). Source: Section of Inventory of Roads. Regional Government of Biscay.

Network	Paying motorways	Free motorways	Other Motorways	Multilane roads	Road	Total
Preferential Interest (Red)	68,881	30,009	27,295	13,854	106,794	246,833
Basic (Orange)			39,437	19,470	151,570	210,477
Complementary (Blue)				4,803	24,080	28,883
Provincial (Green)				1,070	208,847	209,917
Local (Yellow)				1,217	609,685	610,902
TOTAL	**68,881**	**30,009**	**66,732**	**40,414**	**1.100,976**	**1.307,012**

(1) Only main carriageway of the road, without including ramps, connections and junctions.

Table 6.1. Summary of the ownership and management of the road network of Biscay. Source: Section of Inventory of Roads. Regional Government of Biscay

Ownership	Management	Road code	Road name	Milepost	Length (km)
Regional Government of Biscay	**Regional Government of Biscay**	***Rest of roads***	-		**1.223,649**
	Concession	*AP-8*	*Motorway of the Cantabric Sea (Paying)*	*74+0905 – 129+ 504*	*46,531*
	Concession	*BI-625*	*Orduña Bilbao (Tunnel La Salve – Ugazko)*	*395+0075 – 396+0115*	*1,009*
	Concession	*BI-626*	*Tunnel Artxanda – La Salve (Paying)*	*1+0757 – 3+0718*	*1,961*
	Concession	*BI-627*	*Tunnel Artxanda - Ugazko (Paying)*	*2+0109 – 3+0621*	*1,512*
	Other (Ministry of Public Works)	*N-629*	*Burgos - Colindres (N-629)*	*60+0810 – 64+0675*	*3,870*
				Total not managed by the RGB	**54,883**
				Total of the RGB	**1,278.532**
Not the Regional Government of Biscay	**Government of Spain (Concession)**	*AP-68*	*Paying motorway Bilbao-Zaragoza (Not RGB)*	*0+0000 – 22+0350*	*22,350*
	Port Authority	*BI-711*	*Bilbao to Getxo (Not RGB)*	*7+0200 – 13+0330*	*6,130*
				Total of not RGB	**28,480**
				Total Road Network (main road, without ramps)	**1.307,012**

6.4. “State Agenda”, the Pavement Management System of the RGB

6.4.1. Introduction

The Regional Government of Biscay had collected data about road state by means of various indicators and indices since 2000. It has listed all the information related to roads that must manage, including information like length of roads, its classification (two-lane roads, multilane-highways and freeways), geometric characteristics (number of lanes, lane width, radii of curves, etc.), traffic volumes, inventory of structures, etc. Furthermore, all the projects that were funded and managed by the RGB were collected. However all the information was dispersed in several places, physically and even when data were computerized they do not share the same file format, so information had to be checked in various files, usually not compatible.

Consequently, decisions about maintenance and rehabilitation activities were taken based on engineers' experience without following any rational method. Due to their expertise, good alternatives were selected but it was not known on how pavements evolve depending on the applied solutions. Briefly, resources, human and economic ones, were invested subjectively.

Since 2010 it was understood that a Pavement Management System was necessary to manage all the information and that a more rational method had to be applied to carry out the best possible solution, after optimisation. Consequently, invitation to tender was issued and the proposal of three engineering firms won the contract. Those companies are Dair Engineering, Euskontrol, S. A. and Ingeplan Consulting.

Since then, they have developed the Pavement Management System of the Regional Government of Biscay, called State Agenda and they continue implementing with the daily information reported from the services of the Regional Government of Biscay.

The State Agenda is the basic management tool of the road administration for taking decisions related to the activities and priorities to be adopted for the conservation activities of the road network.

The State Agenda takes into accounts the failings, deterioration and lack of adaptation of the parameters that calibrate the functional adequacy and condition of the infrastructure, as well as the required improvements, immediate and low-cost in order to ensure better services and increase road safety.

The State Agenda will carry out the following functions:

- Diagnose the results from the data collections and/or inventories, taken into account the diverse temporary conditions.
- Determine different alternatives and their efficiency and propose a solution
- Evaluate the solution, not only from the economical point of view, but also considering the function and temporal limitations of the solution
- Propose indicators that allow technicians evaluating the evolution of the affected parameters.

The main aim of the State Agenda is to achieve a faithful picture of what the road administration (RGB) would need to invest to maintain its road network according to current regulations, as well as having improved the road safety and functionality according to the emplacement.

All the information included in the inventory is uploaded to an informatics programme, which store all the data. This software is called Gestivía (Road Management) and is accessible to all the people who work in the pavement management in the RGB.

The initial appearance of the programme is shown in Fig. 6.3. It has 6 main sections which can be displayed and subsections are shown. The main areas are: General [General], Inventarios [Inventories], Agenda AE [Agenda of the state], Agenda AE Pruebas [Agenda of the state, proofs] (is not considered as a section as it is the same as the previous one, but changes can be made to observed produced modifications), Obras [Works], Informes [Reports] and Agenda 2G [2^{nd} generation of the agenda]. The subsections of each main area are listed in Table 6. to Table 6..

In next section data included in State Agenda are described according to the usual inputs that a pavement management system must have, following the points explained in Chapter 2. Hence, available data are checked taking into account the necessary information that a Pavement Management System must have.

Fig. 6.3. Appearance of Gestivía, the software of Pavement Management System of the Regional Government of Biscay.

Table 6.6. Subsections of the area General [General] in the Gestivía software and data included in each one.

1) GENERAL [GENERAL]
Datos auxiliares [Auxiliary data]
General [General]: Initial and final KP and distances of the road. Type of network **Tramos [Stretches]:** Stretches in which the road is divided. **D. Ejes [Data of axes]:** Existing axles in each road: Main road (conventional road, unique carriageway), ascending or descending carriageway and ramps (with their KP) **D. Áreas conversation [Conservation areas]:** The area which the road belong to, from Area 1, 2, 3 or 4. **D. Ámbitos [Spaces]:** Name or references to indicate the sub-stretches in which the road is divide. They are mainly popular names using existing known features (a crossroad, a petrol station, etc.) **D. Ámbitos special [Special spaces] :**It indicates how the road is classified in the Road Sector Plan **D. Tramos visualizador [Stretches of the viewer]:** Codes (denomination) of the stretches according to the ones used in the viewer of the webpage of the RGB **D. Aforos [Traffic volumes]:** Traffic and heavy traffic volumes and traffic category of each road since 2000 **Visualizador DFB [Viewer of RGB]:** All the geometric data shown in the viewer of the RGB in the webpage. In each road every 10 m. **Travesía [Urban stretches]:** Indication of the urban stretches of each road, with their code **ZCA (Tramos de concentración de accidentes) [Zone of accident concentration]:** List of existing zones of accident concentration on the road network of Biscay, indicating accident rating and type of point **Deflexiones [Deflexions]:** Some coefficients of an old research carried out by GEOCISA

Table 6.7. Subsections of the area Inventorios [Inventories] in the Gestivía software and data included in each one

2) INVENTORIOS [INVENTORIES]

Infraestructura AE [Infraestructure]

General [General]: Division in sub-stretches according to structure, embankment, etc.

Contención [Protection elements]: Concrete New Jersey, metallic defences

Firmes [Pavements]: Year of opening to public. Lane width, shoulder width.

Señalización vertical [Vertical signs]: All the vertical signs on each road, with their codes

Auscultación de firmes AE [Pavement data collection]

CRT [Transversal skid resistance, SCRIM Coefficient]: Every 20 m, the SCRIM Coefficient and the texture (MPD) is provided in each road, in the data collection years

IRI International Roughness Index: Every 100m, the IRI value is provided in each road, in the data collection years

Deflexiones Deflectógrafo [Deflexions with Deflectograph): Right and left deflexions obtained with the Deflectograph during data collections

Deflexiones Dynatest [Deflections with Dynatest (FWD)]: Deflections obtained with Dynatest

Deflexión característica [Characteristic deflection]: Calculated characteristic deflection for each stretch

Defleción de cálculo [Calculation deflection]: Deflection obtained for each sub-stretch according to Ministerio de Fomento (2003a)

Roderas [Rutting]: Obtained left and right rutting values during data collections

Índices de Fisuración [Cracking Indices]: Cracking indices expressed as the portion of road with cracking defects, in decimal and in percentages in different years

Inventario Firmes [Pavement inventory]

Firme inicial [Initial pavement]: Stretches with pavement initial files

Actuactiones [Projects and work]: Projects and works carried out. Different possibilities of classification according to variable characteristics.

Deterioros superficiales [Surface defects]: Patching, cracking and other defects observed in pavement surface, indicating the dimensions, by road. Date of reparation too

DESU [Surface defects]: An attempt to create a surface defects managing system. Not developed

Sondeos [Core drilling tests]: Places and year of core drilling test, with all the data obtained

Inventario [Inventory]: Shows the Initial Pavement file (stretches where the initial pavement is known, as introduced in the project), the rehabilitation projects, the Pavement Structure file, surface defects, and Surface Layer file, for each road

Gestión de Firmes-GEFI [Pavement management]: Classification of pavement structure according to previous attempts of PMS, not in use at present

Accidentes [Accidents]

Parte de accidente [Accident file]: All de data about the accident

Pluviometría [Rainfall level]: Rainfall data of each month in years from 2000 to 2016

Pluviometría Euskalment [Rainfall level according to Basque Meteorology Agency]: Similar to the previous one, from 2011 to 2016 and differentiated by conservation areas

Obras especiales [Special works]: Works conducted in each roads

Puntos especiales [Special points]: High or low points, crossroads, etc.

Table 6.8. Subsections of the area Agenda de Estado [Agenda of state] in the Gestivía software and data included in each one

3) AGENDA DE ESTADO [State agenda]

Agenda estado [State agenda]

Algoritmo Elementos de contención local [Algorithm of contention local elements]: Existing elements for contention and project velocity

A. Elementos de contención motorists [Algorithm of motorcycle user contention elements]: Existing elements for contention for motorcycle users

A. Firmes (Pavement algorithm]: Calculates the extension of area to be repaired according to variable criteria (SCRIM Coefficient, Cracking indices, etc.) which can be modified in Configuration.

A. Firmes vida remanente [Remaining life algorithm of the pavement]: Algorithm (proposed by GEOCISA) to predict the remaining life of the road according to deflections

A. DESU SIN CRT [Superficial defects algorithm without SCRIM Coefficient]: Severity of superficial defects without taking into account the SCRIM coefficient

A. Cerramiento [Fences]: List of existing fences along the roads

A. Drenajes transversales [Transverse drainage]: Existing transverse drainage

A. Drenajes longitudinales [Longitudinal Drainage]: Existing longitudinal drainage

A. Drenajes superficiales [Surface drainage]: Existing surface drainage

A. Drenajes profundos [Deep drainage]: Existing deep drainage

A. Calado lámina agua [Water height in culverts]: Water height in culverts

A. Señalización caducidad [Date of expiry of vertical sign]: Date of expiry of vertical sign

A. Señalización deceleración [Decelerating signs]: Existing vertical signs for deceleration

A. Señalización adelantamiento [Vertical signs for passing]: Existing vertical signs for passing

A. Señalización adecuación altura[Vertical signs indicating height]: Height indicating vertical signs

A. Balizamiento [Road defences]: List of road defences

A. Balizamiento 2ª vuelta [New of Road defences]: 2nd list of road defences

A. Balizamiento comprobación estado [State control of defences]: Control of defence state

A. Estructuras [Structures]: Existing structures

A. Taludes [Slopes]: Existing slopes

A. Puntos inestabilidad [Points of instability]: Existing points of instability

A. Estudio velocidad [Speed studies]: Analysis of speeds

A. Ponderación de carreteras [Road weigh]: Importance given to road according to their road network

Configuración algoritmos [Algorithm configuration]*

Firmes-Límites CRT [Pavement-Limits for SCRIM Coefficient]: No action, preventive action and required intervention can be established to roads according to their category

FIRM-Variación estacional [Pavement-Seasonal variation]: Correction on SCRIM coefficient for data collected in winter according to surface layer material and traffic category

FIRM-Precios [Pavement-prices]: Prices established for each rehabilitation and maintenance work per m^2

FIRM-Prioridades [Pavement-Priorities]: Coefficients to ponderate the works, by road network, curve radius, if stretch with accident concentration, if urban]

FIRM-Porcentajes deterioros [Pavements-Defect percentages]: Limits for defect percentages can be included to indicate that an action is required

Ponderación carreteras [Road weigh]: Minimum traffic volume and interest category for each road to be considered

Variables Vida remanente [Variables for remaining life]: Values of deflection, traffic volumes, heavy traffic percentage, traffic increasing rate, lane distribution, truck weight, axle loads for each type, load distribution.

* Not all factors that modify the algorithm configuration are commented. Only those related to the pavements.

Table 6.9. Subsections of the area Obras [Works] in the Gestivía software and data included in each one

4) OBRAS [Works]
Obras [Works]
Obras en curso [Works conducted at present]: Projects conducted at present can be selected according to various criteria
Obras ejecutadas [Finished works]: Finished projects and works can be selected according to various criteria

Table 6.10. Subsections of the area Informes [Reports] in the Gestivía software and data included in each one

5) INFORMES [Reports]
Histórico reuniones [List of meetings]: List of the meetings about the pavement management systems that took place

Table 6.11. Subsections of the area Agenda 2G [Agenda next generation] in the Gestivía software and data included in each one

6) AGENDA 2G [Agenda next generation]
Accidentabilidad [Accident rates]: A new part of the PMS related to accidents

6.4.2. Inventory data

Inventory data of a road network include a huge quantity of data, which are necessary to identify and describe the different pavement sections in the territory. Available information in the PMS of the RGB is commented following the items indicated in subsection 2.7.1.

6.4.2.1. Road or/and segment identification data

e) **Road name and identification and segment identification**. For road identification, the road denominations are employed. They refer to the road code that each road have, those indicated in Annex 1, such as, N-634, AP-8, BI-625, etc. Sometimes, although an entire road has the same denomination, not all the length is managed by the RGB. It is the usual case where a road stretch crosses a village and that stretch was transferred to the municipality. Therefore, that length is managed by the local authorities and not by the RGB. In other situation, the road crosses other provinces and, hence, those parts do not belong to the territory of Biscay. When this kind of circumstances appears, the road with the same denomination is divided in various stretches, and they are called with consecutive codification, i.e, BI-625-1, BI-625-2, etc. Moreover, stretches transferred to municipalities or managed by other road administrations (such as N-629) are also referred and coded as BI-625-C1, for example. Furthermore, other stretch codes are also employed by the software, but they all are related univocally to a specific stretch.

f) **Initial and ending points, segment length and location referencing systems**. The initial and ending points of each segment are included. The location reference system deployed by the RGB is the **kilometre post** (or milepost) method, following a linear referencing system. Kilometre Posts (KP) are placed along the road, usually spaced 1 km approximately. It was considered the best method as it allows changing the kilometre post in case of new outlines without changing the kilometre post of the entire road. Hence, stretches that were not modified still maintain their references and there is no need to change the data referred to them. Kilometre post references are indicated as 0+0000, indicating the first number (before the sign of sum) the kilometre and the four digits after represent the distance in metres from the kilometre post indicated previously. Four digits are necessary because the distance between two KP can be longer than 1.000 m. Other times, distances between consecutive KP are less than 1.000 m. The total length of each stretch, after adding all the partial distances, is also included.

g) Geographic coordinates. Initial and final points of each stretch are also geographic located by means of geographic coordinates. Specifically, UTM coordinates are employed. Furthermore, the UTM coordinates for every 10 m of the stretch are also available. This information is also available in the public webpage of the RGB (www.bizkaia.eus), where all the roads managed by the RGB are displayed (Fig. 6.4). As seen, the denomination of the road, the code of the stretch and the Kilometre Post are available too.

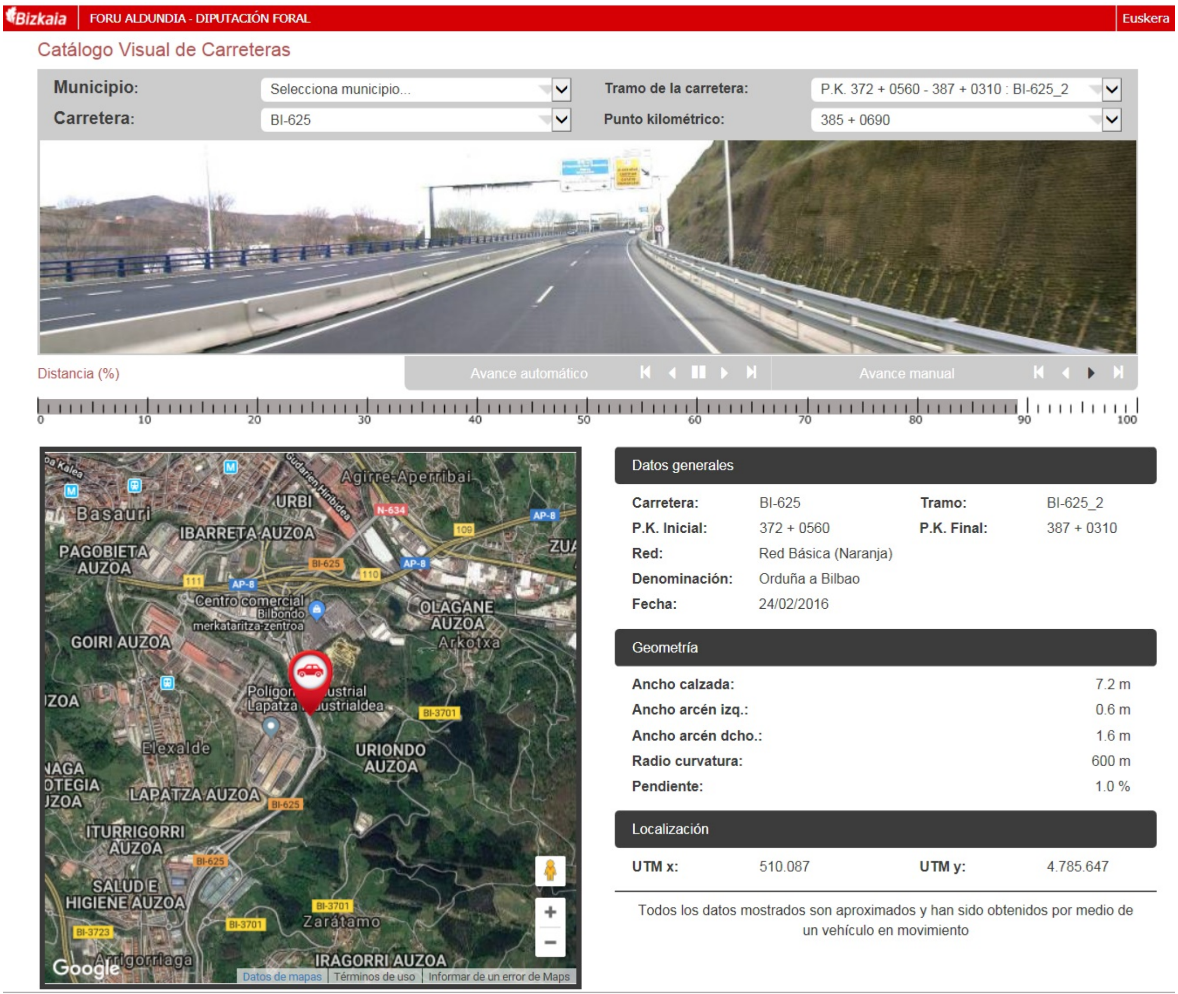


Fig. 6.4. Example of road visualization in the webpage of the RGB. Source: www.bizkaia.eus

h) Road classification, function, administrative zone and owner. Roads belonging to the network managed by the RGB are classified according to the criteria exposed in Table 6.. The function of each road is introduced in the network classification. The management of the entire network is divided in 4 conservation areas: **Área 1** includes Uribe-Kosta eta Gernikaldea areas. **Área 2** includes the Lea-Artibai and Durangoaldea areas. **Área** 3 includes Nerbioi-Arratia and Enkarterriak areas. **Área 4** includes the metropolitan area around Bilbao. For each stretch, the corresponding conservation area (Area 1, 2, 3, or 4) or the road agency responsible of its maintenance (Interbiak, Autonomous Port, Ministry of Public works or transferred to local authorities) are indicated, as shown in Table 6.1.

6.4.2.2. Carriageway geometric data

Geometric data of the road are also included in the PMS of the RGB. To indicate if the stretch is single or double carriageway and the number of lanes, the PMS follows the instructions of the "*Guía para la actualización del inventario de firmes de la Red de Carreteras del Estado*" [Guide for updating the pavement database of the State Road Network] (Ministerio de Fomento, 2011a).

If the road has a unique carriageway, that carriageway is indicated as 0. When there are two carriageways, the one on the right in the ascending direction is called 1 and the one on the left is called 2. When there are more than 2 carriageways, numbers 1 and 2 are reserved to the most important ones. If all the carriageways have the same importance, numbers 1 and 2 are applied to the ones in the centre, following the rule indicated for only 2 carriageways. The carriages with the same direction as the increasing direction are denominated with odd numbers (1, 3, 5, etc.). In the contrary direction, carriageways use even numbers (2, 4, 6, etc.). Exposed cases are summarized in Fig. 6.5.

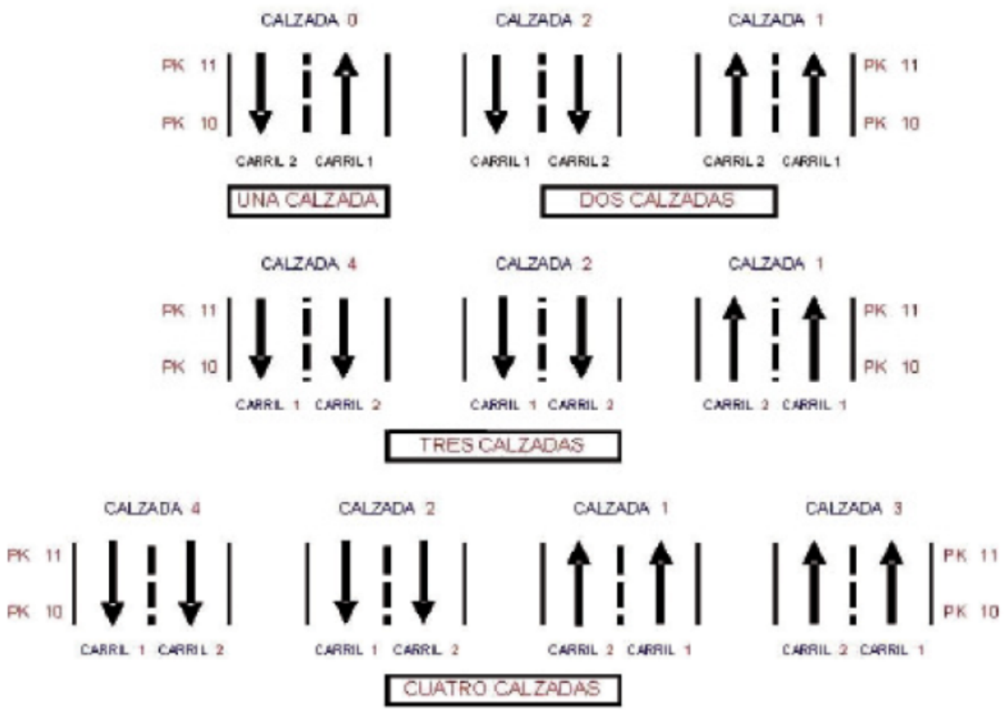


Fig. 6.5. Rules for denominating carriageways (Ministerio de Fomento, 2011a).

For identifying the lanes of each carriageway, the following rules are employed.

- **In conventional roads with a unique carriageway**, the lane on the right in the direction of ascending Kilometre Posts is denominated 1 and the one on the left 2. In case of auxiliary lanes, it is used number 7 to denominate the one on the right in the direction of ascending KP, and 8 for the one on the left. In case of lane for slow vehicles, they are referred as 0.

- **In double carriageway roads**, the lanes are denominated with correlative numbers, ascending from the exterior lane to the interior lane (0, 1, 2, 3, 4, etc.). It starts with 0 if the exterior lane is a lane reserved for slow vehicles; or with 1 if the exterior lane is a conventional lane. In case of auxiliary lanes, they are referred as 7 to an exterior auxiliary lane and 8 to the interior one. Fig. 6.6 shows schemes of some situations for denominating lanes.

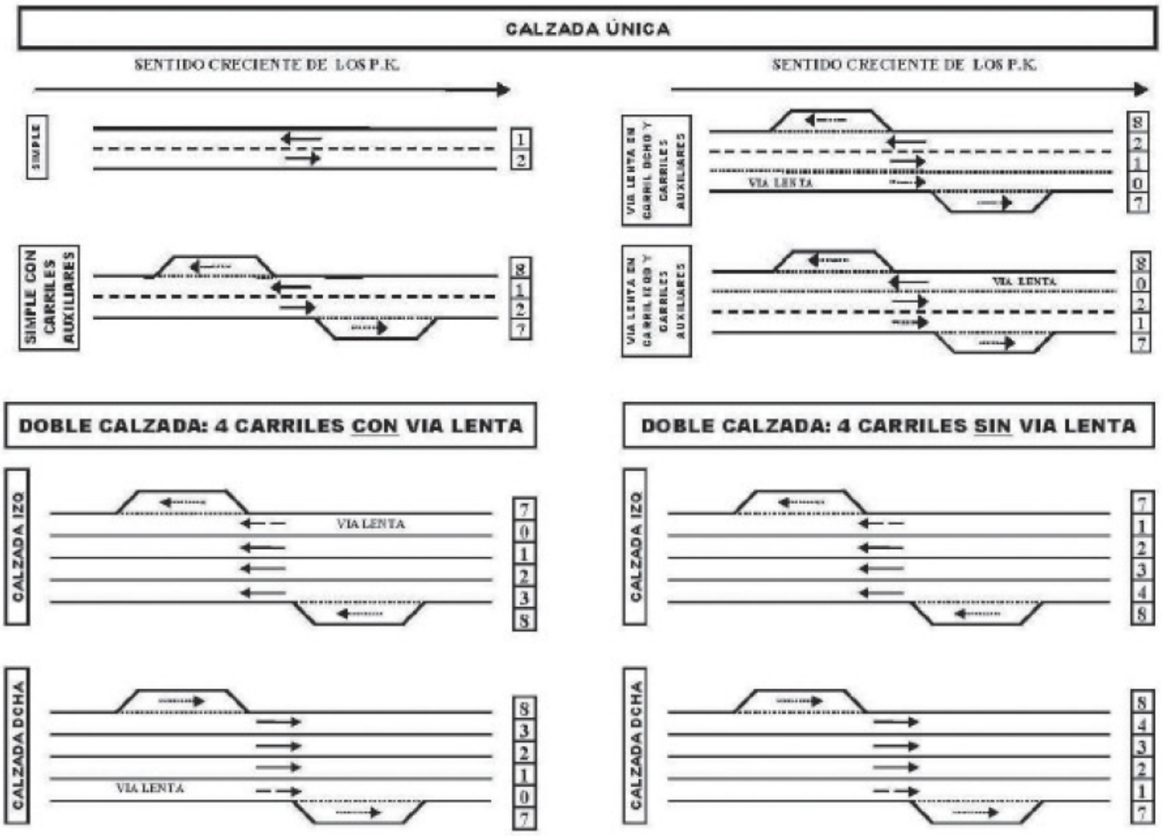


Fig. 6.6. Scheme of lane denomination in some situations (Ministerio de Fomento, 2011a).

The lane width, the total carriageway width and the shoulder width are also introduced in the PMS. Data are available for every 10 m of the road, as seen in Fig. 6.4. The radius of the curvature and the longitudinal slope are also available in the webpage of the RGB (Fig. 6.4). More geometric data, such as the visibility distance, the inverse visibility distance are also stored in the database.

Finally, the nature of the stretch is also included. It is indicated if the stretch is over an structure or tunnel, if urban areas are crossed. These stretches must be at least 200 m long to be indicated. Examples of possible nature of the stretch are: tunnel, urban area, by-pass, structure, interurban, port area, industrial area or experimental zone. Other observation can be also introduced.

6.4.2.3. Pavement structure, maintenance and rehabilitation works and projects

The Guide for updating the pavement database of the State Road Network (Ministerio de Fomento, 2011a) proposes a methodology to introduce pavement sections in the database. The methodology uses two types of files: an initial pavement file and a rehabilitation and maintenance file. The first one introduces the structural section of the entire pavement, indicating if it is one of the solutions of the Spanish pavement design guides [*Norma 6.1. IC "Firmes flexible" and Norma 6.2. IC "Firmes rígidos"* (Ministerio de Obras Públicas, 1976a); *Secciones de firme en autovías* (Ministerio de Obras Públicas y Urbanismo, 1986b); *Instrucción 6.1 y 6.2 Secciones de Firme* (Ministerio de Obras Públicas y Urbanismo, 1989a); *6.1-IC Secciones de Firme* (Ministerio de Fomento, 2003b)], and records all the materials employed and their thickness. Similarly, the Rehabilitation and Maintenance file introduces maintenance and rehabilitation activities carried out over an existing pavement. Therefore, the methodology allows observing the last transverse section of the stretch, the previous ones until the initial one.

The Pavement Management System of the Regional Government of Biscay utilizes some of the ideas of the mentioned guide (Ministerio de Fomento, 2011a). Nonetheless, instead of introducing information about the existing pavement structure or the maintenance history individually for each stretch, these data are introduced by means of the projects that were carried out in the road. As the methodology employed is the key factor for the development of pavement deterioration models, it is further and exhaustively described in section 6.5.

6.4.2.4. Other structure data

Interchanges, bridges, culvert and drainage structures are also included in the software. There are special sections which list all the features in each road. It is not further commented since it is not necessary for the development of deterioration models.

6.4.2.5. Appurtenance features

Other information about features in the road network is also registered in the PMS: Traffic control devices, signs, lighting, road defences, guardrail, barriers are also included in the database. Once again, it is not further commented as it is not necessary for the development of deterioration models.

6.4.3. Traffic data

The Regional Government of Biscay has recorded traffic data since competences about the road network were transferred. Additionally, the RGB published annually a report of the traffic volume data registered in the road network in a document entitled "Evolution of the traffic in the roads of Biscay", specifying to which year it refers (Pérez-Acebo, 2018). It is a complete document where all the obtained data are presented, commented and numerous tables and figures are included to better show the data.

By means of the traffic data collection programme, in 2016 the RGB controlled 1.304,1 km of the 1.309,2 km that it manages, a 99,6% of the road network. Those road stretches that are not controlled belong to the local network (Table 6.3). Table 6. indicates that there are 56,2 km that do not belong to the RGB but they are ramps, stretches transferred to local authorities and a stretch of the A-625 which belongs to the province of Alava.

Table 6.12. Distribution of road length by network type and controlled length. Source: Evolución del tráfico en las carreteras de Bizkaia 2016 (Diputación Foral de Bizkaia, 2017).

Network type	Length of the network (km)			Number of stations
	Controlled	**Catalogued**	**%**	
Preferential interest (Red)	248,6 *	245,7	100,0	99
Basic (Orange)	210,4	210,4	100,0	91
Complementary (Blue)	32,5 **	28,9	100,0	22
Provincial (Green)	209,5	209,5	100,0	75
Local (Yellow)	603,1	314,7	98,1	183
Total of catalogued network	**1.304,1**	**1309,2**	**99,6**	**470**
Network not classified	5,6			16
Transferred stretches	50,6			30
Total of not catalogued network	**56,2**			**46**
Total Controlled network	**1360,3**			**516**

* It is included a stretch of the AP-8 that is not included in the catalogue but it was controlled

** It was included part of the BI-737, already transferred, but it was included not to modify the traffic values of the corridor.

The RGB employed 516 traffic counter stations in 2016. Some stations are permanent. Until 1990, there were only 3 permanent stations in Biscay and hence, the traffic of the rest of the network was deducted from primary stations (Pérez-Acebo, 2018). In 1991, the RGB equipped 97 stations with magnetic induction counters which could be employed as permanent stations if necessary. At present, there are 124 stations equipped with magnetic devices and 22 are integrated in the telemetric control network of the metropolitan area. In 2016, 18 stations were selected as permanent according to representation and space distribution criteria. Moreover, there were 41 primary stations 139 secondary stations, 256 cover stations, 29 are under a concession and 24 are controlled telemetrically (Pérez-Acebo, 2018).

Traffic volume data since 2000 are introduced in the database. Previous data are registered in documents but they are not recorded in the software. For each station, which represent a specific stretch of a road, the Annual Average Daily Traffic (AADT), the percentage of heavy vehicle, the Annual Average Daily Traffic of Heavy Vehicles (H.AADT) in the project lane, and the traffic category are indicated in each year, from 2000 to present (Table 6.). As commented, according to Spanish regulations (Ministerio de Fomento, 2003b), a heavy vehicle is considered when it weights over 3.500 kg. This is the only information about heavy traffic. No data about Equivalent Single Axle Load is provided.

Table 6.13. Example of information about traffic volumes introduced in the PMS of the RGB.

Traffic counter station	Road	Road carriageway	Initial KP	Final KP	Length of the stretch (m)	Lane type	AADT 2000	% of heavy vehicles, 2000	H.AADT in project lane, 2000	Traffic category
115A	BI-625	BI-625 Carriageway 0	382+0200	384+0050	2030	2 lanes	18.892	18.4	1.739	T1
154A	BI-625	BI-625 Carriageway 0	384+0050	386+0050	850	2 lanes	25.491	18	2.295	T0
153A	BI-625	BI-625 Ascending (Carriage 1)	386+0050	387+0310	1270	2 lanes per direction	29.062	19	2.761	T0

6.4.4. Environmental data

Environmental data are not introduced in Gestivía, the Pavement Management System of the Regional Government of Biscay. Biscay is a small province with only 2.217 km^2 and it has an oceanic climate, homogeneous over the entire province, with high precipitations all year round (1.200 mm/year) and moderate temperatures. Temperatures can be more extreme in the higher lands of inner Biscay, where snow is more common during winter. The average high temperature in Bilbao, the capital is between 13 ºC in January and 26 ºC in August.

Therefore, it can be stated that environmental agents affect uniformly the entire province and there is not differences about climate in the entire road network. Consequently, there is no division of the territory according to climate variations, as suggested by AASHTO (2012) due to the small size when compared to states of the USA. As indicated in Table 6., data about rainfall levels are incorporated to Gestivía. Monthly rainfall data from 2000 to present according to the Spanish Meteorology Agency and monthly rainfall data from 2011 to present from the Basque Meteorology Agency, and divided by conservation areas (6.4.2.1 d) are included in the PMS. These data are included in the sub-section of Accidents of Inventories.

Moreover, there is environmental an additional climate data introduced in the database. As commented in section 5.4.5.1, rainfalls influence the skid resistance values. Therefore, with the aim of analyzing the possible influence of the rainfalls occurred before the friction data collection, the precipitation in mm collected 15 days before the data collection of 2016 in the nearest meteorological station is recorded.

6.5. Methodology for introducing pavement structures in Gestivía

6.5.1. Introduction of projects

As commented, information about pavement sections is introduced by means of the projects that were carried out in the road network of Biscay. This methodology allows including real information in the database, although it can be incomplete. Despite of its incompleteness, all the information is real and it was verified. It can be described as information introducing by projects, instead of examining all the road stretches, which would be more expensive.

Most of the projects carried out from 1983, where competences about roads were transferred to the Regional Government of Biscay, are introduced in the database. Some previous information is available, from the time when the roads were managed by the Spanish Government; but the majority of this information was stored in paper format and was lost due to the flooding of Bilbao in 1983. From 1983, the most important projects, especially those that imply new outlines, are available and were introduced, and from 1990 it can be said that all the construction projects are included in the database. From 2000, all the rehabilitation and maintenance works are also registered and included in Gestivía. Previous rehabilitation and maintenance projects before 2000 are not always recorded and, hence, information from projects that were carried out in the 80s and that have not been maintained since then may be analyzed with caution due to the lack of information. However, this is not habitual and a complete history is available for the majority of stretches.

For both construction projects of a new stretch and rehabilitation or maintenance projects, the following information is introduced in the software:

- **Code.** A code is assigned to each project, starting from 1 (PROJ-1, PROJ-2, etc.) The order was established according to the order that was employed to start analyzing all the roads of Biscay. Normally, most of the projects of a road are gathered together, in consecutive number. When all the projects were introduced, new projects take the following number. By 2017, more than 2000 projects are registered, including maintenance and rehabilitation activities.
- **Road.** The following information is included:
    - **Road denomination**. The denomination of the main road where the project takes place
    - **Carriageway.** The carriageway where the project takes place: the unique one in case of conventional roads, the ascending or the descending carriageway in double carriageways or a ramp.
    - **Initial and ending KP and length**. The initial and finishing KP of the main road and the coordinates of these points. The real length between the initial and ending points is also recorded.
- **Project data.** The project data include the following information:
    - **Name of the project**. The complete name of the project, as indicated in the title.
    - **Author.** The author, usually an engineering company or some companies together, which wrote the project. In minor projects, the author may be the road agency itself.
    - **Kind of project**. It is specified if the project implies a variation of the pavement structure or not. On one hand, it can be classified as *Project/Surface work*, which implies that pavement section is modified. On the other hand, if classified as *Patching*, it means that punctual reparations were made, not affecting the pavement structure. If a patching work is carried, further information is introduced, such as, the date, the kind of work and if it affected the entire carriageway or only one or some lanes.
    - **Origin.** It specifies where the project was found. As commented, previous to the start of the PMS, projects were located in different places because different departments conducted them. This section indicates the department or place (even not physical) where the project was found. Some of the places introduced are the Archive of the RGB, the conservation department, the inventory section, Interbiak (the public company that manage some projects), the Pavement Plan (an important plan carried out in 1990, which included several actuations in the entire network to improve the quality of the pavement sections), the webpage of the RGB (www.bizkaia.eus), and the BOE and BOPV (Official Bulletin of the State and the Basque Government, respectively. Some projects were not stored, but some information could be found in the Official Bulletins and, hence, some data can be obtained).
    - **Department.** It indicates the department responsible of the project. Some of the departments are: Construction department, conservation area, management area, modernization area and other.
    - **Conservation area.** It indicates in which conservation area is included the project. It can

be Conservation Area 1, 2, 3 or 4, a concession stretch or a transferred stretch, as commented in subsection 6.4.2.1.

- **Project type.** It indicates the type of project that was introduced to include the works in the database. It can be a *planning project* (a project related to a general planning), a *construction project* (the project where the entire project is described in detail), *a complementary project* (a project that is written when another one was being carried out to included additional works), *a modified project* (the project that is modified during the works) and *final project* (the project that is prepared when finishing the works, including the as-built projects). Other type of projects can be introduced, such as, *ante-project*, *outline study*, *informative study*, *outline project*, *provisional final project*, etc.
- **Construction type**. It indicates the construction that is conducted from the point of view of the pavement section. It distinguishes between *new outline* (when a new outline is carried out and, hence, a new pavement section is made), a *pavement improvement* (when some new layers are added to existing pavement section) and a *pavement improvement and outline modification* (when the outline is partially modified and some pavement layers are renovated). Other construction types, not so frequent, are a *project for a new third lane, road safety project, reconstruction after damages* (floods, etc.), etc.
- **Expedient number**. It shows the expedient number of the project, as previously referred in the various databases of the RGB.
- **Date of the project**. It indicates when the project was finished by the author (usually an engineering company).
- **Date of reception of the works**. It indicates when the works were finished and, therefore, the day when the project is open to traffic. If exact date is not available, 2 year after the redaction of the project is introduced in the database of the PMS. It is an important data.
- **Project conducted**. It must be indicated if the project was actually carried out or if not. Sometimes, although projects were prepared, they were not finally performed and, hence, they do not alter pavement section.
- **Fiction project**. It must be specified if it is a fiction project. The vast majority of projects are not fiction, but sometimes, there are no documents about a road, but the road exists and it is known that a specific surface layer is placed. Then, in these cases, a fiction project is introduced in Gestivia, in order to indicate that this kind of surface layer is present there, although further information is not available (age, rehabilitation history, etc.).
- **Observations**. Some notes can be introduced to explain some unusual data of the project.

* **Pavement information**. The following information about the pavement structure is included.
    - **Pavement code**. Within a project, different works and pavement sections can be carried out in the same road or in other roads, therefore, each individual work is recorded with an individual code. It uses the project code followed by a hyphen and some digits. For example, PROJ-30-1, PROJ-321-2020.
    - **Road denomination**. It specifies all the roads that are affected by the project. Although the main road was previously specified in the project data, a project usually implies

modifications in various roads due to new links with them. Therefore, all the affected roads within the project are indicated.

- **Road data**. In each of the roads affected by the project, it is indicated the carriageway where works are carried out (the unique carriageway in conventional roads or the ascending or descending carriageway in double carriageway highways), the initial and final KP of the stretch, the UTM coordinates (X and Y) of the initial point and the total length of the stretch. Furthermore, it is stated if both lanes of the carriageway are included and if not, the affected lanes are indicated. Finally, the nature of the stretch is also included: structure, interurban, urban, tunnel, by-pass, port area or nothing special.
- **Type of pavement action**. It is perhaps one of the most important data in the database. It indicates if the pavement structures carried out in the project represent a **new outline** (as in new outlines of new roads such as a by-pass or a new motorway stretch) or if is a **rehabilitation or maintenance work** in the pavement section, implying that previous pavement structure must be considered. Therefore, projects can be differentiated. On one hand, initial pavement structures are identified, by means of new outline projects, and, on the other hand, rehabilitation or maintenance works are considered independently from new stretches. Sometimes, rehabilitation and maintenance projects are applied over pavement structures that are unknown. Consequently, only layers on the surface are known, but not the entire section. However, at least, part of the existing pavement section can be known and analyzed.
- **Milling depth**. If milling activities are conducted, only in rehabilitation and maintenance activities, milling depth over existing pavement can be introduced in the database. The information about milling is limited to the milling depth (in cm) and it is not as exhaustive as proposed by Ministerio de Fomento (2011a), because reclaimed activities are not frequent in Biscay.
- **Bituminous base layer**. It indicates the denomination of the employed material in the bituminous base layer, according to the present regulation, the previous denomination (from Table 6.), the used binder (but normally is not indicated) and the thickness of the layer, in cm.
- **Bituminous intermediate (or binder) layer**. It indicates the denomination of the employed material in the bituminous binder layer according to the present regulation, the previous denomination (from Table 6.), the used binder (but normally is not indicated) and the thickness of the layer, in cm.
- **Bituminous surface layer**. It indicates the material used in the bituminous surface layer (from Table 6.), the specific denomination of the employed material according to the present regulation, the previous denomination (from Table 6.), the used binder (but normally is not indicated) and the thickness of the layer, in cm.
- **Subbase layer**. It indicates the material employed in the subbase layer or layers, among the materials that can be employed (Table 6.) and the thickness of each one, in cm.
- **Observations**. Any observation related to the pavement structure can be included to clarify

any doubts or to leave an explanation about anything discordant.

Table 6.14. Possible materials in bituminous base layers (Ministerio de Fomento, 2011a)

Bituminous base layer	
Denomination according to UNE-EN 13108	**Previous denomination**
AC 32 base S	S25
AC 22 base G	G20
AC 32 base G	G25
AC 22 base S MAM	MAM

Table 6.15. Possible materials in bituminous intermediate (or binder) layers (Ministerio de Fomento, 2011a)

Bituminous intermediate layer	
Denomination according to UNE-EN 13108	**Previous denomination**
AC 22 bin D base S	D20
AC 22 bin SG	S20
AC 32 bin S	S25
AC 22 bin S MAM	MAM

Table 6.16. Possible materials in surface layers (Ministerio de Fomento, 2011a)

Material for the surface layer	
AC	Asphalt concrete. Hot mix asphalt concrete
BBTM	Discontinuous bituminous mixing
PA	Porous asphalt
LB1, LB2, LB3, LB4	Slurry
TS	Single surface dressing
DTS	Double surface dressing
HF	Concrete pavement
HM	Lean concrete
O	Other material

Table 6.17. Possible materials in bituminous surface layers (Ministerio de Fomento, 2011a)

Bituminous surface layer		
Bituminous layer type	**Denomination according to UNE-EN 13108**	**Previous denomination**
Discontinuous mixing	BBTM 8 A	F8
	BBTM 8 B	M8
	BBTM 11 A	F10
	BBTM 11 B	M10
Porous Asphalt	PA 11	PA 12
	PA 16	-
Asphalt Concrete (Hot Mix Asphalts)	AC 16 surf D	D12
	AC 16 surf S	S12
	AC 22 surf D	D20
	AC 22 surf S	S20

Table 6.18. Possible materials in subbase layers (Ministerio de Fomento, 2011a).

Materials in subbase layers	
GC	Gravel-cement (Cement treated base material)
SC	Soil-Cement (Cement treated base material)
MG	Unbound material (Untreated crushed stone)
HF	Concrete pavement
HM	Lean concrete
GE	Slug and gravel

With all the projects introduced in the database, it is possible to filter them by any of the items described. Projects of a specific road can be selected; projects that imply a new outline can be differentiated, etc. It is a versatile tool with many possibilities.

As seen, data are introduced in the database following several of the ideas from Ministerio de Fomento (2011a). However, there are not different files for initial pavement structures and for rehabilitated (or maintained) pavements. All the information about the pavement structure is introduced similarly in both cases, but, it is indicated if the pavement can be considered as the initial pavement or as rehabilitation (or maintenance) work.

6.5.2. Visualization of pavement structures in Gestivía

When all the information of the projects is introduced in the software of Gestivía, it is possible to visualize this information in two different ways: by means of the Pavement Structure file or by means of the Surface Layer file. It is an adequate method to visualize separately the complete structure of the pavement and the surface properties of the tyre-pavement contact layer, respectively.

6.5.2.1. Pavement Structure file

The Pavement Structure file allows observing an entire road, divided in different stretches, with the known pavement structure. Sometimes the pavement transverse section is totally registered and other times, only the layers that were introduced by means of projects are available. The file provides a table like the one shown in Table 6..

Table 6.19. Example of Pavement Structure file in road BI-2701.

Road carriageway	Initial KP	Final KP	Length between KP (m)	Carriageway	Lane 1	Lane 2	Lane 3	Lane 4	Lane 5	Lane 6	Nature of the stretch	Date of initial pavement	Subgrade
BI-2701_Main	27+0000	27+0200	200	0	Yes	Yes					Interurban	01/06/1994	
BI-2701_Main	27+0200	27+0340	140	0	Yes	Yes	Yes	Yes			Interurban	01/06/1994	
BI-2701_Main	27+0340	27+0550	310	0	Yes	Yes					Interurban	01/06/1994	
BI-2701_Main	27+0550	27+0580	30	0	Yes	Yes					Interurban	01/06/1994	
BI-2701_Main	27+0580	27+0750	170	0	Yes	Yes					Interurban	01/06/1988	
BI-2701_Main	27+0750	27+0900	150	0	Yes	Yes					Interurban	01/06/1988	
BI-2701_Main	27+0900	27+0960	60	0	Yes	Yes					Interurban		
BI-2701_Main	27+0960	28+0050	150	0	Yes	Yes					Interurban		
BI-2701_Main	28+0050	28+0160	110	0	Yes	Yes					Interurban		
BI-2701_Main	28+0160	28+0260	100	0	Yes	Yes					Interurban		
BI-2701_Main	28+0260	28+0500	240	0	Yes	Yes					Interurban		

Initial KP	Final KP	Bituminous mixing layers thickness (cm)	Concrete layer thickness (cm)	Lean concrete layer thickness (cm)	Gravel-cement layer thickness (cm)	Soil-cement layer thickness (cm)	Unbound material layer thickness (cm)	Slag and gravel layer thickness (cm)	Other material layer thickness (cm)	Pavement classification	Surface layer-Material	Surface layer-Material denomination	Surface layer-Binder	Surface layer-Previous denomination	Surface layer thickness (cm)	Surface layer-Other material	Surface layer data
27+0000	27+0200	19	0	0	0	0	25	0	0		LB2				0		01/12/2004
27+0200	27+0340	19	0	0	0	0	25	0	0		LB2				0		01/12/2004
27+0340	27+0550	19	0	0	0	0	25	0	0		LB2				0		01/12/2004
27+0550	27+0580	24	0	0	0	0	25	0	0		AC	AC 16 surf S		S12	5		01/12/2004
27+0580	27+0750	26	0	0	0	0	15	30	0		AC	AC 16 surf S		S12	5		01/12/2004
27+0750	27+0900	30	0	0	0	0	15	30	0		AC	AC 16 surf S		S12	5		01/12/2004
27+0900	27+0960	15									AC	AC 16 surf S		S12	5		01/12/2004
27+0960	28+0050	9									AC	AC 16 surf S		S12	5		01/12/2004
28+0050	28+0160	0									LB2				0		01/12/2004
28+0160	28+0260	5										AC 16 surf S		S12	5		01/06/2009
28+0260	28+0500	3										BBTM 11 A		F10	3		01/06/2012

Each data displayed is commented:

- **Road carriageway**. The main carriageway in conventional roads or the ascending or descending carriageway (according to KP) in double carriageway highways.
- **Initial and final KP.**: The initial and final KP of the stretch with similar characteristics
- **Length between KPs**. The real distance between the initial and final KP, taking into account the real length between KP.
- **Carriageway**. 0, 1 or 2 is indicated, according to the instructions of Ministerio de Fomento (2011a).
- **Lane 1, 2, 3, 4, 5, 6.** Lanes affected by the description of the stretch are indicated. It also serves to indicate the number of lanes of the stretch. Sometimes, various stretches are indicated with the same pavement structure, if variable number of lanes ispresent.
- **Nature of the stretch**. The nature of the stretch is shown: interurban, urban, by-pass, port area, etc.
- **Date of initial pavement**. One of the most important data of the file. If the pavement section is introduced by means of a New Outline project, as commented in 6.5.1, it is mentioned the date when the road was opened to traffic. Apart from the date, it indicates that the complete pavement structure is known due to a New Outline project. It allows knowing the initial data when a new stretch was carried out. If the data is not available, it means that partial layers are only known in that stretch, by means of Rehabilitation and Maintenance projects.
- **Subgrade**. If available, it indicates the classification of the subgrade according to the Spanish pavement design guides (Ministerio de Obras Públicas, 1976a; Ministerio de Obras Públicas y Urbanismo, 1986b; 1989a; Ministerio de Fomento, 2003b) or the pavement design guide of the Basque Country (Departamento de Transportes y Obras Públicas, 2006; Departamento de Vivienda, Obras Públicas y Transportes, 2012). However, this information is not usually available.
- **Bituminous mixing layer thickness**. It indicates the total thickness of the bituminous mixing in the section, in cm. If the complete pavement section is known, as a consequence of a New Outline project (and subsequent rehabilitation and maintenance projects), the total thickness of the bituminous mixing in the section is displayed. If the initial pavement of the stretch is unknown, this column shows the thickness of the bituminous layers that are known (as a consequence of introduction of Rehabilitation and Maintenance projects), but the real total thickness of bituminous mixings in this section can be higher. It indicates the known real thickness.
- **Concrete layer thickness (HF)**. It indicates the total thickness of the concrete layer in the section, in cm.
- **Lean concrete layer thickness (HM)**. It indicates the total thickness of the lean concrete layer in the section, in cm.
- **Cement treated base material layer thickness (GC)**. It indicates the total thickness of the cement treated base material layer in the section, in cm, usually designed as Gravel-cement (GC).
- **Soil-cement layer thickness (SC)**. It indicates the total thickness of the soil-cement layer in the section, in cm. It is possible to have a cement treated material (GC) and soil-cement layer because it is a possible section in design guides.
- **Unbound material layer thickness (MG)**. It indicates the total thickness of the unbound material

layer in the section, in cm. Crushed stone (Zahorra in Spanish) is the most employed material in this kind of layers.

- **Gravel and slag layer thickness (GE)**. It indicates the total thickness of the gravel and slag layer in the section, in cm. Due to the availability of slag in Biscay as a result of the presence of furnaces; it is a material that was commonly employed in roads.
- **Other material layer thickness (OM)**. It indicates the total thickness of the layers in the section with materials different from the ones presented before, in cm.
- **Pavement classification**. It should classify the pavement in flexible, semi-flexible, semi-rigid and rigid, but normally, this column is not used.
- **Surface layer-material.** It indicates the material employed in the surface layer, from the list of Table 6..
- **Surface layer-**Material denomination. It indicates the denomination of the material employed in the surface layer, from the ones in Table 6..
- **Surface layer-Previous denomination.** It indicates the previous denomination of the material employed in the surface layer, from the ones in Table 6.
- **Surface layer-Other material**. It indicates a material different from the ones in Table 6., employed in the surface layer.
- **Surface layer-Data**. It indicates the data when the present surface was carried out. If the data is the same as in Date of the initial pavement, it implies that no rehabilitation and maintenance activities were carried out. If not, it means that some rehabilitation and maintenance works were carried out. When the Data of the initial pavement is not available, it shows the last work conducted in the road, which is responsible of the surface layer described previously.

As observed, the Pavement Structure File is a useful source of information to know the sections that can be found in a road. It is highlighted the stretches where the entire pavement structure is known and shows all the available data. Even for stretches with no complete pavement section, it provides the available information.

Nevertheless, some deficiencies can be detected.

- It does not provide the code of the project of the Initial Pavement. Moreover, when only surface information is available, the code of the Rehabilitation and Maintenance project is not shown.
- In stretches with initial pavement information, it does not show the complete structure. It only displays the total thickness of the bituminous layers, but without indicating the layers that compose the pavement. It must be checked in the information of the project.
- The files do not show information about milling. It must be found directly in the project data.
- Although information about the initial pavement and the present surface layer is available, the file does not show the possible intermediate rehabilitation and maintenance activities that can be conducted in the stretch. Sometimes it can be detected due to differences in the bituminous mixing layer thickness, but there is no information about those intermediate projects. In these cases, the information must be found searching between all the projects carried in that road.
- In the surface layer, only materials indicated in Table 6. are displayed. If the present pavement

surface is slurry or single dressing, this file does not show it. These types of materials are considered not to add more thickness to the pavement section. Consequently, they are not registered as the surface material because they are said not to modify the pavement structure. This information must be checked in the Surface Layer file (subsection 6.5.2.2).

6.5.2.2. Surface Layer file

The Surface Layer file allows observing an entire road, divided in different stretches as a function of the material in the surface layer. Even in stretches where all the pavement structure is unknown, information can be available because the rehabilitation and maintenance projects since 2000 have been recorded in the database. This file also provides information about superficial treatments if they were conducted, differentiating them from the surface materials. The file displays a table like the one shown in Table 6.. Each data displayed is commented.

- **Code**. The code of the project is shown. It indicates the specific code of the project, including the subdivision of the work, with the two numbers as explained in Pavement Code.
- **Road carriageway**. Similar to the Pavement Structure File
- **Initial and final KP.**: Similar to the Pavement Structure File
- **Length between KPs**. Similar to the Pavement Structure File
- **Carriageway**. Similar to the Pavement Structure File
- **Lane 1, 2, 3, 4, 5, 6.** Similar to the Pavement Structure File.
- **Surface layer-Material**. Similar to the Pavement Structure File.
- **Surface layer-Material type**. It indicates the nature of the material employed in the surface layer.
- **Surface layer-Material denomination**. Similar to the Pavement Structure File.
- **Surface layer-Binder**. Similar to the Pavement Structure File
- **Surface layer-Previous denomination**. Similar to the Pavement Structure File
- **Surface layer-Date**. Similar to the Pavement Structure File
- **Surface treatment-Type**. It indicates the surface treatment applied to the pavement, if conducted. As previously commented, Gestivía understands differently the surface layers. On one hand, materials shown in Table 6. are introduced with their thickness and they modify the pavement structure. On the other hand, superficial treatments, such as slurries or surface dressings, are introduced as if they do not modify the pavement structure and its thickness.
- **Surface treatment-Data**. It indicates the data when the surface treatment was carried out. Obviously, it must be after the Surface layer-Data.

The Surface Layer file is another useful and key output of the database, as it allows observing the different stretches in which a road can be divided taking into account the surface material. It divides a long stretch with the same surface material in various stretches according to the last surface treatments applied. Even if the treatment is similar, it distinguishes as a function of the date. This file complements the Pavement Structure file, making a zoom on the surface layer, where some of the main characteristics and properties of the pavement rely. It is especially useful in the cases in which the complete pavement section is unknown, as it provides at least information about the surface layer. Moreover, in these files it is shown the project code that made the present surface layer, and therefore, that project can be found more easily in the database.

Nevertheless, some deficiencies can be underlined:

- Although information about present surface characteristics is available, it is not possible to know all the rehabilitation and maintenance history of the stretch. Once again, all the projects in that road must be consulted to know the work history. Only present information is displayed.
- Although the code is displayed in the table, if there is a surface material and a surface treatment, it refers to the last one, the surface treatment and, hence, the code of the project about the surface material is unknown. However, as two data are available, surface layer-data and surface treatment-data, research is not so difficult.
- Some characteristics about the surface layer are not displayed, such as, the Polished Stone Value of the aggregates, etc.

Table 6.20. Example of Surface Layer file in road BI-2701.

Project code	Road carriageway	Initial KP	Final KP	Length between KP (m)	Carriageway	Lane 1	Lane 2	Lane 3	Lane 4	Lane 5	Lane 6
PROJ-926	BI-2701_Main	27+0000	27+0130	130	0	Yes	Yes				
PROJ-2098	BI-2701_Main	27+0130	27+0200	70	0	Yes	Yes				
PROJ-2098	BI-2701_Main	27+0200	27+0340	140	0	Yes	Yes	Yes	Yes		
PROJ-2098	BI-2701_Main	27+0340	27+0420	80	0	Yes	Yes				
PROJ-926	BI-2701_Main	27+0420	27+0840	420	0	Yes	Yes				
PROJ-116-92	BI-2701_Main	27+0840	28+0050	270	0	Yes	Yes				
PROJ-116-1	BI-2701_Main	28+0050	28+0160	110	0	Yes	Yes				
PROJ-927-952	BI-2701_Main	28+0160	28+0260	100	0	Yes	Yes				
PROJ-1562-1256	BI-2701_Main	28+0260	29+0280	540	0	Yes	Yes				

Initial KP	Final KP	Surface layer-Material	Surface layer-Material type	Surface layer-Material denomination	Surface layer-Binder	Surface layer-Previous denomination	Surface layer thickness (cm)	Surface layer-Other material	Surface layer-data	Surface treatment - Type	Surface treatment - Data
27+0000	27+0130	AC		AC 16 surf S		S12	5		01/06/1994	LB2	01/07/2012
27+0130	27+0200	AC		AC 16 surf S		S12	5		01/06/1994	MICROMILLING	08/05/2017
27+0200	27+0340	AC		AC 16 surf S		S12	5		01/06/1994	MICROMILLING	08/05/2017
27+0340	27+0420	AC		AC 16 surf S		S12	5		01/06/1994	MICROFRESADO	08/05/2017
27+0420	27+0840	AC		AC 16 surf S		S12	5		01/06/1994	LB2	01/07/2012
27+0840	28+0050	AC		AC 16 surf S		S12	5		01/12/2004		
28+0050	28+0160									LB2	01/12/2004
28+0160	28+0260			AC 16 surf S		S12	5		01/06/2009		
28+0260	29+0280			BBTM 11 A		F10	3		01/06/2012		

6.6. Pavement condition data

As fundamental input in any pavement management system, pavement condition data must be introduced to produce deterioration prediction models (Figure 2.3). In Chapter 3, the main pavement properties were commented, the usual devices to measure them were described and the most used indicators or indices were listed and discussed. The Regional Government of Biscay also recorded these data during the condition data collections. These data includes roughness, skid resistance, structural capacity and surface defects. They are described individually, with special attention to the roughness and skid resistance data.

6.6.1. Pavement Roughness

As many other road agencies over the world, the Regional Government of Biscay collected roughness data of the road network under its responsibility by means of the International Roughness Index (IRI), which become the most widely employed index in pavement roughness and, even, in general pavement characterization. IRI data were obtained by means of an inertial profiler, i.e. a precise profile belonging to Class I, as categorized by Sayers *et al*. (1986b). IRI became a compulsory index to check the quality of new roads or existing one after being developed by Sayers *et al.* (1986b), including it in 1989 in the Spanish regulations about final characteristics of newly constructed pavement as a new index to be controlled to be accepted (Ministerio de Obras Públicas y Urbanismo, 1989b), starting the replacement of the coefficient of the Viagrafo, a profilograph (3.4.4.2.2), and the rolling straightedge (3.4.4.2.1). After that, IRI was included as roughness index for acceptance of new and rehabilitated pavements (Ministerio de Fomento, 1997; 2001; 2002; 2004a; 2004b; 2008; 2009; 2011b; 2015). Therefore, there is a long tradition about the use of IRI as roughness index.

IRI data collection was not carried out every year, but only in some specific years. IRI data were collected in all the entire road network of the RGB in the following years: 2000, 2002 (partially, only some roads not recorded in 2000), 2004, 2007, 2011 and 2016. It is not registered the date of the data collection, but according to people in the department of the RGB, they were conducted in summer, but it is not possible to determinate the exact date (month and day), except for data in 2016. Moreover, seasonal variation in flexible pavement are due to changes in volume in the surface layers, and it can be said to be below 0,26 in dry-freeze regions and below 0,50 m/km (IRI) in wet-freeze regions (Perera and Kohn, 2002; Pérez-Acebo, 2018a).

Furthermore, data are collected with a time interval of 3 4 or 5 years, and therefore, the pavement deterioration must be higher than the possible seasonal variation. Consequently, the exact date is not a determinant data since data were always collected in summer, avoiding cold months, and due to the long time interval (between 3 and 5 years) between data collections.

IRI data are provided every 100 m of the road, specifying the exact initial and final Kilometre Post data of the stretch. In every stretch of 100 m, the value for that stretch is recorded for the right and left lane in conventional roads. For double carriageway roads, separate data are provided for each carriageway, distinguishing the ascending and the descending carriageway according to the Kilometre Post direction. In each carriageway, IRI data for the right lane (the most external one) and for the left lane (the lane on the left of the right lane) are provided. In case of three or more lanes in a carriageway, no additional data are provided, only the two lanes on the right.

Data in each 100 m stretch are reported in different ways in each data collection. As seen in Table 6. in each year the stretch has a different distance (90 in 2000, 2004 and 2011, 99 m in 2007, 100 m in 2016), although the frequency of 100 m is maintained. Sometimes, the first value does not correspond to a 100 m stretch due to adjustments for the initial point of the stretch managed by the RGB, as seen in Table 6.. This implies that the subsequent stretches are not the same in all the years. Moreover, new stretch introduced in a road, as a consequence of a new by-pass, for example, also modifies the pattern of subsequent stretches. However, this does not represent a problem, since the evolution of a specific stretch can be observed from 2000 to 2016 despite changing the initial and final point some meters.

Table 6.21. Example of IRI data in road BI-631. First five data of the road in each data collection. Source: Gestivía

Data collection year	Initial distance	Final distance	Initial KP	Final KP	IRI Right lane	IRI Left lane
2000	0	120	31+0450	31+0570	2.45	2.97
2000	130	220	31+0580	31+0670	1.95	1.94
2000	230	320	31+0680	31+0770	5.88	4.7
2000	330	440	31+0780	32+0010	3.58	2.49
2000	450	540	32+0020	32+0110	2.64	2.17
2004	0	40	31+0450	31+0490	3.83	3.03
2004	50	140	31+0500	31+0590	3.16	2.71
2004	150	240	31+0600	31+0690	2.66	2.26
2004	250	340	31+0700	31+0790	4.68	3.96
2004	350	460	31+0800	32+0030	2.81	2.42
2007	0	99	31+0450	31+0549	2.92	2.86
2007	100	199	31+0550	31+0649	2.26	2.23
2007	200	299	31+0650	31+0749	5.72	5.35
2007	300	399	31+0750	31+0849	2.65	2.53
2007	400	519	31+0850	32+0089	2.38	2.49
2011	0	40	31+0450	31+0490	5.34	6.77
2011	60	150	31+0510	31+0600	1.77	1.7
2011	160	250	31+0610	31+0700	4.58	4.47
2011	260	350	31+0710	31+0800	7.26	6.38
2011	360	470	31+0810	32+0040	3.57	3.82
2016	0	100	31+0450	31+0550	2.71	2.88
2016	100	200	31+0550	31+0650	1.54	1.4
2016	200	300	31+0650	31+0750	1.04	1.36
2016	300	400	31+0750	31+0850	2.22	2.38
2016	400	500	31+0850	32+0070	2.3	2.32

6.6.2. Skid resistance and texture data

The Regional Government of Biscay also collects skid resistance data because road safety, measured by the friction factor, is an essential factor for serviceability of the road network. Although pavements with enough structural capacity and acceptable roughness are desired, the Regional Government of Biscay controlled pavement friction due to the importance in road accidents (section 3.1.1). As explained in Chapter 3, although the International Friction Index (IFI) was developed, it has not achieved the international status that the International Roughness Index achieved, and hence, it was not been so universally adopted. The RGB collects skid resistance values by means of the Sideway-force Coefficient Routine Investigation Machine (SCRIM), developed in the United Kingdom. As commented in Chapter 3, it is a sideway force device. It provides the Side-force Coefficient, SFC. Initially, a SCRIM Reading (SR) is the output for each subsection of the tested road, usually 5, 10 or 20 m. The SR value is the average SFC value over the entire subsection length, expressed as an integer value, and multiplied by 100, after correction for speed (Eq. 3.17).

In the United Kingdom, when the truck-mounted style SCRIMs were introduced, leaving the SCRIM motorbike, a correction index (a coefficient of 0,78) was incorporated to correlate existing historical records with new measures, and a SCRIM Coefficient (SC) was created. In Spain, there is a long tradition of using the SCRIM to obtain skid resistance values (Ministerio de Obras Públicas y Urbanismo, 1982; 1986a; 1991), but it is not necessary to apply that correlation coefficient because all the measures were conducted with truck style SCRIMs. Therefore, it can be said that all the values, in Spain and in Biscay, can be regarded as SCRIM Coefficients (SC).

Initially, Spanish regulations about qualities about the new and rehabilitated roads (included in the PG-3) expressed SC values as a decimal fraction, i.e., from 0 to 1 (Ministerio de Obras Públicas y Urbanismo, 1988a; 1989b; Ministerio de Fomento, 1997). At present, SC is expressed from 0 to 100, i.e., multiplied by 100 (Ministerio de Fomento, 2001; 2004a; 2008; 2011b; 2015). In Gestivía, in order to maintain a constant criterion, SC values in Biscay, are also expressed from 0 to 100.

Similarly to IRI, the Regional Government of Biscay collected SCRIM Coefficient data some specific years in the entire road network, the same years that IRI was collected: 2000, 2002 (partially, only some roads not recorded in 2000), 2004, 2007, 2011 and 2016. For data collection in 2000, 2002, 2004 and 2007, it is not registered the date of the data collection. According to people in the Regional Government of Biscay, data were collected in summer, although it is not specifically recorded. As extensively commented in 5.4.5.5, the skid resistance values are deeply influenced by the season when data are recorded. Due to the important influence of the time of recording of friction data, the Regional Government of Biscay included in the pavement management system the exact date of the data collection. Thus, in 2011, it is registered that most of the data were obtained in February and March. Values are concordant with collection dates since values are higher. As commented n 5.4.5.5, in winter, friction values are at their maximum. Furthermore, some data were collected in summer 2011.

On the contrary, in 2016 the Regional Government of Biscay collected skid resistance data in summer, when values are at their minimum and the variation is at their least. The RGB, as the road agency in the UK, aims to know the minimum level of skid resistance available. Obviously, if obtained minimum value is over the established minimum threshold, it can be state that road provides a safe infrastructure.

SCRIM Coefficient data are provided every 20 m of the road, specifying the exact initial and final Kilometre Post data, as with IRI data. In each stretch of 20 m, skid resistance values are recorded in one of the two lanes in conventional roads. It is not indicated which lane is recorded, but it usually refers to the right lane, in the direction of ascending KPs. For double carriageway roads, separate data are provided for each carriageway, distinguishing the ascending and descending carriage according to the KP direction. In each carriageway, friction values of the right lane (the most external one) are registered, since the majority of the heavy traffic runs on this lane.

Data in each 20 m stretches are reported for different distances, although all the data are referred to a 20 m stretch of road. As exposed in Table 6., in 2000 and 2004, 10 m were recorded to represent a stretch of 20 m. In 2011 and 2016, SFC values were obtained along a distance of 19 m to measure the SCRIM coefficient over a 20 m stretch. As exception, in 2007, stretches of 17 or 18 m were tested continuously, not leaving a distance between them.

Table 6.22. Example of skid resistance data in the initial meters of road BI-633. Source: Gestivía-

Data collection year	**Initial distance**	**Final distance**	**Initial KP**	**Final KP**	**SCRIM Coefficient**	**Texture**
2000	0	20	31+0450	31+0470	38	0,80
2000	30	40	31+0480	31+0490	41	0,70
2000	50	60	31+0500	31+0510	39	0,80
2000	70	80	31+0520	31+0530	40	0,70
2000	90	100	31+0540	31+0550	40	0,90
2000	110	120	31+0560	31+0570	41	0,80
2004	0	20	31+0450	31+0470	40	1,02
2004	30	40	31+0480	31+0490	37	0,83
2004	50	60	31+0500	31+0510	39	0,95
2004	70	80	31+0520	31+0530	37	1,18
2004	90	100	31+0540	31+0550	38	1,07
2004	110	120	31+0560	31+0570	43	1,18
2007	0	18	31+0450	31+0468	66	1,28
2007	19	37	31+0469	31+0487	72	1,34
2007	38	56	31+0488	31+0506	79	0,91
2007	58	77	31+0508	31+0527	84	0,73
2007	78	97	31+0528	31+0547	81	1,02
2007	98	117	31+0548	31+0567	85	1,03
2011	0	19	31+0450	31+0469	58	1,42
2011	20	39	31+0470	31+0489	63	0,86
2011	40	59	31+0490	31+0509	60	0,80
2011	60	79	31+0510	31+0529	61	0,80
2011	80	99	31+0530	31+0549	65	0,81
2011	100	119	31+0550	31+0569	66	0,76
2016	0	19	31+0450	31+0469	44	0,63
2016	20	39	31+0470	31+0489	47	0,68
2016	40	59	31+0490	31+0509	53	0,77
2016	60	79	31+0510	31+0529	51	0,87
2016	80	99	31+0530	31+0549	48	0,86
2016	100	119	31+0550	31+0569	43	0,66

Once again, sometimes the first values in a road do not correspond to a 20 m stretch. However, in this case, it is even easier to follow the evolution of a specific stretch over the time due to the short distance, 20 m, of each one. Once more, new stretch introduced in a road modifies the pattern of subsequent stretches. Nonetheless, it does not represent a problem to follow the friction evolution of a determined stretch.

Additionally, texture data were collected simultaneously to friction data. The employed index is the Mean Profile Depth (MPD), described in 3.3.9.3.2.2, obtained by a laser-based device and it is expressed in mm. The values refer to the same distance as for SCRIM coefficient, as shown in Table 6..

6.6.3. Structural capacity

The Regional Government of Biscay collected structural capacity data of the road network managed by means of the deflectograph and the Dynatest.

Data with the Deflectograph were collected every 5 or 10 m depending on the road in the following years: 2000, 2002, 2004, 2007, 2011, 2012 and 2016, although in some years the entire road network was not collected. The maximum deflection for the right and left lane was recorded in conventional roads and in the two most external lanes of each carriageway in double carriageway roads.

Dynatest is one device that can be categorized as Falling Weight Deflectometer (FWD) (section 3.5.3.6). Data with Dynatest were collected with variable distance between tested points, ranging from 20 to 50. Data collection years were the same as with Deflectograph (2000, 2002, 2004, 2007, 2011, 2012, and 2016) but not all the years the entire road network was tested. In conventional roads a unique load is applied, not differentiating the lanes. In double carriageway roads, loads are applied in a unique point in each carriageway. Apart from the maximum deflection, values registered in the adjacent geophones are also recorded in Gestivia.

Additionally, roads are divided in homogeneous segments according to the characteristic deflection and calculation deflection, which are calculated following the instruction of Ministerio de Fomento (2003a).

6.6.4. Pavement distresses

The Regional Government of Biscay collected pavement distresses in its road network, including cracking, potholes and rutting.

Cracking data were collected in 2011 and 2016 and include various indexes. On one hand, in 2011, a total cracking value is presented; meaning the cracked area in percentage, and it is obtained as an average of left and right lanes, which are obtained from crocodile cracking values, longitudinal cracking values and transverse cracking values and pothole values. All these partial values are also expressed as percentages of the considered area. On the other hand, in 2016 data are presented as a Total Cracking Index, expressed in m/m, i.e. from 0 to 1, composed by a Longitudinal Cracking Index, a Transverse Cracking Index and a Cracking related to Rutting Index. These values are also expressed as the relation of cracked area and the considered area in m/m. Rutting data were collected in 2004, 2007 and 2016 every 20 m.

6.7. Conclusions

The chapter described the Pavement Management System in use in Biscay, by the Regional Government of Biscay. The historical reasons for having the total competence over the entire road network (except a motorway under concession by the Spanish Government) were explained. The laws that give those competences were mentioned.

Then, the characteristics of the pavement management systems were commented, comparing them with the necessary information that a PMS must have, as described in Chapter 2. Geometric data follows the general ideas of the main PMS guides. The traffic data does not have information about the Equivalent Single Axle Load (ESAL) and the PMS only deploys information about heavy vehicle traffic, defined as vehicles with total weight over 3.500 kg. With regard to environmental data, very few data are introduced in Gestivía, due to the small area of Biscay and is uniform climate.

Regarding the pavement structure information and project history, it was shown that many of the ideas used in the PMS come from the Guide for updating the pavement database of the State Road Network (Ministerio de Fomento, 2011a), but adapted to the local circumstances. The main difference lies in the way of introducing the pavement information. In the case of the PMS of the RGB, it is based on project information. With this information, the software provides two important files, the Pavement Structure file and the Surface Layer file, where data about the pavement section and the surface layer can be found, respectively.

With regard to pavement condition data, the RGB collected information about roughness, skid resistance, structural capacity and pavement distresses. Roughness is expressed by means of the International Roughness Index (IRI) every 100 m and skid resistance is collected by means of SCRIM, providing a SCRIM Coefficient (SC) every 20 m. Data collections for IRI and SC were conducted in the entire road network in 2000, 2004, 2007, 2011 and 2016.

Chapter 7. Selection of predicted variables and modelling type

7.1. Introduction

The aim of this chapter is to expose the selection of the modelling type for the pavement performance models for the roads of the province of Biscay, based on the available data in the Pavement Management System of Biscay, called State Agenda (Agenda de Estado).

Firstly, as exposed in section 4.3, it must be selected the indices that are going to be forecasted by the deterioration models. From the possible deterioration indices, those that are going to be predicted are chosen.

Secondly, a complete explanation for the selection of the model type is provided. This is key moment in the development of any pavement model since it conditions the rest of the process. Hence, the reasons for discarding each of the possible model types are presented and the selection of the modelling type is based on well-reasoned arguments.

Finally, a brief exposition of the main characteristics of the selected model type is included. It is necessary to understand the problems that can appear when developing that kind of model and the possible solution to overcome the arisen problems. Selected factors or predictors to be used in each modelling are described in Chapter 8 and 9 when describing the developed models for each index.

7.2. Selection of the predicted variables

In Chapter 4 it was exposed that the first step in any pavement modelling is the selection of what the model is going to forecast. Possible variables may be individual pavement condition indices, from the wide variety exposed in Chapter 3, composite indices, which are made up of some individual indices or even the distress severity and extend of a specific distress.

In Chapter 6 the data collected for the Pavement Management System of the Regional Government of Biscay are exposed. Some of them, such rutting or cracking have been only collected in some data collection campaigns, and hence, it is not possible to follow their performance over their life cycle. However, usual indices for roughness, skid resistance and structural capacity are registered in the data collection of 2000, 2004, 2007 and 2011 and 2016. Consequently, pavements carried out during the last decades can be employed for deterioration modelling.

From the available indices, the following ones have been selected for prediction:

- The **International Roughness Index (IRI)** for longitudinal roughness. It evaluates the comfort that road users perceive. It has proved to have a good correlation with pavement distresses and hence, it is employed as the basis for pavement performance prediction (Meegoda and Gao, 2014; Park *et al.*, 2007; Sandra and Sarkar, 2013). Due to its stability with time and its transportability, it has become the most widely used parameter.

- **SCRIM coefficient and texture values** for skid resistance. Road safety is also a vital feature of any road and highway administrations must provide a road with enough friction to minimize road crashes, especially in wet roads.

7.3. Selection of pavement prediction models

In this section, the different prediction model types are exposed and commented. First of all, the level for modelling the deterioration must be established: project level or network level modelling. Then, advantages and disadvantages for each model type are discussed. Principally, the variety of models is presented for roughness modelling, since many models have been proposed (section 5.3). For skid resistance few models have been suggested, and all of them are deterministic. After the description of the models, and according the data available in the pavement management system of the Regional Government of Biscay, the models providing more advantages and fewer disadvantages is selected.

7.3.1. Project level versus network level modelling

As commented in section 2.4, there are two fundamental modelling levels in pavement management systems: project level and network level modelling.

- **Project level modelling** aims to describe the actual deterioration and its likely variability of a limited length road section. It depends on a relatively well-know future traffic and detailed pavement data and may need in situ measurements, such as, deflection by means of FWD or core drilling rigs. Based on this detailed information, it is possible to select a repair or rehabilitation technique to achieve the required functionality level.
- **Network level modelling** is used to predict the average deterioration for an extensive, linked road network. It is based upon future traffic that may vary, depends on pavement data that are known from construction records and present condition measurements that are usually conducted at normal traffic speed, such IRI, skid resistance and deflection measurements. Based on this information, it can be decided the standardized rehabilitation or maintenance work options, which can be graduated as a function of the required network condition level and available funds.

Hence, at project level the real performance is determined for a specific road section. This is not the aim of this thesis, which aims to develop deterioration models for the entire road network. There is information for the roads in Biscay, but not with the required detail for producing project level models. Moreover, a project level model will be only employed for one road or even only for a road section. However, a general forecast of future condition of all the roads managed by the Regional Government of Biscay is aimed to be obtained, which would contribute to a better budget allocation.

Therefore, average parameters for all the roads in Biscay are aimed to be calculated as a function of influencing factors. Consequently, network level models are selected.

7.3.2. Subjective models

Subjective models may be deployed in deterioration modelling. They use experts' knowledge and expertise to create models, which can be any of the models presented in section 5.3. As commented in 5.3.6, subjective models are specially interesting and useful when the road agency lacks of complete historical data (Osorio-Lird *et al.*, 2017).

Before the establishment of the pavement management system in Biscay, road experts in the RGB predicted subjectively the pavement evolution of the road network of Biscay. This technique can provide enough good results as long as experts had a real good knowledge and expertise, as happened in the RGB. Engineers with more than 20 years managing and supervising road construction have an excellent background about pavement performance of Biscayan network.

Nonetheless, one of the aims of this thesis is to provide a helpful, not biased tool to predict the deterioration evolution of pavement to any road engineer or worker of any road administration. Furthermore, as presented in Chapter 6, a huge quantity of information is available in Gestivía, with various data collection campaigns and structure, traffic and construction, rehabilitation and maintenance history of the entire road network during the last 3 decades.

Consequently, subjective models are discarded as possible deterioration models for the road network of Biscay. More accurate performance model are aimed to be developed, indicating the factors that really affect the selected indices, which can be useful to predict future condition if these factors are known or predicted.

7.3.3. Artificial Neural Networks (ANN)

These computational models are able to learn from examples to simulate the relationship between inputs and outputs. The process does not need detailed relationship between variables and can be employed when several variables influence the result (Osorio-Lird *et al.*, 2017; Simpson *et al.*, 1994), providing solution to some types of problems that can be difficult to solve by means of traditional numerical and statistical method. After a training period, new data can be introduced for evaluation. Nevertheless, modelling is complex, requires a large amount of experimental data, and the main limitation is that neural networks act as a "***black box***", and it is not possible to easily extract the path followed to explain a solution (Flintsch and Chen, 2004).

The quantity of data in the PMS of the RGB, especially for roughness prediction, could be enough to serve an Artificial Neural Network. A portion of them could be employed to the training stage, and after it, the rest of data could be checked to establish the efficiency of the model.

Nonetheless, the fact that the ANN acts as a "*black box*" is thought to be a limit to its employment and hence, the Artificial Neural Network modelling is discarded. An accurate enough model may be developed by means of ANNs, helping road engineers in the RGB to predict the future condition of the roads, even for IRI prediction with a high correlation (Abdelaziz *et al.*, 2018). However, this type of model does not show the correlations established between the different factors introduced and the obtained results. It is not possible to know the relative importance of each factor, and obtained knowledge could not be transferred to other road agency with similar conditions. The thesis aims to develop a transferable model, which could be employed in similar circumstances in any place.

7.3.4. Probabilistic models

The two main probabilistic models, the Bayesian models and the Markov chains, are discussed individually.

7.3.4.1. Bayesian models

The Bayesian models offer the objectivity in expert panel evaluations and delivers reliable results for a small database. They combine prior knowledge of certain event probabilities with observed data (likelihood) in order to produce an adjusted expression of the event probabilistic distribution (known as posterior).

The advantage of Bayesian models is the capability to incorporate uncertainty, as well as incorporating expert opinions to supplement historical data when historical data are not available (Amador-Jiménez and Mrawira, 2011a; 2011b). This model relies on the prior probability distribution chosen for the analysis.

On the other hand, the main disadvantage of this model type is that Bayesian inference includes the evaluation of different posteriors, which is a comprehensive analysis. Markov chain and Monte Carlos methods have been recognised to facilitate this analysis (Lunn *et al*., 2000).

Once again, the pavement management system of RGB does not require a performance model which handles with the problem of scarce data. Thanks to the data recompilation conducted by the engineers of the UTE Agenda de Estado (the venture company created between the three engineering companies), a great quantity of information is available. Therefore, it is not necessary to include experts' opinion to complete historical data since there is a big amount of data accessible. Moreover, the bias that experts could introduce is aimed to be avoided. Consequently, a Bayesian model as a modelling technique for pavement deterioration modelling is rejected.

7.3.4.2. Markov chains models

As exposed extensively in Chapters 4 and 5, Markov chain models consider the probabilistic deterioration of pavements over time by means of a transition probability matrix (TPM) to forecast future pavement condition based on its current stage. If combined with Monte Carlo simulations, the model reflects the stochastic transition of pavement condition over time.

Literature shows that it is possible to develop these models without large historical database (Pérez-Acebo *et al*., 2017b; 2018a) and have successfully been applied in other researches (Black *et al*., 2005; Chamorro and Tighe, 2011, Hassan *et al*., 2015; Silva *et al*., 2000; Yang *et al*., 2006). According to Ortiz-García *et al*. (2006), the limitations of Markov chain process to forecast pavement deterioration are: is discrete on time, should have finite state and should satisfy the memory-less property, (Markov property), which states that the future condition of the pavement is only dependent on its present condition and not on its past condition. These limitations could be addressed to use Markov chain to predict pavement deterioration considering (Ortiz-García *et al*., 2006):

- Pavement deterioration is continuous in time but the condition of the road network is analysed at specific points in time.
- The state space is infinite but is defined as a finite number of fixed bands of the indicator considered
- It is assumed that the pavement deterioration satisfies the Markov property (Kerali and Snaith, 1992) and the definition of scenarios considered to define the TPM is a key action to crease the uncertainty of assuming this property.

In this kind of modelling, collection data from at least two different stages is needed for all the condition ranges considered in the TPM. One limitation of this model type is that it is based on the TPMs. Hence, it is necessary to develop a TPM for each combination of factors that affect the pavement performance, such material, structure, traffic, climate, etc. Researchers have usually developed a short series of TPMs for all the possible conditions, gathering them in families, for example, by pavement type (Osorio-Lird *et al*., 2017; Pérez-Acebo *et al*. 2017b; 2018a) by road hierarchy, which includes traffic volume and as a consequence of it, pavement structure (Hassan *et al*., 2015; Osorio-Lird *et al*., 2017; Pérez-Acebo *et al*., 2017b; 2018a; Soncim *et al*., 2017), climate (Chamorro, 2012; Chamorro and Tighe, 2015; Soncim *et al*., 2017). Consequently, if several different groups are proposed the quantity of TPMs to be calculated is enormous. For example, Soncim *et al*. (2017) combined the 11 transition matrices due to the different combinations of traffic with the established 4 climates (sub-humid, humid, arid and dry), resulting in a total of 44 values. However, in this case, experts' opinion was employed to develop them, due to the lack of historical data.

If this methodology is transferred to Biscay, 8 traffic levels could be considered, according to the traffic levels established in the Spanish regulations (Ministerio de Fomento, 2003b). A new division would imply the pavement structure, distinguishing between flexible and semi-rigid pavement (rigid pavement, i.e., Portland Cement Concrete pavements, cannot be found in Biscay). Among semi-rigid pavements, different subbases are deployed, like cement treated materials (mainly soil-cement and gravel-cement and even a combination of both) and even gravel and slug. Finally, the different surface layers could be also a factor for dividing the pavement structure (Asphalt concrete, porous asphalt, discontinuous mixes and slurries). Therefore, the amount of matrices that should be calculated will be huge, and if considering non-homogeneous TPM, even greater. Obviously, the quantity of TPMs can be reduced by gathering some types. This would lead to a simplification. In this case, the enormous information available in Gestivía would not be profited. These simplifications are generally carried out when few data are available or inexistent apart from general classification of the roads (Pérez-Acebo *et al*., 2017b; 2018a). Moreover, these simplifications lead to not so accurate models, which cannot be reliable in some cases.

Consequently, Markov chain modelling is discarded. The inherent variability of the pavement deterioration is lost if this kind of models are not applied. Nonetheless, it has been preferred to develop accurate models which can change their values, even if minimally, to take account all the possible variations of the factors.

7.3.5. Deterministic models

Among deterministic models, statistical regression analysis is the most widely model used to develop deterioration models (Amador-Jiménez and Mrawira, 2011b). Apart from linear relations, regression models can develop non-linear relations and are simple to understand and apply (Hassan *et al.*, 2015). Their efficiency has been proven with large experimental and historical data (Arambula *et al.*, 2011; Dong *et al.*, 2015; Meegoda and Gao, 2014).

One disadvantage that it is usually mentioned about these models is that they cannot be extrapolated beyond the limits of the experimental data and produce a single value of the dependent variable (Amador-Jimenez and Mrawira, 2011a). Hence, calibration is generally recommended when transferring a deterministic model from a site to another.

The accuracy of performance models using regression analysis is dependent on the availability of historical information, including condition over time and pavement age (TAC, 2013). Deterministic models have a long tradition in pavement performance modelling, especially for roughness evolution. Some of the most important and widely used models belong to this type, such the ones proposed by the World Bank (which are necessary to employ for developing countries to get budgets for road construction) (Paterson, 1987; Kerali *et al.*, 2004) or the ones proposed by the AASHTO, the mechanistic models (AASHTO, 1993) or, at present, the recent mechanistic-empirical models (AASHTO, 2008, 2010, 2015). As exposed in Chapter 5, many road agencies have developed several deterministic deterioration models (European Commission, 1997; European Communities, 1999; Busch *et al.*, 2010), especially related to roughness and pavement distress progression. Furthermore, individual researches and local agencies have also produced a high quantity of deterministic models. It must be noted that the Long-Term Pavement Performance (LTPP) database is facilitating the employment of this modelling type, although other types are used, too.

Due to the great amount of information available in the PMS of the RGB, **it is decided to select the deterministic models** for prediction future roughness and skid resistance condition in the road network in Biscay. The historical background stored in Gestivía allows observing the evolution of different sections. Moreover, due to the introduction of Initial Pavement files, where all the pavement structure and the age are known, facilitates observing the progression of different combination of pavement layers, both bituminous and subbase layers, as a function of age or traffic. Furthermore, the fact that the subsequent rehabilitation and maintenance works are introduced facilitates the production of rehabilitated and maintained models, making possible the assessment, from a technical and economical point of view, of each repairing activity individually. Briefly, the impressive quantity of data regarding pavement structure, surface layers, traffic volume data, age of construction and rehabilitation has lead to select a deterministic model, aiming to take advantage of all the variations that are created when varying some of the factors.

7.4. Mechanistic, empirical or mechanistic-empirical modelling

Next step when a deterministic model is selected for pavement performance modelling is to choose between a mechanistic, an empirical or a mechanistic-empirical model, mainly with regard to roughness progression.

As commented in section 4.4.5, mechanistic models are developed analyzing some primary response (behaviour) parameters such as stress, strain or deflection (Haas *et al.*, 1994). They are based on the stress and strains in the pavement obtained from fundamental theories of behaviour. Generally, this kind of models are developed for project-level evaluations, after extensive laboratory testing and accelerated pavement testing (Roberts *et al.*, 2003; Molenaar, 2003). Mechanistic models have been used in the past for pavement design, especially in North America (AASHTO, 1993) and in the Soviet Union and territories under its influence (Pérez-Acebo *et al*., 2017b; 2018a). However, mechanistic models are recommended to be calibrated in situ, with real data, as they may not adequately take into consideration material variability, weather, traffic loads and their combination (Uddin, 2006). Therefore, appropriate performance measures should be identified and collected so as to develop deterioration models. A good example is the Long-Term Pavement Performance (LTPP) database, where more than 2500 test sections are located on North American existing and newly constructed highways and provides several data to researchers. Models combining mechanistic principles of deterioration and field measurements are denoted as mechanistic-empirical (ME) models, and they are the base of the new pavement design guide of North America, the Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008).

On the contrary, empirical models are used for network level analysis. They related a dependent variable, observed or measured structural or functional parameter, to one or more independent variables, acting as predictors. These models are generally based on statistical analysis of deterioration trends observed in a specific area. As regression analyses are usually employed to best fit the variables, they are also called regression models (Haas *et al*., 1994).

As exposed in section 7.3.1, the aim of this thesis is to develop deterioration models that can be applied to all the pavement section that can be found in the province of Biscay, e.g., it is aimed to develop a network-level model. Moreover, mechanistic models are very data intensive and rely on parameters that are very difficult to obtain in situ. Furthermore, no laboratory tests have been carried out in this thesis, not allowing producing these models.

Additionally, mechanistic-empirical models, especially the ones provided in the MEPDG (AASHTO, 2008) calculate pavement response (stresses, strains and deflections) and employ those response to compute incremental damage over time, and the procedure relates the cumulative damage to pavement distresses observed in the field. As presented in equations 5.42 to 5.45, models proposed in the MEPDG for IRI prognosis include parameters like the area of fatigue cracking, the length of transverse cracking, the average rut depth and a site factor which, apart from weather conditions of the site, needs the percent passing the 0,02 and 0,075 mm sieves of the subgrade. The cracking data need values such as the tensile strain at critical locations and calculated by the structural response model, the dynamic modulus of the HMA measured in compression, the effective asphalt content by volume, the percent air voids in the HMA mixture, etc. For rut depth calculation, it is needed the plastic vertical strain under specific conditions for the total number of trucks within that condition., requiring the rate or accumulation of plastic deformation, measured in the laboratory using repeated load permanent deformation triaxial test for both HMA mixtures and unbound materials. Therefore, all the ME models needs data from laboratory tests and specific information from each project data, which are not available in the PMS of the Biscay. Consequently, mechanistic-empirical models cannot be developed for IRI prediction.

As a result, empirical models are selected as the appropriate deterministic models for the road network of Biscay. Empirical models are said to be the most widely employed models due to its simplicity and capacity to incorporate all measurable predictors, with a great literature background, especially for roughness prediction (Haas *et al.*, 1994; Giummarra *et al.*, 2007; Mubaraki, 2010; Alaswadko *et al.*, 2017). Regression analysis, including linear and non-linear methodologies, is chosen for producing the deterioration models for roughness evaluation. For skid resistance, all proposed models are empirical, based on in situ data (Szatkowski and Hosking, 1972; Roe and Hartshorne, 1998; European Commission, 1997; Transit New Zealand, 2002a; NZTA, 2013a) or laboratory tests (Rezaei *et al.*, 2009; Rezaei and Masad, 2013; Khasawneh, 2017). Consequently, similar empirical models are decided to be developed based on in field data, registered in Gestivía.

As a conclusion, after revising all the performance models for pavement in the literature and according to the available data of the pavement management system of Biscay (Agenda de Estado), it is decided to develop a deterministic empirical model for predicting the International Roughness Index (IRI) and the SCRIM Coefficient (SC) of the pavement network of Biscay. The main reasons for this decision are summarized below:

- Network-level models were selected as models for all the pavement networks of Biscay were aimed to be developed. They are wanted to be applied for all the existing pavement sections.
- The aim of avoiding biased models, mainly introduced by experts' opinion in subjective models.
- The huge historical data incorporated to the PMS, which allows analyzing the complete life-cycle of the pavement sections, makes avoiding other models which are employed with scarce historical data, like probability models. It eliminates the probabilistic nature of pavement performance, but it has been thought to be unnecessary.
- The aim is to develope models that can be employed by anyone in Biscay or elsewhere. Artificial Neural Networks were discarded because they act as a "*black box*", not providing information about the influence of each parameter or predictor in the final result. Although better coefficient of determination (R^2) can be achieved by ANN models, it was preferred to know, understand and show the exact relationship between the predicting variables and the predicted variable.
- The absence of laboratory tests has led to discard mechanistic and mechanistic-empirical models within deterministic models. However, information about influencing factors that these models provide is considered when developing empirical models.

7.5. Regression analysis methodology

As the selected performance models for pavements in Biscay is a deterministic empirical model by means of regression analysis, it has been considered useful to include some exposition of this methodology, the main problems that can arise and the usual solutions that are applied to overcome the deficiencies detected.

7.5.1. General overview about data analysis techniques

Statistical predicting techniques can be considered as a sub-set of the more general statistical techniques of data analysis, which include both the predicting techniques focused on modelling and classifying ad hoc and the descriptive techniques focused on the post hoc classification, among other techniques..

Predicting techniques, which can be used in econometrics, medicine, biology, epidemiology and engineering, indicate the model for the data according to a previous theoretical knowledge. Once the theoretical model has been identified for the data, the model is estimated and then is checked before accepting as valid. In that moment it is possible to use the model to predict (Pérez López, 2014).

Among predicting techniques it can be included all the regression types, temporal series, variance and covariance analysis, discriminating models, decision trees and neuronal networks. The last three techniques can be also classified as classification techniques for obtaining performance profiles or classes, allowing a model that classify a new data (Pérez López, 2014).

On the other hand, in the descriptive techniques, there is no a predetermined role for the variables. The existence of dependent and independent variables is not assumed as well as a model for the data. Models are created automatically from pattern finding. In this group, the following techniques can be found: clustering and segmentation techniques, association and dependence techniques, data explanatory techniques and dimension reducing techniques (factorial, main components, correspondences, etc.). Fig. 7.1 shows the general classification of data analysis techniques

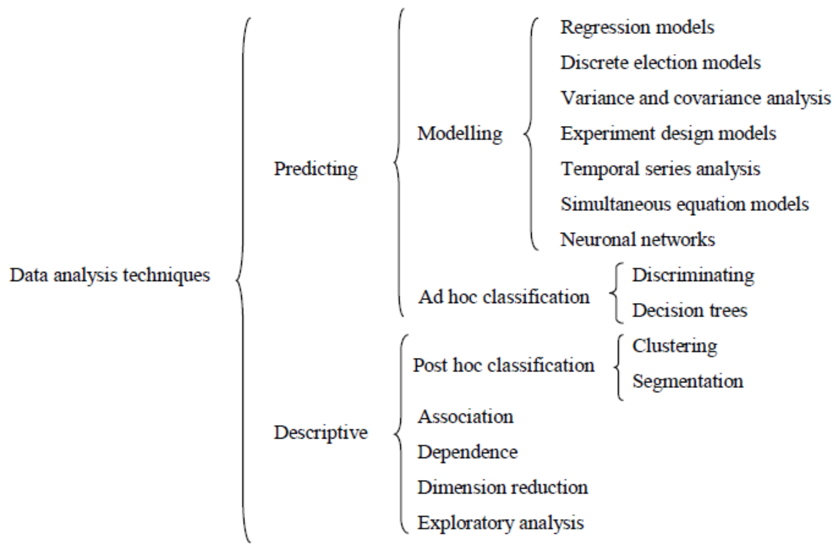


Fig. 7.1. Classification of data analysis techniques. Adapted from Pérez López, 2014.

As explained, if there is dependence between the explained variables and its corresponding explicative variables, which can be explained by a model, it is a predictive technique or an explicative method (Pérez López, 2014). These analysis techniques can be classified as a function of the quantitative or qualitative nature of the independent and dependent variables as shown in Fig. 7.2.

The **multiple regression analysis** is a statistical technique employed for analyzing the relationship between a quantitative dependent variable and various independent variables, which are also quantitative (Hair *et al*., 1999; Pérez López, 2004). The aim of the multiple regression analysis is to use the independent variables, whose values are known, to predict the dependent variable (response). The functional expression of the multiple regression is:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.1]$$

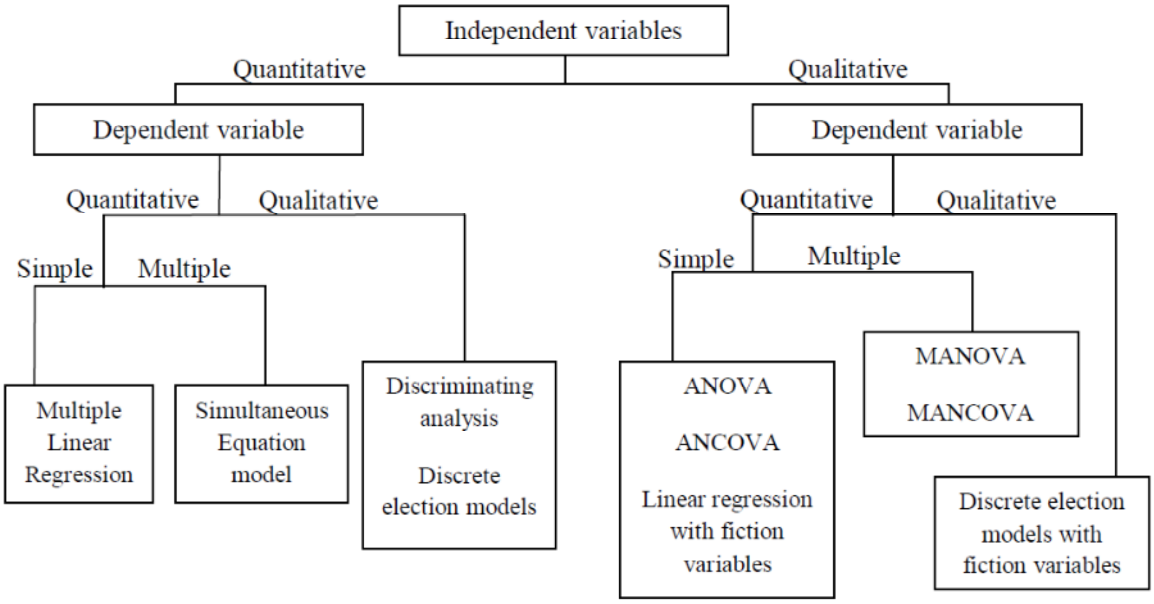


Fig. 7.2. Classification of predictive techniques. Adapted from Pérez López, 2014.

Where initially, both the dependent variable y and the independent variable x_i are quantitative. Additionally, the multiple regression allow working with independent variable that are qualitative if fiction variables are used (regression models with fiction variables), after transforming to quantitative variables (Pérez López 2004).

The **canonical analysis** (canonical correlation) is a generalization of the multiple regression as it allows analyzing the relationship between multiple quantitative dependent variables and various quantitative independent variables. The functional expression of the canonical correlation is:

$$G(y_1, y_2, \cdots, y_n) = F(x_1, x_2, \cdots, x_n) \quad [7.281]$$

This technique can be employed for dependent or independent variables (or both) which are qualitative.

The **simultaneous equation models** are a statistical technique for analyzing the relationship between multiple quantitative dependent variables and various quantitative independent variables. The functional expression is:

$$\vec{G}(y_1, y_2, \cdots, y_n) = \vec{F}(x_1, x_2, \cdots, x_n) \quad [7.3]$$

This model can be considered as a generalization of the multiple regression model to the case of various dependent variables.

The **discriminating analysis** is a statistical technique for analyzing the relationship between a qualitative dependent variable (response) and various quantitative independent variables (predictors). It aims to use the known values of the independent variables to predict to which categories of the dependent variables corresponds. The function expression of the discriminating analysis is Eq. 7.4:

$$y = F(x_1, x_2, \cdots, x_n) \quad [7.4]$$

Where the dependent variable y is qualitative and the independent variables are quantitative. It is a particular case of the multiple regression analysis.

The **discrete election models** have the same nature as the discriminating models but in this case what is predicted is the probability of pertinence to a category (class) for known values of the independent variables. Hence, the discrete election models predict directly the probability of happening of a success which is defined by the independent variables. As the probability values are between 0 and 1, the predictions must fall in the range of 0 and 1. The general model that fulfils this requirement is a particular case of the multiple regression model that is called the probability lineal model and has the following functional expression:

$$P_i = F(x_i, \beta) + u_i \qquad [7.5]$$

It is observed that if F is the distribution function of an random variable, P varies between 0 and 1. In the particular case of being F the logit function, the model is a Logit model, whose function expression is:

$$P_i = F(x_i, \beta) + u_i = \frac{e^{x_i \beta}}{1 + e^{x_i \beta}} + u_i \qquad [7.6]$$

The **Analysis Of Variance (ANOVA)** is a statistical technique employed for analysing the relationship between a quantitative dependent variable (response) and various qualitative independent variables (predictors). The aim is to determine if the various samples come from populations with same mean. The qualitative values of the independent variables will determine a series of groups in the dependent variable. Thus, the ANOVA model measures the statistical significance of the differences between the means of the groups determined in the dependent variable by the values of the independent variables. The functional expression is the analysis of variance is:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.7]$$

Where the dependent variable y is quantitative and the independent variables are not quantitative.

The **Analysis of Covariance (ANCOVA)** is a statistical technique employed for analysing the relationship between a quantitative dependent variable and various independent variables, which part of them are qualitative and the rest quantitative. The functional expression is:

$$y = F(x_1, x_2, \cdots, x_n) \qquad [7.8]$$

Where the dependent variable y is quantitative and some of the independent variables are qualitative and the rest are quantitative.

The **Multiple Analysis of Variance (MANOVA)** is a statistical technique employed for analysing the relationship between various quantitative dependent variables and various qualitative independent variables. The aim is to check if the qualitative values of the independent variables will determine the equality of the mean vectors of a series of groups determined by them in the dependent variables. Thus, the MANOVA model measures the statistical significance of the differences between the vectors of the means of the groups determined in the dependent variables by the values of independent variables. The functional expressions is:

$$G(y_1, y_2, \cdots, y_n) = F(x_1, x_2, \cdots, x_n) \quad [7.9]$$

Where the dependent variables are quantitative and the independent variables are qualitative.

The **Multiple Analysis of Covariance (MANCOVA)** is a statistical technique employed for analysing the relationship between quantitative dependent variables and independent variables, which part of them are qualitative and the rest are quantitative. The functional expression of the MANCOVA is:

$$G(y_1, y_2, \cdots, y_n) = F(x_1, x_2, \cdots, x_n) \quad [7.10]$$

Where the dependent variables are quantitative and some of the independent variables are qualitative and the rest are quantitative.

A **decision tree** analyses the relationship between a qualitative dependent variable and various qualitative independent variables. Its expression is:

$$y = F(x_1, x_2, \cdots, x_n) \quad [7.11]$$

As the logit regression and the discriminating analysis, the aim is to predict the category of the qualitative dependent variable in which the individuals are classified according the values of the qualitative independent variables. Usually, in the decision trees quantitative variables are also used grouping their values in groups (normally in five at maximum).

The multiple regression allows working with qualitative independent variables if fiction variables are used for their transformation in quantitative. For each qualitative variable a numerical value is assigned.

The multiple regression models with fiction variables is similar to the multiple regression analysis with the difference that the independent variables can be qualitative too. Therefore, it is a technique employed for the analysing the relationship between a quantitative dependent variable and various independent variables, which qualitative, quantitative or both. The aim of the regression analysis is to use independent variables, whose values are known, to predict the unique response variable (dependent) selected by the researcher. The functional expression is:

$$y = F(x_1, x_2, \cdots, x_n) \quad [7.12]$$

Table 7.1 summarizes the predictive techniques according the nature of the dependent and independent variables (quantitative or qualitative).

As seen, the most appropriate models for predicting the IRI and SCRIM coefficient in the road network of Biscay are the multiple regression linear models (or linear models if the only depend on one variable) and the ANCOVA models. IRI and SCRIM coefficients are quantitative variables. The possible independent variables will be quantitative variables (age, layer thickness, traffic volumes, as seen in other models) or a combination of quantitative and qualitative variables. Among the qualitative variables, the types of pavement section (flexible, semi-rigid) and the different materials employed in each layer could be introduced. Further explanation about the selected independent variables is provided in Chapter 8 and 9. In next subsection, further description of the multiple linear regression models is included.

Table 7.1. Nature of the dependent and independent variables of the predictive techniques (Pérez López, 2004)

Technique	Dependent variables	Independent variables
ANOVA and MANOVA	Quantitative	Qualitative
ANCOVA and MANCOVA	Quantitative	Quantitative and qualitative
Multiple regression	Quantitative	Quantitative
Multiple regression with fiction variables	Quantitative	Quantitative and qualitative
Canonical correlation	Quantitative and qualitative	Quantitative and qualitative
Discrete election	Qualitative	Quantitative
Discrete election with fiction variables	Qualitative	Quantitative and qualitative

7.5.2. Multiple lineal regression

As commented previously, the multiple linear regression aims to analysis a model that wants to explain the performance of variable (dependent variable, response), which is denoted as *Y*, using the information provided by the values of a series of explanatory variables (predictor, regressor variable), denoted as X_1, X_2,..., X_k. The linear model has the following shape (Sierra Bravo, 1994; Montgomery *et al.*, 2012; Pérez López, 2014):

$$Y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \ldots + \beta_K X_K + u \qquad [7.13]$$

The regression coefficients (parameters) β_1, β_2,..., β_k indicate the magnitude of the effect that the explanatory variables (independent) X_1, X_2, ..., X_K have over the response variable (dependent) *Y*. The coefficient β_0 is called the intercept. The component *u* is denoted as the model error. If *T* observations are available for each of the independent and dependent variables, the model can be expressed as:

$$Y_t = \beta_0 + \beta_1 X_{1t} + \beta_2 X_{2t} + \ldots + \beta_k X_{kt} + u_t \quad t = 1,2,3,\ldots,T \qquad [7.14]$$

The presence (not necessarily) of an independent component in the model can be interpreted as a first X_0 variable, whose value is always 1.

The main problem is that, supposing the relationship between the variable *Y* and the series of variables X_1, X_2,..., X_k is as described in Eq. 7.13 and that a series of *T* observations for each of the independent and dependent variables is available, how can be addressed the numerical values of the parameters β_1, β_2,..., β_k based on the sample information. These values are denoted as parameter estimators. The linear model is formulated under the following hypothesis (Pérez López, 2014):

- The variables X_1, X_2, ..., X_K are deterministic, i.e. they are not random variables, since their value is a constant value from a sample.
- The variable *u* (the error component) is a random variable with mean response is null and the covariance matrix is constant and diagonal (scalar matrix). In other words, for every *t*, the variable u_t has a mean response zero and the variance σ^2 does not dependent on t, and additionally, $Cov(u_i,u_j)=$

0 for every i and every j different between them. The fact that the variance of u_t is constant for every t (that is not dependent on t), is called the hypothesis of homoscedasticity. The fact that $Cov(u_i,u_j)=0$ for every i different from j is called the hypothesis of no autocorrelation.

- The variable Y is random, as it depends on the random variable u.
- It is assumed that there are not specification errors, i.e. it is supposed that all the variables X that are relevant for the explanation of the variable Y are included in the linear model
- The variables X_1, X_2, ..., X_K are linearly independent, i.e, there is no exact linear relationship between the,. This hypothesis is called independence hypothesis and when it is not fulfilled it is said that the model has multicollinearity.
- Sometimes the hypothesis of normality of the residuals is also considered, which implies that the variables u_t are normally distributed for every t.

Supposing that the model is wanted to adjust to the expression of Eq. 7.13 and a series of T observations for each independent and dependent variable is available, which allows expressing the model following the Eq. 7.14, the method of least squares (or ordinary least squares, OLS) can be used to estimate the regression coefficients. This method considers that the best function that best fits the data is the one that minimises the variance of the error u (Montgomery *et al*., 2012; Pérez López, 2016). The least square function, S, is

$$S(\beta_0,\beta_1,\ldots,\beta_k)=\sum_{t=1}^{T}u_t^2=\sum_{t=1}^{T}\left(y_t-(\beta_0+\beta_1X_{1t}+\beta_2X_{2t}+\ldots+\beta_kX_{kt})\right)^2 \qquad [7.15]$$

Function S must be minimised with respect to β_1, β_2,..., β_k. The least squares estimators of β_1, β_2,..., β_k indicate must satisfy:

$$\frac{\partial S}{\partial\beta_0}=2\sum_{t=1}^{T}\left(y_t-(\beta_0+\beta_1X_{1t}+\beta_2X_{2t}+\ldots+\beta_kX_{kt})\right)(-1)=0 \qquad [7.16]$$

And for regression coefficients β_1 to β_k:

$$\frac{\partial S}{\partial\beta_j}=2\sum_{t=1}^{T}\left(y_t-(\beta_0+\beta_1X_{1t}+\beta_2X_{2t}+\ldots+\beta_kX_{kt})\right)(-x_{jt})=0,\quad j=1,2,\ldots,k \qquad [7.17]$$

Simplifying Eq. 7.16 and 7.17 the least squares normal equations are obtained:

$$\begin{aligned}
\sum_{t=1}^{T}y_t &= T\beta_0+\beta_1\sum_{t=1}^{T}x_{1t}+\ldots+\beta_k\sum_{t=1}^{T}x_{kt}\\
\sum_{t=1}^{T}y_tx_{1t} &= \beta_0\sum_{t=1}^{T}x_{1t}+\beta_1\sum_{t=1}^{T}x_{1t}^2+\ldots+\beta_k\sum_{t=1}^{T}x_{1t}x_{kt}\\
&\vdots\\
\sum_{t=1}^{T}y_tx_{kt} &= \beta_0\sum_{t=1}^{T}x_{kt}+\beta_1\sum_{t=1}^{T}x_{kt}x_{1t}+\ldots+\beta_k\sum_{t=1}^{T}x_{xt}^2
\end{aligned} \qquad [7.18]$$

These equations of [7.18] compose a system denoted as system of least squares normal equations, which can be solved for β_1, β_2,..., β_k by any adequate model to solve linear equations. Note that there are p = k + 1 normal equations, one for each of the unknown regression coefficients. Thus, the model is estimated. It is more convenient to deal with multiple regression models is they are expressed in matrix notations. This allows a very compact display of the model, data, and results. The model is given by Eq. 7.19 (Groβ, 2003; Bingham and Fry, 2010; Olive, 2017):

$$y = X\beta + u \tag{7.19}$$

Where

$$y = \begin{bmatrix} y_1 \\ y_2 \\ \vdots \\ y_t \end{bmatrix};\ X = \begin{bmatrix} 1 & x_{11} & x_{12} & \cdots & x_{1k} \\ 1 & x_{21} & x_{22} & \cdots & x_{2k} \\ & \vdots & \vdots & & \vdots \\ 1 & x_{t1} & x_{t2} & \cdots & x_{tk} \end{bmatrix};\ \beta = \begin{bmatrix} \beta_0 \\ \beta_1 \\ \vdots \\ \beta_k \end{bmatrix};\ u = \begin{bmatrix} u_1 \\ u_2 \\ \vdots \\ u_t \end{bmatrix}$$

In general, y is a T x 1 vector of the observations, X is a T x k matrix of the levels of the regressor variables, β is a k x 1 vector, and u is a T x 1 vector of random errors.

If the estimated model is denoted as $\hat{y} = X\hat{\beta}$, the error vector can be expressed as $\hat{u} = y - \hat{y} = y - X\hat{\beta}$. If the sum of the squares of the errors is denoted as S:

$$S = \hat{u}'\hat{u} = (\hat{u}_1, \hat{u}_2, \ldots, \hat{u}_T)\begin{bmatrix} \hat{u}_1 \\ \hat{u}_2 \\ \vdots \\ \hat{u}_T \end{bmatrix} = \sum_{t=1}^{T} \hat{u}_1^{\,2} \tag{7.20}$$

Eq. 7.20 can also be expressed as:

$$S = \left(y - X\hat{\beta}\right)'.\left(y - X\hat{\beta}\right) = y'y - \hat{\beta}'X'y - y'X\hat{\beta} + \hat{\beta}'X'X\hat{\beta} = y'y - 2\hat{\beta}'X'y + \hat{\beta}'X'X\hat{\beta} \tag{7.21}$$

Since $\beta'X'y$ is a 1 x 1 matrix, or a scalar, and its transpose $\left(\beta'X'y\right)' = y'X\beta$ is the same scalar. Applying the least squares criterion is equivalent to minimize the scalar S. The least squares estimators must satisfy:

$$\frac{\partial S}{\partial \hat{\beta}} = -2X'y + 2X'X\hat{\beta} = 0 \tag{7.22}$$

It simplifies to

$$X'X\hat{\beta} = X'y \tag{7.23}$$

Equations in [7.23] are the least-squares normal equations. They are the matrix analogue of the scalar presentation in Eq. 7.18. To solve the normal equations, both sides of the Eq. 7.23 must be multiplied by the inverse of $X'X$, provided that the inverse matrix $(X'X)^{-1}$ exists. The $(X'X)^{-1}$ matrix will always exist if the regeressors are linearly independent, that is, if no column of the X matrix is a linear combination of the other columns. In other words, the range of the matrix is *k*. If the condition is fulfilled, both sides of the system can be pre-multiplied by $(X'X)^{-1}$ (Montgomery *et al.*, 2012; Rao *et al.*, 2017):

$$(X'X)^{-1}(X'X)\hat{\beta} = (X'X)^{-1}X'y \Rightarrow \hat{\beta} = (X'X)^{-1}X'y \qquad [7.24]$$

7.5.2.1. Properties of the Least-Squares Estimators

The statistical properties of the least-squares estimator $\hat{\beta}$ may be easily demonstrated (Montgomery *et al.*, 2012). Consider first bias, assuming that the model is correct:

$$E(\hat{\beta}) = E\left[(X'X)^{-1}X'y\right] = E\left[(X'X)^{-1}X'(X\beta + u)\right] = E\left[(X'X)^{-1}X'X\beta + (X'X)^{-1}X'u\right] = \beta \qquad [7.25]$$

Since $E(u) = 0$ and $(X'X)^{-1}X'X = I$. Hence, $\hat{\beta}$ is an **unbiased estimator** of β if the model is correct.

The variance property of $\hat{\beta}$ is expressed by the **covariance matrix.**

$$Cov(\hat{\beta}) = E\left\{\left[\hat{\beta} - E(\hat{\beta})\right]\left[\hat{\beta} - E(\hat{\beta})\right]'\right\} \qquad [7.26]$$

Which is a *p* x *p* symmetric matrix whose *j*th diagonal element is the variance of $\hat{\beta}_j$ and whose *(ij)*th off-diagonal element is the covariance between $\hat{\beta}_i$ and $\hat{\beta}_j$. The covariance matrix of $\hat{\beta}$ is found by applying a variance operator to $\hat{\beta}$:

$$Cov(\hat{\beta}) = Var(\hat{\beta}) = Var\left[(X'X)^{-1}X'y\right] \qquad [7.27]$$

And $(X'X)^{-1}X'$ is a matrix of constants, and the variance of *y* is $\sigma^2 I$, so

$$Var(\hat{\beta}) = Var\left[(X'X)^{-1}X'y\right] = (X'X)^{-1}X'Var(y)\left[(X'X)^{-1}X'\right]' =$$

$$' = \sigma^2 (X'X)^{-1}X'X(X'X)^{-1} = \sigma^2 (X'X)^{-1} \qquad [7.28]$$

Consequently, if we let $A = (X'X)^{-1}$, the variance of $\hat{\beta}$ is $\sigma^2 A_{jj}$ and the covariance between $\hat{\beta}_i$ and $\hat{\beta}_j$ is $\sigma^2 A_{ij}$. Furthermore, it can be demonstrated that the least-square estimator $\hat{\beta}$ is the best linear unbiased estimator of β (the Gauss-Markov theorem). If it assumed that the errors u_i are normally distributed, then $\hat{\beta}$ is also the maximum-likelihood estimator of β. The maximum-likelihood estimator is the minimum variance unbiased estimator of β (Montgomery *et al.*, 2012).

On the other hand, it can be developed an estimator of σ^2 from the residual sum of squares, SS_{res} (Montgomery *et al.*, 2012):

$$SS_{res} = \sum_{t=1}^{T}\left(y_t - \hat{y}_t\right)^2 = \sum_{t=1}^{T}\hat{u}_t^2 = \hat{u}'\hat{u} \qquad [7.29]$$

Substituting $\hat{u} = y - X\hat{\beta}$, it is obtained:

$$SS_{res} = \left(y - X\hat{\beta}\right)'\left(y - X\hat{\beta}\right) = y'y - \hat{\beta}'X'y - y'X\hat{\beta} + \hat{\beta}'X'X\hat{\beta} = y'y - 2\hat{\beta}'X'y + \hat{\beta}'X'X\hat{\beta} \qquad [7.30]$$

Since $X'X\hat{\beta} = X'y$, this last equation becomes

$$SS_{res} = y'y - \hat{\beta}'X'y \qquad [7.31]$$

It can be demonstrated that the residual sum of squares has $T - (k+1) = T - k - 1$ degrees of freedom associated with it since $p = k + 1$ parameters are estimated in the regression model. The **residual mean square** is:

$$MS_{res} = \frac{SS_{res}}{T - (k+1)} \qquad [7.32]$$

It can be demonstrated that the expected value of MSr_{es} is σ^2, so, an unbiased estimator of σ^2 is given by Eq. 7.33 and it is model dependent (Montgomery *et al.*, 2012).

$$\hat{\sigma}^2 = MS_{res} \qquad [7.33]$$

7.5.2.2. Analysis of variance

The following concepts are introduced (Pérez López. 2014; 2016):

- **Total sum of squares**, SS_T, is the corrected sum of squares of the observations, which measures the total variability of the observations around the mean of the dependent variable, $\bar{y}$ (Eq. 7.34)

$$SS_T = \sum_{t=1}^{T}\left(y_t - \bar{y}\right)^2 = y'y - T\bar{y}^2 \qquad [7.34]$$

- **The regression or model sum of squares**, SS_R, which measures the amount of variability in the observations y_i accounted for by the regression model. It measures the variability of the predicted values $\hat{y}_i$ around the mean of the dependent variable (Eq. 7.35).

$$SS_R = \sum_{t=1}^{T}\left(\hat{y}_t - \bar{y}\right)^2 = \hat{y}'y - T\bar{y}^2 \qquad [7.35]$$

- **The residual or error sum of squares**, which has been introduced in Eq. 7.29. It indicates the level of error the model when trying to account for the variability of y_t.

It is known from Eq. 7.29 and 7.30 that:

$$SS_{res} = \sum_{t=1}^{T}(y_t - \hat{y}_t)^2 = \hat{u}'\hat{u} = \left(y - X\hat{\beta}\right)'\left(y - X\hat{\beta}\right) = y'y - \hat{\beta}'X'y = y'y - \hat{y}'y \qquad [7.36]$$

So, the expression $y'y = \hat{y}'y + \hat{u}'\hat{u}$ can be written. If $T\overline{y}^2$ is substracted to both components of the equation, it is obtained that:

$$\left(y'y - T\overline{y}^2\right) = \left(\hat{y}'y - T\overline{y}^2\right) + \hat{u}'\hat{u} \Rightarrow SS_T = SS_R + SS_{res} \qquad [7.37]$$

So, the total sum of squares SS_T is portioned into a sum of squares due to regression, SS_R, and a residual sum of squares, SS_{res}. The three components are called Sum of Squares (Hair *et al*., 1999; Montgomery *et al*., 2012; Darlington and Hayes, 2017).

Each sum of squares divided by its degrees of freedom is called **Mean Square**. If the hypothesis of normal distribution of the residuals is assumed, SS_R follows a χ^2 (Chi-Squared) distribution with k degrees of freedom, the number of regressor variables in the model. SS_{res} follows a χ^2 distribution with $T - k - 1$ degrees of freedom. SS_T follows a χ^2 distribution with $T - 1$.

Therefore, the Mean Square of the model is expressed in Eq. 7.38 and the Mean Square of the residuals (or residual mean square) is expressed in Eq. 7.39.

$$MS_R = \frac{SS_R}{k} \qquad [7.38]$$

$$MS_{res} = \frac{SS_{res}}{T - k - 1} \qquad [7.3982]$$

The coefficient of determination, R^2, is a descriptive measure of the global adjustment of the regression model, defined as:

$$R^2 = \frac{SS_R}{SS_T} = 1 - \frac{SS_{res}}{SS_T} \qquad [7.40]$$

Since SS_T is a measure of the variability in y without considering the effect of the regressor variables and SS_{res} is a measure of the variability in y remaining after the independent variables have been considered, R^2 is often called the proportion of variation explained by the model. Since $0 \leq SS_{res} \leq SS_T$, it follows that $0 \leq R^2 \leq 1$. Values of R^2 that are close to 1 imply that most the variability in y is explained by the regression model. In general, R^2 never decreases when a regressor is added to the model, regardless of the value of the contribution of that variable. Consequently, it is difficult to judge whether an increase in R^2 is really indicating anything important. Some regression model builders prefer to use an adjusted coefficient of determination, R^2_{adj}, a statistic defined as

$$R_{adj}^2 = 1 - \frac{SS_{res}/(T-k-1)}{SS_T/(T-1)} = 1 - \left(1 - R^2\right)\frac{T-1}{T-k-1} \qquad [7.41]$$

Since $SS_{res}/(T - k - 1)$ is the residual mean square and $SS_T/(T - 1)$ is constant regardless of how many variables are in the model, R_{adj}^2 will only increase on adding a variable to the model if the addition of the variable reduces the residual mean square. The adjusted R^2 is a helpful to guard against overfitting the model, i.e. adding terms that are unnecessary (Pérez López, 2014). Moreover, it is observed that when $T \to \infty$. i.e. for big samples, $(T-1)/(T-k-1) \to 1$ and it does not depend on k, the number of independent variables of the model. Moreover, $T \to \infty \Rightarrow \overline{R}^2 \to R^2$ (Pérez López, 2016; Darlington and Hayes, 2017)

From the distributions of SS_R and SS_{res}, it can be deducted that the statistic, F, follows an F distribution

$$F = \frac{\frac{SS_R}{k}}{\frac{SS_{res}}{T-k-1}} = \frac{MS_R}{MS_{res}} \to F(k, T-k-1) \qquad [7.42]$$

This statistic is used in the test for significance of regression, which determines if there is a linear relationship between the response y and any of the regressor variables, X_1, X_2, ..., X_K. This procedure is often thought of as an overall or global test of model adequacy. The appropriate hypotheses are:

$$H_0 : \beta_0 = \beta_1 = \cdots = \beta_k = 0 \qquad [7.43]$$

$$H_1 : \beta_j \neq 0 \quad \text{for at least one } j. \qquad [7.44]$$

Rejection of this null hypothesis implies that at least one of the regressors, X_1, X_2, ..., X_K contributes significantly to the model. If the observed value of F is large, then it is likely that at least one $\beta_j \neq 0$. Thus, to test the hypothesis $H_0 : \beta_0 = \beta_1 = \cdots = \beta_k = 0$ it must be computed the test statistic F and reject if:

$$F > F_{\alpha,k,T-k-1} \qquad [7.45]$$

Generally, the test procedure is summarized in an analysis-of-variance-table such as Table 7.2 (Montgomery *et al.*, 2012; Darlington and Hayes, 2017; Binghan and Fry, 2010).

Table 7.2. Analysis of variance for significance of regression in multiple linear regression

Source of variation	**Sum of squares**	**Degrees of freedom**	**Mean Square**	***F***
Regression (model)	SS_R	k	$MS_R = SS_R/k$	MS_R/MS_{res}
Residual	SS_{res}	$T - k - 1$	$MS_{res} = SS_{res}/(T - k - 1)$	
Total	SS_T	$T - 1$		

The statistical in Eq. 7.42 can also be expressed as Eq. 7.46 and follows an F of Fisher-Snedecor distribution

$$F = \frac{\left(\hat{\beta} - \beta\right)' X'X\left(\hat{\beta} - \beta\right)}{k\hat{\sigma}^2} \to F(k, T-k-1) \qquad [7.46]$$

This statistical allows verifying the null hypothesis $\left(\hat{\beta}_1, \hat{\beta}_2, \cdots, \hat{\beta}_1\right) = (0,0,\cdots,0)$. If the p-value of the contrast of the F of Fisher-Snedecor is low, the global significance of the estimated parameters of the model is accepted. This statistical also allows finding Confidence Intervals at α% of significance level for the conjunct of parameters β_i of the model.

Once it has been determined that at least one of the regressors is important, a logical question becomes which one(s). Adding a variable to a regression model always causes the sum of squares for regression to increase and the residual sum of squares to decrease. It must be decided whether the increase in the regression sum of squares is sufficient to warrant using the additional independent variable in the model. The addition of a regressor also increases the variance of the fitted value $\hat{y}$, so it must be careful to include only variables that are of real value in explaining the response. Moreover, adding an unimportant regressor may increase the residual mean square, which may decrease the usefulness of the model.

As seen in Eq. 7.28, the variance of $\hat{\beta}_i$ is $\sigma^2 A_{ii}$, where A_{ii} is the element (ii)th of $[X'X]^{-1}$. Similarly, the covariance between $\hat{\beta}_i$ and $\hat{\beta}_j$ is $\sigma^2 A_{ij}$, where A_{ij} is the element (ij)th of $[X'X]^{-1}$. From those results, it can deducted that the estimator $\hat{\beta}_i$ of any of the coefficients β_i, has as mean response β_i, and as variance $\sigma^2 A_{ii}$, where A_{ii} is the element (ii)th of $[X'X]^{-1}$. Thus, under the hypothesis of normal distribution of the residuals, the statistical in Eq. 7.47 follows a normal distribution.

$$N_i = \frac{\hat{\beta}_i - \beta_i}{\sigma \sqrt{A_{ii}}} \rightarrow N(0,1) \qquad [7.47]$$

The estimator from least square method (and also from maximum likelihood, which has not be commented) of σ^2 is $\hat{u}'\hat{u}/T$ but it is not unbiased. An unbiased estimator of the error variance is:

$$\hat{\sigma}^2 = \frac{\hat{u}'\hat{u}}{T - k - 1} \qquad [7.48]$$

On the other hand, it can be demonstrated that the statistic $G = u'u/\sigma^2$ follows a (Chi-Square) χ^2 with T–k–1 degrees of freedom, which allow calculating confidence intervals and hypothesis contrast for σ and its square. The distributions of the statistics N_i and G leads to conclude that the statistical $N_i/[G/(T - k - 1)]^{1/2}$ is a Student's t distribution with $T - k - 1$ degrees of freedom- Thus, it can be stated that the statistical:

$$T_i = \frac{\hat{\beta}_i - \beta}{\hat{\sigma} \sqrt{A_{ii}}} \qquad [7.49]$$

Follows a Student's t distribution with $T - k - 1$ degrees of freedom. It allows finding confidence intervals and hypothesis contrast for the parameters β_i of the model.

The hypotheses for testing the significance of any individual regression coefficient, such as, β_i, are:

$$H_0 : \beta_i = 0, \qquad H_1 : \beta_i \neq 0 \qquad [7.50]$$

If Ho: $\beta_i = 0$ is not rejected, then this indicates that the independent variable X_i can be deleted from the model. The test statistic for this hypothesis is (Montgomery *et al*., 2012; Pérez López, 2014; 2016):

$$T_i = \frac{\hat{\beta}_i}{\sqrt{\hat{\sigma}^2 A_{ii}}} = \frac{\hat{\beta}_i}{\hat{\sigma}\sqrt{A_{ii}}} = \frac{\hat{\beta}_i}{se(\hat{\beta}_i)} \qquad [7.51]$$

Where A_{ii} is the diagonal element of $[X'X]^{-1}$ corresponding to $\hat{\beta}_i$, i.e. the element (*ii)*th of $[X'X]^{-1}$. The denominator of the test statistic is often called the **estimated standard error**, or more simply, the **standard error** of the coefficient $\hat{\beta}_i$ and it is:

$$se(\hat{\beta}_i) = \sqrt{\hat{\sigma}^2 A_{ii}} = \hat{\sigma}\sqrt{A_{ii}} \qquad [7.52]$$

The null hypothesis Ho: $\beta_i = 0$ is rejected if

$$|T_i| > t_{\alpha/2, T-k-1} \qquad [7.53]$$

It must be noted that this is really a partial or marginal test because the regression coefficient $\hat{\beta}_i$ depends on all of the other independent variables $X_j (j \neq i)$ that are in the model. Thus, this is a test of the **contribution** of X_i **given the other independent variables in the model**.

7.5.2.3. Confidence intervals in multiple regression

Confidence intervals on individual regression coefficients and confidence intervals on the mean response given specific levels of the independent variables play an important role in multiple regression (Montgomery *et al.*, 2012).

7.5.2.3.1. Confidence intervals on the regression coefficients

To construct confidence interval estimates for the regression coefficients β_i, it is necessary to assume that the errors are normally and independently distribute with mean zero and variance σ^2 (in subsection 7.5.4. it is shown how to verify it). Consequently, the observations y_i are normally and independently distributed with mean $\beta_0 + \sum_{i=1}^{k} \beta_i X_{ji}$ and variance σ^2. Since the least-squares estimator $\hat{\beta}_i$ is a linear combination of the observations, it follows that $\hat{\beta}$ is normally distributed with mean vector β and covariance matrix $\sigma^2[X'X]^{-1}$. This implies that the marginal distribution of any regression coefficient $\hat{\beta}_i$ is normal with mean β_i and variance $\sigma^2 A_{ii}$, where A_{ii} is the *i*th diagonal element of the $[X'X]^{-1}$ matrix. Consequently, each of the statistics

$$\frac{\hat{\beta}_i - \beta_i}{\sqrt{\hat{\sigma}^2 A_{ii}}}, \quad i = 0,1,\ldots,k \qquad [7.83]$$

Is distributed as a Student's *t* with $T - k - 1$ degrees of freedom, where $\hat{\sigma}^2$ is the estimate of the error variance obtained from Eq. 7.33. Based on the result given in Eq. 7.54, it can be defined **a 100(1-α) percent confidence interval for the regression coefficient $\boldsymbol{\beta_i}$**, i = 0,1,…,k, as

$$\hat{\beta}_i - t_{\alpha/2,T-k-1}\sqrt{\hat{\sigma}^2 A_{ii}} \leq \beta_i \leq \hat{\beta}_i + t_{\alpha/2,T-k-1}\sqrt{\hat{\sigma}^2 A_{ii}} \qquad [7.55]$$

7.5.2.3.2. Confidence intervals estimation of the mean response

It is possible to construct a confidence interval on the mean response at a particular point, such as X_{01}, X_{02},...,X_{0k}. The vector X_0 is defined as:

$$X_0 = \begin{bmatrix} 1 \\ X_{01} \\ X_{02} \\ \vdots \\ X_{ok} \end{bmatrix} \qquad [7.56]$$

The fitted values at this point are:

$$\hat{y}_0 = X_0'\hat{\beta} \qquad [7.57]$$

This is an unbiased estimator of E(y/X_0), since $E(\hat{y}_0) = X_0'\beta = E(y|X_0)$, and the variance of $\hat{y}_0$ is

$$Var(\hat{y}_0) = \sigma^2 X_0'[X'X]^{-1} X_0 \qquad [7.58]$$

Therefore, a 100(1 – α) percent confidence interval on the mean response at the point X_{01}, X_{02},...,X_{0k} is

$$\hat{y}_0 - t_{\alpha/2,T-k-1}\sqrt{\hat{\sigma}^2 X_0'[X'X]^{-1} X_0} \leq E(y|X_0)_i \leq \hat{y}_0 + t_{\alpha/2,T-k-1}\sqrt{\hat{\sigma}^2 X_0'[X'X]^{-1} X_0}$$

7.5.3. Hypotheses in a multiple linear regression model.

The basic hypotheses that must be fulfilled in any multiple linear regression model can be grouped in 6 big groups according to the components of the model. The hypotheses in each group are commented in the next subsections. The first group refers to the global indices of the models. The second group is made of the test of individual and joint significance of the estimated parameters of the models. The third group regards the random variable u. The forth group refers to the regressors. The fifth group is based on the functional shape. The last group is made up of the hypotheses relatives to the parameter vectors (β_0, β_1, β_2,..., β_k) (Hair *et al.*, 1999; Pérez López, 2016).

7.5.3.1. Model indices. Coefficient of determination and statistic about the information quantity

The first indicator that must be considered in a regression model is the coefficient of determination. The

coefficient of determination (R^2), defined in Eq. 7.40., is a descriptive measure of the global adjustment of the model, which quantifies the proportion of variation of the dependent variable explained by the independent variables included in the model and varies from 0 to 1. A greater R^2 means a better model, but this idea cannot be considered as a justification for including more predicting variables. This problem can be solved considering the **adjusted coefficient of determination**, R^2_{adj}, defined in Eq. 7.41. As previously commented, with a lot of observations, the adjusted coefficient of determination is not dependent on k, the number of variables of the model. Therefore, it can be considered as a good measure of the quality of the regression. A greater R^2_{adj} means a better model.

Another group of indicators that are employed are the statistics of the information quantity (or of predicting capacity).

When adjusting a multiple regression model, there are different ways (Hair *et al.*, 1999; Pérez López, 2014; 2016). It can be calculated with the introduction of all the variables in a unique step, which is the usual method. The **step by step selection** analysis allows specifying the criteria for including new variables and for excluding existing ones if some requirements are fulfilled in each step. A new variable is introduced in the equation from those not included and with the lower probability of F if that probability is low enough. The variables introduced in the model are eliminated if their probability of F is big enough. The procedure ends when new candidates for being introduced or rejected are available. The **forward stepwise** regression analysis can be made, where independent variables are included in the model after obtaining the ideal adjustment. The firstly introduced variable is the one that has a higher correlation, negative or positive, with the dependent variable and only if the introducing criterion is met. After a first variable is introduced, the next one candidate will be the one that is not in the equation and whose partial correlation is the highest. The procedure ends when no more variables satisfy the introducing criterion.. The **backward stepwise** analysis includes all the possible variables in the model and eliminates the necessary ones to fit an optimum model without problems. The variable with the lowest partial correlation will be the first one to be considered and will be eliminated if the eliminating criterion is met. After the first variable had been eliminated, the procedure is repeated with the variables in the equation. The procedure finishes when there are no variables in the equation that satisfy the rejecting criterion.

The AKAIKE's AIC and Schwarz's SC statistics allow selecting the adjusted model with better predicting capacity as the one with the lowest value for the following statistics (Pérez López, 2016):

$$AIC = -\frac{2l}{T} + \frac{2(k+1)}{T} \qquad [7.59]$$

$$SC = -\frac{2l}{T} + \frac{(k+1)\log(T)}{T} \qquad [7.60]$$

$$l = -\frac{T}{2}\left(1 + \log(2\pi) + \log\frac{e'e}{T}\right) \qquad [7.61]$$

Where k is the number of independent variables in the model (without including the constant), T is the number of observations available and e is the model error.

7.5.3.2. Hypothesis related to the significance of the parameters estimated in the model

The parameters estimated of the model must be significantly different from zero individually, $\hat{\beta}_i \neq 0$ for every i = 1,2,…,k (individual significance) and globally $\left(\hat{\beta}_1, \hat{\beta}_2, \ldots, \hat{\beta}_k\right) \neq (0,0,\ldots,0)$ (global significance).

For the individual significance of the regressor coefficients, the statistical T_i (Eq. 7.51) is used, which follows a Student's t distribution with $T - k - 1$ degrees of freedom. It allows testing the null hypothesis (Eq. 7.50) and obtaining confidence intervals (Eq. 7.55).

For the global significance of the model, the statistic F (Eq. 7.46) is used, which follows a Fischer-Snedecor distribution $F(k, T - k - 1)$ and allows verifying the null hypothesis (Pérez López, 2016).

7.5.3.3. Hypotheses related to the random perturbation

The multiple regression model was formulated under the following classic hypotheses related to the random perturbation or error, u (Pérez López, 2016).

- The variable u (the error term) is a random variable with mean response null and a covariance matrix constant and diagonal (scalar matrix). In other words, for every observation t, the variable u_t has a mean value of zero and the variance σ^2 is not dependent on t, and, moreover, $Cov(u_i, u_j) = 0$ for every observation i and j different from each other, Var(u) = $\sigma^2 I_k$. The fact that the variance of u_t is constant for every t (is not dependent on t), it is called the **hypothesis of homoscedasticity (no heteroscedasticity)** and can also be expressed as $V(u | X_1, X_2, \ldots, X_k) = \sigma^2$ and $V(y | X_1, X_2, \ldots, X_k) = \sigma^2$. The fact that $Cov(u_i, u_j) = 0$ for every i different from j is called **hypothesis of no autocorrelation**.
- The error term u is a random variable non observable, which implies that the variable y is random since it depends on it (Eq. 7.13).
- The normality of the residuals (errors) is also considered, which implies that the variables u_t are normal for every t. In other words, the vector of random errors of the model has a normal distribution with mean zero, E(u) = 0 and the matrix of covariance scalar E(uu') = $\sigma^2 I$. It can be written that $u \rightarrow N(0, \sigma^2 I)$.
- The residuals should not have outliers.

7.5.3.4. Hypotheses related to the independent variables (regressors)

With regard to the independent variables the following hypotheses are considered (Pérez López, 2016):

- The variables X_1, X_2,…,X_k are not significantly correlated, i.e. there is not a linear relationship between them. This hypothesis is referred as independence hypothesis and when it is not met, it is said that the model has multicollinearity. It can also be said that the matrix of regressors has a range k.

- The variables X_1, X_2,…,X_k are deterministic (they are not stochastic regressors) since their value is constant from a sample taken and they are not correlated with the error term, u, i.e., $E(u|X_1, X_2, \ldots, X_k) = 0$ (hypothesis of exogenity). When the hypothesis of exogenity is not meet, there is a problem of endogenity.
- The regressors do not have observation or measure errors
- There are not influencing observations in the data set.

7.5.3.5. Hypotheses related to the vector of parameters

It is assumed that the vector of parameters β is a constant vector (Pérez López, 2016). This hypothesis assures the stability in the time of the estimations. The study of this hypothesis gives path to the theory of cointegration.

7.5.3.6. Hypothesis related to the functional form

Two hypotheses are assumed (Pérez López, 2016):

- The relationship between Y and X_1, X_2,…,X_k is effectively linear (linearity hypothesis).
- It is supposed that there are not specification errors, i.e., all the variables X that are relevant for the explanation of the variable Y are included in the linear model definition. In other words, there are not omitted variables and redundant variables in the model.

7.5.4. Analysis of the residuals

Once the regression model is constructed, there are some hypotheses that must be verified, the ones explained before. Among them, the hypotheses of linearity, normality, homocesdasticity, no autocorrelation and independence must be verified (Pérez López, 2014). In the following subsection, it is presented how these problems can be checked and, in case of existence, some of the usual solutions are mentioned.

7.5.4.1. The problem of the autocorrelation

In a linear model of the shape of Eq. 7.19, it is assumed a series of hypothesis and among them, the variable u (error term) is a random variable with null mean response ($E(u) = 0$) and the matrix of covariance is constant and diagonal ($Var(u) = \sigma^2 I_k$, a scalar matrix). In other words, for all the t observations, the variable u_t has a null mean and the variance is σ^2, not depending on t and, moreover, $\mathrm{Cov}(u_i,u_j) = 0$ for every i and j different between. This last fact is denominated as the no autocorrelation hypothesis (Pérez López, 2014).

In order to analyze the autocorrelation of a model, it usually begins with the plot of the residuals, especially the plot of the residuals (if possible studentized) with respect to the observation (or time), where a random structure must be observed. Apart from the plot analysis, it is usually verified by some formal contrasts:

Durbin-Watson, Wallos, h-Durbis, Creusch-Godfrey and Cochrane-Orcutt.

Among them, the Durbin-Watson statistic is the most employed and can be defined as:

$$DW = \frac{\sum_{t=2}^{T} \left(\hat{u}_t - \hat{u}_{t-1}\right)^2}{\sum_{t=1}^{T} \hat{u}_t^2} \cong 2 \cdot (1-\rho) \Rightarrow \begin{cases} DW \cong 2 & if\ \rho = 0 \\ DW \cong 0 & if\ \rho = 1 \\ DW \cong 4 & if\ \rho = -1 \end{cases} \qquad [7.62]$$

Where ρ is coefficient of perturbation between successive error of observations, u_t. Generally, the random perturbation can follow autoregressive models of mobile means of any order, but usually the autoregressive model of 1st order is the most widely employed, which can be defined as:

$$u_t = \rho \cdot u_{t-1} + e_t \qquad [7.63]$$

It can be stated that if *DW* is 0 it means that there is perfect positive autocorrelation, if *DW* is around 2 there is not autocorrelation and if *DW* is near 4 there is a perfect negative autocorrelation. If DW is between 1,5 and 2,5 it can be said that it has no autocorrelation. However, there are some tables to define the interval for accepting no autocorrelation according to the number of variables *k*.

The presence of autocorrelation in the model, which is more frequent in serial series, can be solved with the method of Cochrane-Orcutt or introducing an adequate dummy variable in the model. The Method of the Generalized Least Squares is also adopted. Other techniques that are not so frequently used are the method of the estimation of Durbin and the procedure of Prais-Winstein.

7.5.4.2. The problem of the heteroscedasticity

As previously said, the fact that the variance of u_t is constant for all the t observations (is not dependant on *t*) is denominated as homoscedasticity. When this hypothesis is not fulfilled and the variance of u_t is not constant, there is heteroscedasticity. The importance of meeting this hypothesis is that the estimators obtained from the Least Squares Method have not the minimum variance despite being unbiased. Moreover, for each variable of the model, an error variance will be calculated.

For analysing the heteroscedasticity it is usually to start with the plot analysis of the residuals. It is very useful the plot of the standardized residuals versus the standardized predicted values, which must fit the diagonal of the first quadrant. Moreover, it can also be analyzed the plot of the residuals (if possible studentized) versus the dependent variable and the independent variables, which must show a random pattern. The plot of the residuals vs. each independent variable allows detecting which variable is the most responsible of the heteroscedasticity, the one that whose plot is furthest from a random pattern. Additionally, some forma contrast of the heteroscedasticity can be conducted: Goldfeld-Quant, Glesjer, Breush-Pagan, White, GARCH, ARCH and Ramsey's RESET (Pérez López, 2014).

Generally, the problem of heteroscedasticity is solved estimating by means of the Generalized Least Squares or by using the Least Squares Method in two phases.

7.5.4.3. The problem of the multicollinearity

In section 7.5.3.4 it was commented that the variables X_1, X_2,…,X_k are linearly independent, i.e., there is not an exact relationship between them. This hypothesis is denominated hypothesis of independence and when the model does not meet it, the model has multicollinearity.

Some of the symptoms of the multicollinearity are:

- High values in module in the correlation matrix of the independent variables
- Low significance of the independent variables and at the same time, a high coefficient of determination (R^2).
- High global significance of the model (the hypothesis of $R^2 = 0$ is rejected)
- High influence on the estimators if one observation of the data set is eliminated.
- If the Variance Inflation Factor (*VIF*) defined as in Eq. 7.64 is high (> 10).

$$VIF = 1/\left(1 - R_j^2\right) \qquad [7.64]$$

Where R_j^2 is the R^2 of the independent variable *j* as a function of the rest of variables.

The most employed solutions for the multicollinearity are (Pérez López, 2014):

- Take more observations or transform some variables (for example, using ratios or differences).
- Eliminate some variable with a statistical and economical justification
- Substitute some predicting variables by their most significant principal components (punctuations) and calculate the regression with the components (which are uncorrelated and there is not multicollinearity).
- Employ the model with difference bewaring of the autocorrelation.

7.5.4.4. Residual normality

One of the most important hypothesis in the multiple regression model is the normal distribution of the residuals. Although this hypothesis is not necessary for obtaining the estimators of the parameters of the model by means of the least squares method, it is necessary for conducting the inference in the model.

For testing the normality of the residuals it can be employed any contrast to test the normal distribution, as the Chi-Square contrast or the statistic of Kolmogorov-Smirnov. However, for big data set (> 30 observation), the statistic of Shapiro-Wilks is recommended, which is said to be more conservative.

The statistic of Shapiro-Wilks measures if the residuals of the regression fit a line when drawn in a normal probabilistic paper. The normality is rejected when the fitting is low, which corresponds to low values of the test statistical. The statistical is:

$$w = \frac{1}{ns^2}\left[\sum_{j=1}^{h} a_{j,n}\left(x_{(n-j+1)} - x_{(j)}\right)\right]^2 = \frac{A^2}{ns^2} \qquad [7.65]$$

Where. $ns^2 = \sum (x_i - \bar{x})^2$, h is $n/2$ if n is even and $(n-1)/2$ if n is odd. The coefficients $a_{j,n}$ are tabulated and $x_{(j)}$ is the ordered value in the sample that is place in the j position. The distribution of w is tabulated, and the normality is rejected when its value calculated from the data set is lower than the corresponding critical value, provided in tables. However, it can be adopted the criterion of the *p*-value, rejecting the null hypothesis of the normal distribution of the residuals with a α level when the *p*-value is lower than α, and accepting it in the other case.

Generally, the lack of normality on the residuals comes from the presence of outliers which generates a non symmetric or different distribution. These problems on the residuals appear when important predicting variables are omitted in the models or when linearity lacks in their specification. If these problems are solved, the problems of normality disappear. When the residuals are not normal and have more than one mode, data usually come from different samples, which can be solved by introducing fiction variables. In other cases, the solution of the lack of normality is the adequate transformation of the variables (Pérez López, 2014;.

7.5.4.5. Analysis of the leverage

The analysis of the leverage has the aim of knowing the observations that has a greater influence in the estimation of the model and the atypical or heterogeneous observations that are fit to the model.

It can be observed by the distance of Mahalanobis, which is the distance of the observations $\vec{x}_i = (x_{i1}, x_{i2}, \ldots, x_{ik})$ $i = 1,2,\ldots,n$, to the mean point of the cloud of the predicting variables $\bar{x} = (\bar{x}_1, \bar{x}_2, \ldots, \bar{x}_k)$, where $\bar{x}_j$, $j = 1,2,\ldots,k$ is the mean value of the data of the variable *j*. This distance is defined by the expression:

$$d_M^2(\vec{x}_i ; \bar{x}) = (\vec{x}_i - \bar{x}) S^{-1} (\vec{x}_i - \bar{x})^t \qquad [7.66]$$

Where *S* is the matrix of variance-covariance of the variables vector $(x_1, x_2, \ldots, x_k)$.

The distance of Mahalanobis is a statistical distance that generalizes the Euclidean distance between two vectors in which the dispersion of the variables and their dependence is considered. A high value of the Mahalanobis distance means that the point is far from the center of the cloud, and hence, it is a possible influencing observation (Pérez López, 2014).

7.5.5. ANOVA, ANCOVA models and General Linear Models

7.5.5.1. Analysis of the simple variance ANOVA

In Fig. 7.2 it was shown that, when qualitative variables were used to predict a quantitative variable, an Analysis Of Variance (ANOVA) was employed. The model ANOVA measures the statistical significance of the difference between the means of the levels produced in the dependent variable by the categories (levels) of the qualitative independent variables. Eq. 7.7 represents the functional expression of the model.

The Analysis of Variance (ANOVA) of a unique factor is employed when it is analyzed the relationship between the quantitative dependent variable and a qualitative independent variable, studied in its different groups of levels (Pérez López, 2014).

In the case of the model with random effects, it is considered that, from the total population of levels of the factor, the G levels employed in the experiment are chosen randomly. In this case, the model ANOVA of random effects has the form of Eq. 7.67 (Pérez López, 2014).

$$Y_{ij} = \mu_i + \varepsilon_{ij} \qquad [7.67]$$

And a equivalent equation, if it is considered that $\mu_i = \mu + \beta_i$

$$Y_{ij} = \mu_i + \beta_i + \varepsilon_{ij} \qquad [7.68]$$

Where μ is a constant, β_i for i = 1,..., G are independent random variables with a distribution N(0, σ^2_β) ; ε_{ij} for i = 1,..., G and j = 1,..., n_i are independent random variables identically distributed N(0, σ^2_β). In this model it is verified that E[Y_{ij}] = μ, and the variance of Y_{ij}, denoted as σ_Y^2 is $V[Y_{ij}] = \sigma_Y^2 = \sigma_\beta^2 + \sigma^2$ where σ_β^2 and σ^2 are referred as *components of the variance*. Once the model is estimated, it is necessary to check by means of different test the basic hypothesis of it are no contradictory with the observed data.

It is interesting to check if it can be accepted the hypothesis of the mean equality, i.e. the means of all the observed levels after repeating the experiment are equal (u_1 = u_2 = ...= u_G = u). If the tests say that this hypothesis is true, the fact that an observation belongs to a group would be irrelevant, and it could be considered that all the observations comes from a unique population. An alternative point of view, which leads to the same conclusion, is to observe if the difference of mean between the levels are small. Confidence intervals for the mean differences between groups ($u_i - u_j$) can be made with the aim to observe the difference between them. If the interval contains the zero (0), it can be accepted the hypothesis of mean equality for the groups (levels) (Pérez López, 2014).

In general, when analyzing the levels of a factor, apart from knowing globally if the levels of the factors are significantly different between then in their effect on the dependent variable (which is interesting), it also worth knowing if, after knowing that the differences are significant, which levels produce a higher effect than the others on the dependent variable. This is conducted by some contrast tests, such as Bonferroni, Tukey's HSD; Tamhane's T2 or Dunnett's T3 (Pérez López, 2014).

Until here, it has supposed that the levels of the factor are infinite, i.e., the factors are random. In this case, a *model of random effects* is analyzed. However, a model of analysis of variance is a model of fixed effects when the results are only valid for some levels of the factor, analyzed in this case (constant factors), which can be different to what happens in other levels. The table for the analysis of variance is the same for random and fixed effects. The only difference is that with random effects, the components of the variance must be considered (with the presence of σ_β^2) (Pérez López, 2014).

If there are two factors, and both groups of levels of the factor can be considered a sample from a big enough population which is going to be examined. In this case, it is a *bifactorial general ANOVA model with random effects*, which can be expressed as in Eq. 7.69

$$Y_{ij} = \mu_i + \beta_i + \delta_j + (\beta\delta)_{ij} + \varepsilon_{ijl} \qquad \beta\delta = \text{interaction between the factors} \qquad [7.69]$$

Where, for $i = 1,..., h$; $j = 1,..., k$; $l = 1,..., t$, it is verified that μ is a constant, β_i are random independent variables distributed following a normal distribution N(0, σ^2_β); δ_j are random independent variables distributed following N(0, σ^2_β), $(\beta\delta)_{ij}$ are random independent variables distributed following N(0, σ^2_β), ε_{ijl} are random independent variables distributed following N(0, σ^2); and β_i, δ_j, $(\beta\delta)_{ij}$, and ε_{ijl} are independent random variables between any two of them. But in a factorial model of two factors, both factors can be fixed (bifactorial model of fixed effects), both random (bifactorial model of random effect) or one random and the other fixed (bifactorial model of mixed effects).

In a bifactorial ANOVA model of mixed effects, if the factor *A* has fixed levels and the factor *B* has random levels, the effects β_i are constants, the effects δ_j are random variables and the effects of the combination $(\beta\delta)_{ij}$ are random variables because δ_j are random too. Supposing equal sample size for each treatment, the general bifactorial model with mixed effects follows Eq. 7.70 (Pérez López, 2014)..

$$Y_{ijl} = \mu + \beta_i + \delta_j + (\beta\delta)_{ij} + \varepsilon_{ijl} \qquad [7.70]$$

Where μ is a constant, β_i are constant, verifying that $\Sigma\beta i = 0$; δ_j for $j = 1,..., k$ are random independent variables distributed following N(0, σ^2_β), $(\beta\delta)_{ij}$ for $i = 1, ..., h$ are random independent variables following N(0, $\sigma^2_{\beta\delta}(h\text{-}1/h)$) restricted to $\Sigma(\beta\delta)ij = 0$ for every $j = 1,..., k$; ε_{ijl} for $i = 1,..., h$, $j = 1,..., k$ and $l = 1,..., t$ are random independent variables distributed following N(0, σ^2); and β_i, δ_j, $(\beta\delta)_{ij}$, and ε_{ijl} are independent variables between any two of them.

The last possibility is a bifactorial ANOVA model with fixed effects and its general equation for two factors, *A* and *B*, of fixed effects, is Eq. 7.71.

$$Y_{ijk} = u + A_i + B_j + AB_{ij} + E_{ijk} \quad i = 1,\dots,t,\ j = 1,\dots r,\ k = 1,\dots n_{ij}\ \ u = cte \qquad [7.71]$$

A_i and B_j represent the effects of the factors *A* and *B* (principal effects), and are constant with the restriction:

$$\sum_{i=1}^{t} A_i = \sum_{j=1}^{r} B_j = 0 \qquad [7.72]$$

The component AB_{ij} represents the effect of the interaction between the factor *A* and *B*, and are constant and restricted to Eq. 7.73.

$$\sum_{i=1}^{t} (AB)_{ij} = \sum_{j=1}^{r} (AB)_{ij} = 0 \qquad [7.73]$$

The component E_{ijk} represent the experimental error, which corresponds to normal random variable of mean zero and constant variance σ^2 for every *k* (the variables E_{ijk} must be independent).

For a factorial model of three factors *A*, B and *C*, the expression is Eq. 7.74.

$$Y_{ijkl} = u_i + \alpha_i + \beta_j + \gamma_k + (\alpha\beta)_{ij} + (\alpha\gamma)_{ik} + (\alpha\beta\gamma)_{ijk} + \delta_{ijkl}$$

$$i = 1,\ldots,t,\ j = 1,\ldots r,\ k = 1,\ldots s,\ l = 1,\ldots n_{ijk} \qquad [7.74]$$

The components α_i, β_j and γ_k represent the effects of the factor *A*, *B* and *C* (principal effects). The component $(\alpha\beta)_{ij}$ represents the effect of the interaction between the factors *A* and *B*. The component $(\alpha\gamma)_{ik}$ represents the effect the interaction between the factors *A* and *C*. The component $(\beta\gamma)_{jk}$ represents the effect of the interaction between the factors *B* and C. The component $(\alpha\beta\gamma)_{ijk}$ represent the triple interaction between the factors *A*, *B* and *C*. The component δ_{ijkl} represents the experimental error, which correspond to a normal random variable of mean zero and constant variance for every *l*. The variables must be independent. The model can also be considered with constant component. In factorial model of three factors, the three factors can be fixed, the three can be random, one random and two fixed and finally, two random and the other one fixed (Pérez López, 2014).

7.5.5.2. Analysis of the simple covariance ANCOVA

As commented in 7.5. the analysis of simple covariance is a statistical technique used for analyze the relationship between a quantitative dependent variable and some independent variables, and some of them are qualitative and the rest are quantitative (covariables). The expression of the analysis of simple covariance ANCOVA is shown in Eq. 7.8 (Pérez López, 2014).

The simplest ANCOVA model has one factor (qualitative independent variables) and one covariable (quantitative independent variables) and is represented by Eq. 7.75.

$$Y_{ij} = u + A_i + \beta X_{ij} + E_{ij} \quad i = 1,\ldots,t,\ j = 1,\ldots n_i \qquad [7.75]$$

Where *A* is the fixed factor and X_{ij} is the covariable. X_{ij} is not a random variable. The error E_{ij} is a random variable, with the hypothesis of normal distribution, homoscedasticity, independence and null mean response. The variables E_{ij} are normal distributed N(0, σ2) and independent, and since *A* is a fixed factor, its difference level must verify the condition

$$\sum_{i=1}^{t} n_i A_i = 0 \qquad [7.76]$$

If $n_i = n$ for every $i = 1,\ldots, t$, the model is equilibrated.

For a model with two factors and one covariable, the equation of the model is (Pérez López, 2014):

$$Y_{ij} = u + A_i + B_j + \beta X_{ij} + E_{ij} \quad i = 1,\ldots,t,\ j = 1,\ldots n_i \qquad [7.77]$$

Where A_i and B_j are the fixed factors and X_{ij} is the covariable, which is not a random variable. The error E_{ij} is a random variable, with the hypothesis of normal distribution, homoscedasticity, independence and null mean response. The variables E_{ij} are normal distributed N(0, σ^2) and independent, and since *A* and *B* are fixed factors, their difference levels must verify the condition

$$\sum_{i=1}^{t} A_i = \sum_{j=1}^{n} B_i = 0 \qquad [7.78]$$

If $n_i = n$ for every $i = 1,\ldots, t$, the model is equilibrated.

For a model with two factors and two covariables, the equation of the model is (Pérez López, 2014).

$$Y_{ij} = u + A_i + B_j + \gamma X_{ij} + \delta W_{ij} + E_{ij} \quad i = 1,\ldots,t,\ j = 1,\ldots n_i \qquad [7.79]$$

Where A_i and B_j are the fixed factors and X_{ij} and W_{ij} are the covariables, which are not random variables. The error E_{ij} is a random variable, with the hypothesis of normal distribution, homoscedasticity, independence and null mean response. The variables E_{ij} are normal distributed N(0, σ2) and independent, and since *A* and B *are* fixed factors, their difference levels must verify the condition.

$$\sum_{i=1}^{t} A_i = \sum_{j=1}^{n} B_i = 0 \qquad [7.80]$$

If $n_i = n$ for every $i = 1,\ldots, t$, the model is equilibrated.

7.5.5.3. General linear multiple (GLM) regression models

The General Linear Multiple (GLM) regression model is the most general model of linear regression, including the multiple linear regression model with quantitative variables and the multiple regression models with qualitative and quantitative variables at the same time and hence, it includes all the models of analysis of variance (ANOVA) and covariance (ANCOVA) (Pérez López, 2014). This model can be developed by the majority of statistical software programmes.

7.6. Conclusions

The Chapter showed the reasons for selecting the predicted variables for the performance models and the type of model chosen for predicting them. The International Roughness Index (IRI) and the SCRIM Coefficient (SC) were adopted as predicted variables for the roughness and skid resistance prediction, respectively. Other pavement parameters do not have an extensive historical data as these indices, and hence, they were discarded.

After revising all the possible models, a deterministic and empirical model, by means of regression analysis, is selected for prediction. It has been considered as the better option taking into account the data availability and the aim of the road agency. It allows introducing many factors as possible predicting variables, and discard those that are not relevant in the forecasting. Moreover, it can be employed by other practitioners. The predicting variables introduced in the IRI and SCRIM Coefficient models are described in Chapter 8 and 9, respectively, together with the developed models.

Additionally, the regression analysis methodology has been described as an option of the data analysis techniques. More specifically, the multiple linear regression techniques is deeply commented, focusing on the methodology of Ordinary Least Squares (OLS), identifying its properties and its assumptions. Once the model is developed, those hypothesis that are assumed must be verified. Those hypothesis are exposed, the way that they can be controlled and the possible solutions if one of the hypothesis is not fulfilled

Chapter 8. IRI performance models for pavement roads in Biscay

8.1. Introduction

This Chapter exposes the performance models for IRI prediction developed for the road network of Biscay. First of all, factors that may affect IRI progression are commented, according to previous IRI models developed by other researchers, institutions and road agencies. Then, taking into account discussed variables and available data from the pavement management system of the Regional Government of Biscay, pavement sections that can be introduced in the roughness evolution calculation are indicated.

Then, roads are analyzed to obtain sections that can be included in the calculation and different models are proposed according to the pavement structure. For each model, introduced predicting variables are commented. In a next step, after analyzing available data, by means of a regression analysis, deterioration models are produced. Not all the introduced predictors are employed in the final model, since they do not really influence the dependent variable. The introduction or not of the predictors in the models is carried out following statistical analysis, according to the methodology presented in section 7.5.

Finally, the accuracy of each model is commented, mainly observed by means of the coefficient of determination (R^2) and other statistical parameters, like the F-test and Student's t-test for each of the predictors.

8.2. Factors affecting the roughness evolution

In this section, factors included in deterministic models, both empirical and mechanistic-empirical, those commented in sections 5.3.1 and 5.3.2 and other ones, are reviewed and discussed. After revising them, factors selected for inclusion in the deterioration models for the road network of Biscay are suggested in next sections.

Generally, roughness models employs as predicting factors pavement distresses, site conditions, climatic conditions, structural parameters, age and traffic volumes.

As previously stated, the empirical models proposed by the World Bank for roughness progression, HDM-III and HDM-4, are widely used, due to their robustness and the possibility to be adapted to different climates by means of calibration coefficients. Although they are catalogued as deterministic and empirical models, when developing them, Paterson (1987) followed a structured empirical approach in the HDM-III model. Functional form and primary variables that affect each pavement property from both mechanistic and empirical information were indentified and by means of various statistical techniques their influence and impact were combined. Hence, models combined both the theoretical and experimental bases of mechanistic approaches with in situ behaviour in real circumstances. They are defined as **mainly structured empirical models**.

The model for roughness progression proposed by Paterson (1987) included five parameters (Eq. 5.8): structural deformation, rutting, cracking, potholing and environmental factors. Additionally, other values are needed, as the time in years since last overlay o reconstruction and the traffic loads by means of annual number of equivalent standard axles (*YE4*), the and the roughness at the start of the analysis year (IRI_a). The main factor is *SNCK*, which considers the reduction in pavement strength caused by cracking in the all the bituminous layers. Additionally, calibration factors are included to introduce environmental characteristics.

The subsequent roughness evolution model of the World Bank was included in the HDM-4 (Odoki and Kerali, 2000; Morosiuk *et al.*, 2004). It is based in the HDM-III model and also employs the mentioned five parameters. The potholing factor is the only one that has changed. Factors interact between them and to the roughness evolution as shown in Fig. 5.3. The model is another incremental model, which sums the contribution of the five components, as shown in Eq. 5.13. Variables are described in section 5.3.1.1.5. For the pavement strength, the HDM-4 model employs the adjusted structural number (*SNP*) instead of the modified structural number (*SNC*) of the HDM-III model. Although not specifically indicated in Eq. 5.15, the age of the pavement (years since the last overlay or reconstruction), and the traffic loads (annual number of equivalent standard axles) are used in the calculation of the ΔRI_s, as shown in Eq. 5.16. Additionally, although being an incremental model, the IRI at the start of the year affect the IRI increase, as in the environmental component (ΔRI_e), (Eq. 5.27).

Under the COST Action 324 (European Commission, 1997), further described in 5.3.1.2, a compilation of the existing deterioration models in the European Union was carried out. For roughness evolution, 5 models were identified among participating European road agencies. These models are commented below:

- In Denmark, using values obtained by means of the Bump Integrator, the proposed model in Eq. 5.30 only includes age of the pavement as independent variable and material characteristics are introduced by means of coefficients.
- In Finland, models are proposed for calculating the roughness for the next year for Asphalt Concrete overlay and cold mix overlay and the only employs the measured roughness (IRI) in that year (Eq. 5.31 and 5.32).
- Models proposed in Hungary depend on the age of the wearing course or on the traffic volume, expressed in passenger cars, following an exponential model (Eq. 5.33 and 5.34).
- The model used in Sweden calculates the future IRI as a function of the age of the pavements (years since the last measure and the total age), environmental factor (freezing index), structural strength (deflection and thickness of bituminous layers) and geometric characteristics (the width of the road) (Eq. 5.35).

As a consequence of the COST Action 324, the PARIS programme (Performance Analysis of Road InfraStructure) was conducted to develop robust pavement performance models for the European inference space of traffic loading and climate for flexible and semi-rigid pavements (European Communities, 1999). When analyzing IRI evolution, it was observed that a linear progression with time was fit for most test section and hence, a linear relationship to model the annual change in IRI was established, as shown in Eq. 5.36. It was also observed that the IRI change per year was very small at test sections, with a 90 % of the sections with a change in IRI below 0,1 m/km. Even the IRI change could be negligible as the slope of the model was not statistically different from zero. Similar slow changes in IRI were reported by Paterson (1987), with values about 0,1 m/km per year on high-volume roads and about 0,2 m/km per year on low-volume roads.

The NordFoU project was developed by Nordic countries aiming to develop, apart from other objectives, pavement performance models (Busch *et al.*, 2010). Existing pavement deterioration models were compiled. The main factors of the models in each country are summarized below:

- In Denmark, a second degree model for IRI progression in road without maintenance only depending on the age of the pavement was in use, as compiled in the COST Action 324 (European Commission, 1997). A model for improved roughness after rehabilitation is also included, depending only on the roughness of existing pavement.
- In Norway, a table with the predicted maximum, average and minimum annual increasing rate is employed as a function of the Average Annual Daily Traffic (AADT) (Table 5.12).

In a second phase of the NordFoU project, the HDM-4 model was selected to modelling pavement performance in Nordic countries. Calibration factors of the HDM-4 model were calibrated to Nordic conditions through adjustment of the relevant calibration factors. Table 5.13 provides values for calibration factors for Sweden, Norway and Denmark road networks as a function of traffic volumes.

With the database of the Mississippi Department of Transportation (MDOT), George (2000) developed two IRI performance models. The first one was for newly constructed roads (Eq. 8.1-8.4) and the second one was for the overlaid pavements (Eq. 8.5).

$$IRI = \left[2{,}4169 + Age^{0{,}2533} \cdot \left(1 + CESAL^{0{,}2575}\right)\right] \cdot MSN^{-0{,}7753} \qquad [8.1]$$

$$MSN = SN + SN_{SG} \qquad [8.2]$$

$$SN = a_1 \cdot D_1 + a_2 \cdot D_2 \cdot m_2 + a_3 \cdot D_3 \cdot m_3 \qquad [8.3]$$

$$SN_{SG} = 3{,}51 \cdot \log_{10} CBR - 0{,}85 \cdot (\log_{10} CBR) - 1{,}43 \qquad [8.4]$$

$$IRI = \left[3{,}5746 + Age^{0{,}1701} \cdot \left(1 + CESAL^{0{,}6972}\right)\right] \cdot MSN^{-0{,}3438} \cdot TOPTHK^{-0{,}1313} \cdot RES^{-0{,}1056} \qquad [8.5]$$

Where

IRI is the roughness (m/km)
Age is the age of pavement since construction (years)
CESAL is the cumulative 18-kip Equivalent Single Axle Load (ESAL) applied to the pavement (in the heavily trafficked lane) (millions)
MSN is the modified structural number
SN is the structural number
a_i is the *i*th layer coefficient
m_i is the drainage coefficient

D_i is the *i*th layer depth

CBR is the California Bearing Ratio

TOPTHK is the thickness of the top-most overlay (mm)

RES is the resurfacing type

For roads in Dubai, AlSuleiman and Shiyab (2003) developed two IRI regression models, one for the slow lanes (Eq. 8.6) and the other for the fast lanes (Eq. 8.7) by means of an exponential relationship and pavement age as the unique explanatory variable.

$$IRI_s = 0{,}796 \cdot e^{0{,}0539 \cdot Age} \quad [8.6]$$

$$IRI_f = 0{,}824 \cdot e^{0{,}0359 \cdot Age} \quad [8.7]$$

Where

IRI_f is the International Roughness Index (m/km) in the slow lane

IRI_s is the International Roughness Index (m/km) in the fast lane

Age is the age of pavement since construction or last overlays (years)

Choi *et al.* (2004) proposed multiple linear regression analysis and ANN models for roughness evolution based on the LTPP. The regression model suggested is indicated in Eq. 8.8:

$$IRI = 4{,}08 - 0{,}616 \cdot SN + 4{,}51 \cdot AC + 7{,}79 \cdot P_{200} \cdot AC - 3{,}78 \cdot P_{200} + 0{,}709 \cdot CESAL - 0{,}489 \cdot Thick \quad [8.8]$$

Where

IRI is the International Roughness Index (in./miles)

SN is the structural number

P_{200} is the percent passing no. 200 sieze,

Thick is the thickness of top layer

AC is the asphalt content

CESAL is the cumulative ESAL

For roads in north-east Brazil, Alburquerque and Núñez (2011) proposed tow IRI performance models. These models forecast IRI as a function of the Equivalent Single Axle Load (ESAL), SN and precipitation. Owolabi *et al.* (2012) proposed deterioration models for flexible pavement roads in Nigeria by means of stepwise regression analysis, obtaining Eq. 8.9.

$$IRI = 1{,}441 + 0{,}073 \cdot C_{long} - 0{,}336 \cdot P_a + 0{,}029 \cdot R \quad [8.9]$$

Where

C_{long} is the severity level of longitudinal crack (depending on length of longitudinal crack)

P_a is the number of patches

R is the severity level of rut (depending on rut depth)

For IRI modelling for HMA overlay flexible pavements of the state of Louisiana, Khattak *et al*. (2014)

suggested a regression model (Eq. 8.10 and 8.11).

$$\ln(IRI) = -0{,}902 - 0{,}2798 \cdot \left(\frac{1}{F_n}\right) + 0{,}12078 \cdot \left(\frac{\ln(CESAL)}{T_o}\right) + 2{,}66 \cdot 10^{-4} \cdot TI + 9{,}19 \cdot 10^{-8} \cdot CPI \cdot t + \Delta \quad [8.10]$$

$$\Delta = 4{,}388 + 0{,}723 \cdot \ln(SD_0) + 0{,}513 \cdot \ln(IRI_{pp}) \quad [8.11]$$

Where

F_n is the functional classification
$CESAL$ is the cumulative equivalent single axle load
T_o is the thickness of overlay
TI is the temperature index (°C·days)
t is the age of treatment (years)
CPI is the Cumulative Precipitation Index (cm-days)
SD_o is the initial standard deviation of IRI after treatment for each year during the life span of the treatment (0,99 m/km) in this study
IRI_{pp} is the predicted IRI for the previous year

With data from the Texas Department of Transportation, Dalla Rossa *et al.* (2017) developed a model (shown in Eq. 5.37), with the following predictors: the initial IRI (post construction or treatment) and age, both introduced directly; and the climate, subgrade, treatment type, pavement type , traffic loading and functional system (urban and rural) included by calibration coefficient.

Alaswadko *et al*. (2017) developed roughness prediction models for sealed granular roads at network level in Victoria (Australia) using hierarchical multilevel models which consider the correlation among time series data of the same section and capture the effect of the unobserved effects (Eq. 5.391). Included predicting factors are age of the pavement, traffic loading, initial pavement strength and expansion potential of subgrade soils.

Abdelaziz *et al*. (2018) compared two IRI performance models, one developed by regression analysis and the other one by ANN. With regard to the regression model, the proposed equation is:

$$IRI = IRI_0 + 0{,}014479 \cdot Age + 0{,}00382 \cdot FC_{all} + 0{,}00053 \cdot TC_{all} + 0{,}08941 \cdot SDRUT \quad [8.12]$$

Where

IRI_0 is the initial IRI (m/km)
Age is the age after the construction or overlay for new and overlaid pavement (years)
FC_{all} is all severities of fatigue cracking (percent of wheel path area, %)
TC_{all} all severities of transverse cracks length (m/km)
$SDRUT$ is the standard deviation of rut depth (mm)

With regard to the mechanistic-empirical models, those included in the Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO, 2008; 2010; 2015) are attracting great research attention. For predicting the IRI in new Hot Mix Asphalt and Hot Mix Asphalt overlays of flexible pavements the following predictors are needed (Eq. 5.40): the initial IRI after construction, the area of fatigue cracking, the length of transverse cracking, the average rut depth and a site factor (Eq. 5.43), which depends on the age of the pavement, the percent plasticity index of the soil, the average annual freezing index, the average annual precipitation, and the percent passing the 0,02 and 0,075 mm sieves. For calculating the rut depth, it is required the rate or accumulation of plastic deformation, measured in laboratory test. For cracking data specific data of the materials are needed: the dynamic modulus of the HMA measured in compression, the effective asphalt content by volume, the percent air voids in the mixing, the tensile strain at critical locations, etc.

As seen, several predicting variables are introduced in the proposed deterministic models for IRI performance modelling. They can be gathered in different groups: age, initial IRI, distresses, climate factors, subgrade or soil parameters, traffic, structural parameters and other factors. Table 8.1 presents which groups of factors are included in each of the models described in this section.

Table 8.1. Summary of independent variables considered in models found in the literature

Model	Age	Initial IRI	Distresses	Climate	Soil parameters	Traffic	Structural parameters	Other parameters	Goodness of fit
HDM-III (Paterson, 1987)	Yes	Yes	Rutting, potholes, cracking	Yes	-	Yes	SNCK		
HDM-IV (Odoki and Kerali, 2000, Morosiuk et al., 2004)	Yes	Yes	Rutting, potholes, cracking	Yes	-	Yes	SNP		
Denmark (European Commission, 1997; Saba, 2006)	Yes						Material characteristics		
Finland (European Commision, 1997)		Yes							
Hungary (European Commision, 1997)	Yes					Yes			
Sweden (European Commission, 1997)	Yes			Yes	-		D900, th1	Road width	
PARIS project (European Communities, 1999)	Yes								
Denmark (Saba, 2006)		Yes							
Norway (Sabas, 2006)						Yes			
NordFoU project (Skar et al., 2014)	Yes	Yes	Rutting, potholes, cracking	Yes	-	Yes	SNP		
MDOT (George, 2000) new pavement	Yes					Yes	SN		N = 690, R^2 = 0,35
MDOT (George, 2000) overlaid pavement	Yes					Yes	SN-Thickness		N = 4109, R^2 = 0,48
Dubai (Al-Suleiman and Shiyab, 2003)	Yes								N = 440, R^2 = 0,61-0,80
Choi et al. (2004)					P200	Yes	SN, AC, Thickness		R^2 = 0,714
Brazil (Alburqueque and Núñez, 2011)	-	-	-	Yes		Yes	SN		N = 18-27, R^2 = 0,94 - 0,87
Nigeria (Owolabi et al. 2012)			Longitudinal cracking, potholes, rutting						R^2 = 0,78
Lousiana (Khattak et al. 2014)	Yes	Yes	-	Precipitation, temperature	-	Yes	Thickness		N = 623, R^2 = 0,87
Texas (Dallar Rossa et al. 2017)	Yes	Yes		Yes	Yes	Yes	Treatment type, pavement type	Functional type	
Australia (Alaswadko et al. 2017)	Yes				Yes	Yes	SNC0		R^2 = 0,60
MEPDG (AASHTO, 2008; 2015)	Yes	Yes	Rutting,fatigue cracking, longitudinal cracking	Precipitation, freezing index	Yes		Material characteristics		N = 2439, R^2 = 0,57

8.3. Considered pavement sections and roads for IRI prediction and analysis methodology

8.3.1. Considered pavement sections and roads for IRI prediction

As seen in the previous section, some groups of factors are more widely employed in IRI prediction models. According to Table 8.1, the most widely used factors are the **age of the pavement** (years since construction or the last rehabilitation or maintenance), those related to **traffic volumes** and those regarding **structural parameters**, which include information about the materials included in the pavement section and their properties.

As explained in Chapter 6, the information of the pavement sections of each road is introduced in the PMS by means of projects. Those projects include information about the works performed in the road and these works can be classified as **new outline** or **as rehabilitation and maintenance work**.

If the pavement section described in the Project File is catalogued as new outline, the initial pavement structure is known, with a complete layer description (materials and thickness) and exact date of service starting.

On the contrary, if the pavement section is catalogued as rehabilitation and maintenance works, previous pavement structure must be considered. Sometimes, if that pavement section was previously catalogued as initial pavement, existing structure is known and the complete rehabilitation and maintenance history of the stretch can be observed and analyzed. Nonetheless, if that section was not previously included as initial pavement, only the last layers are known, the superficial one(s), remaining the lower ones unknown. Sometimes, drill core tests are carried out to know the complete pavement section. However, although some conclusions can be derived from in situ observations, drill core tests only represent a point (a transverse section) of the road, and the extension of that structure remains unknown.

When obtaining the Pavement Structure file in Gestivía (the PMS of the RGB) (further described in section 6.5.2.1) of a specific road, this road is divided in multiple stretches according to the known pavement structure (and to the quantity of lane and carriageways), as shown in Table 6.19. In this kind of files, stretches which included the date of initial pavement represent the stretches where all the pavement structure is known, which can also be observed in the available data in the other columns of the provided table.

Therefore, due to the importance of the materials displayed in the pavement structure, their thickness and their properties (reflected in the described deterministic models), it seems reasonable to only analyze pavement sections where the entire layer disposal is known. This idea has previously followed by other road agencies for roughness progression modelling or even for other parameters. In the first attempts of modelling pavement deterioration, carried out by the AASHO Road Test in the late 1950s (AASHO, 1962), completely known pavements structures were measured and analyzed. Later, the Long-Term Pavement Performance (LTPP) programme analyzes pavement section over North America (including the USA and Canada) and extensively data are known (not only materials and thickness of each layer), and it has allowed to produce the

Mechanistic-Empirical Pavement Design Guide (MEPDG) (AASHTO), although models must be calibrated to local condition as they were developed globally (Wasem and Yuang, 2013). Furthermore, Paterson (1987) when developing the first HDM-III models indicated that the roughness progression with time was a complex phenomenon that includes a combination of composite distress that depends on the deformation due to traffic loading and rut depth variation, surface defects from cracking, potholes and patching and a combination of aging and environmental factors. The traffic loads induce levels of stress and strain in the pavement layers depending on the stiffness and layer thickness of the materials.

Consequently, only sections which are completely known are going to be included in the IRI progression calculation. As commented, this type of pavement sections, do provide information about the entire pavement structure, the moment where it was opened to traffic, which is considered more important that the date of finishing since traffic loads start deteriorating it when it is opened to traffic, because work traffic can be discarded as negligible. From the contrary point of view, it seems not reasonable to analyze, for example, pavement section where only the last 5 cm of the pavement are identified. The underlying materials are completely unknown and, although for the computation similar data would be introduced, different evolutions could be achieved if very different pavement structures are placed underneath.

The decision of only analyzing pavement stretches with an entire pavement section in the database leads to discard the local network level from this analysis. When introducing data in the PMS, projects of the roads included in the preferential interest (red), basic (orange), complementary (blue) and provincial (green) networks (Table 6.3) were introduced with the detail described in Chapter 6. Road of the local network were not analyzed so deeply due to their low traffic volumes and importance for the global performance of the road network. Although the local network represents the 46,2 % of the total length of the network managed by the RGB (Fig. 8.1), the mobility through that network only represents the 6,5 % of the total (Fig. 8.2) (Pérez-Acebo, 2018). Pavement Structure files can be also obtained from the Pavement Management System (Gestivía) but very few stretches included initial pavement sections, as an exhaustive investigation of projects conducted in each local road was not carried out. In the Regional Government of Biscay, in the Transport Department, it was decided to give priority and funds for the development of a complete pavement management system for the important road network levels.

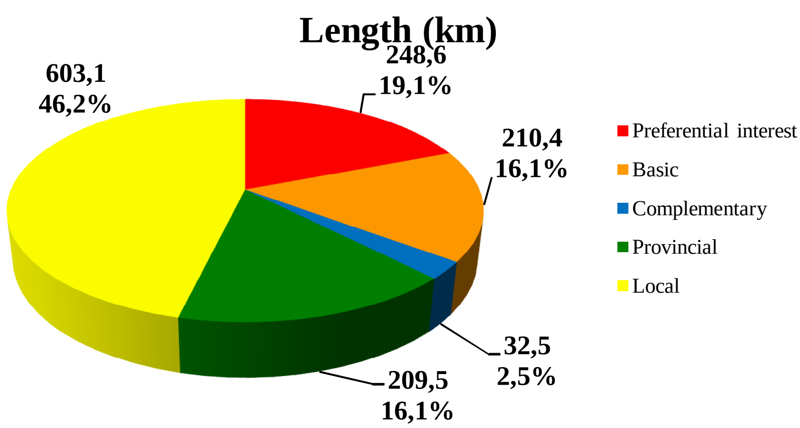


Fig. 8.1. Length of the road networks managed by Regional Government of Biscay in 2016 (Diputación Foral de Bizkaia, 2017)

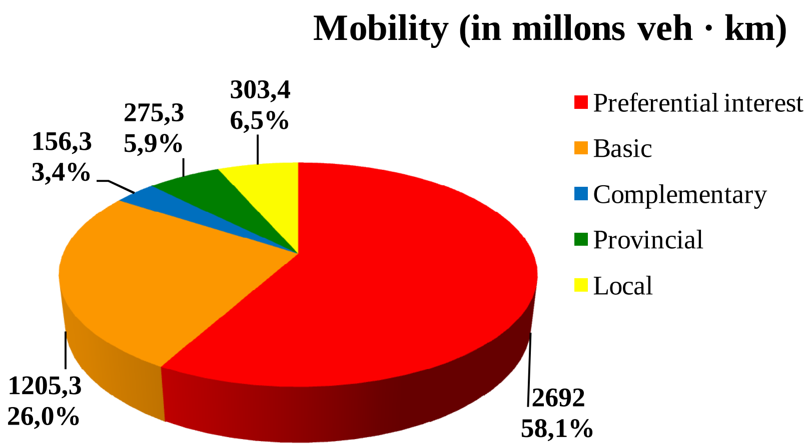


Fig. 8.2. Mobility in the road networks managed by Regional Government of Biscay in 2016 (Diputación Foral de Bizkaia, 2017)

Consequently, although almost half of the road network is discarded, the 93,5 % of the mobility is carried out through the analyzed road networks.

Roads included in each network level are shown in Annex I, with additional data:

5) Present and previous code, the denomination and the location of the stretch.
6) The type of road: paying motorway (*autopista de peaje*), not paying motorway (*autopista*), motorway (*autovía*), multilane highway (*carretera multicarril*) and conventional road (*carretera convencional*) according to Spanish Law of Roads (BOE, 2015).
7) The length of each type of road in each road (in case various types exist in the same road)
8) The initial and final Kilometre Post of each type of road in each road.

8.3.2. Analysis methodology

Once that the roads that could be analyzed were established and the pavement sections that could be introduced in the IRI performance model were determined, the individual analysis of each road was carried out. To this aim, an exhaustive investigation of all the projects conducted in each road of the selected network levels was performed.

As commented previously, the Pavement Structure file provides information about the stretches where the entire pavement section is known, indicated as Initial Pavement and with the date when it was opened to traffic, and also the present pavement section, with the materials and the thickness of each layer. However, as mentioned in the deficiencies of this file, it does not show the intermediate and maintenance activities that were conducted to arrive to the present section. Intermediate projects are not indicated, although they are included in the history of that section to show the final (present) structure.

Consequently, a deeper research must be carried out, apart from observing the Pavement Structure file. In order to prepare this PhD thesis, it was given access to the draft schemes employed by engineers of the UTE Agenda de Estado (the union of the three engineering companies that incorporated all the data to the Gestivía software). In this preliminary file, engineers responsible of the introduction of projects in the Pavement

Management System, indicated notes and provisional schemes in maps. Fig. 8.3 and Fig. 8.4 show an example of this preliminary scheme for the road BI-631 (stretch Mungia – Bermeo). Using a map of the road, projects found in the Archives of the Regional Government of Biscay are marked in their place along the road. The date of opening to traffic, the complete pavement structure (if the project is considered as introducing a “New Outline”) or the layers extended (in case of a rehabilitation and maintenance) and the initial and final Maker Post are noted. The subsequent projects are also indicated, with introduced new layers, date and the stretch where the project is conducted. Different colours are applied to underline the projects allowing a quick view of the lengths affected.

It must be very careful when analyzing the stretches. A new pavement can be conducted in a specific length. However, rehabilitation or maintenance projects are not carried out on that entire length, but only on part of it. Then, a new project may be performed affecting a part of the initial stretch that was not affect by the previous project and a part of the previously affected stretch. Therefore, the initial stretch of initial pavement may be divided in 4 parts with varying pavement structure (Fig. 8.5.).

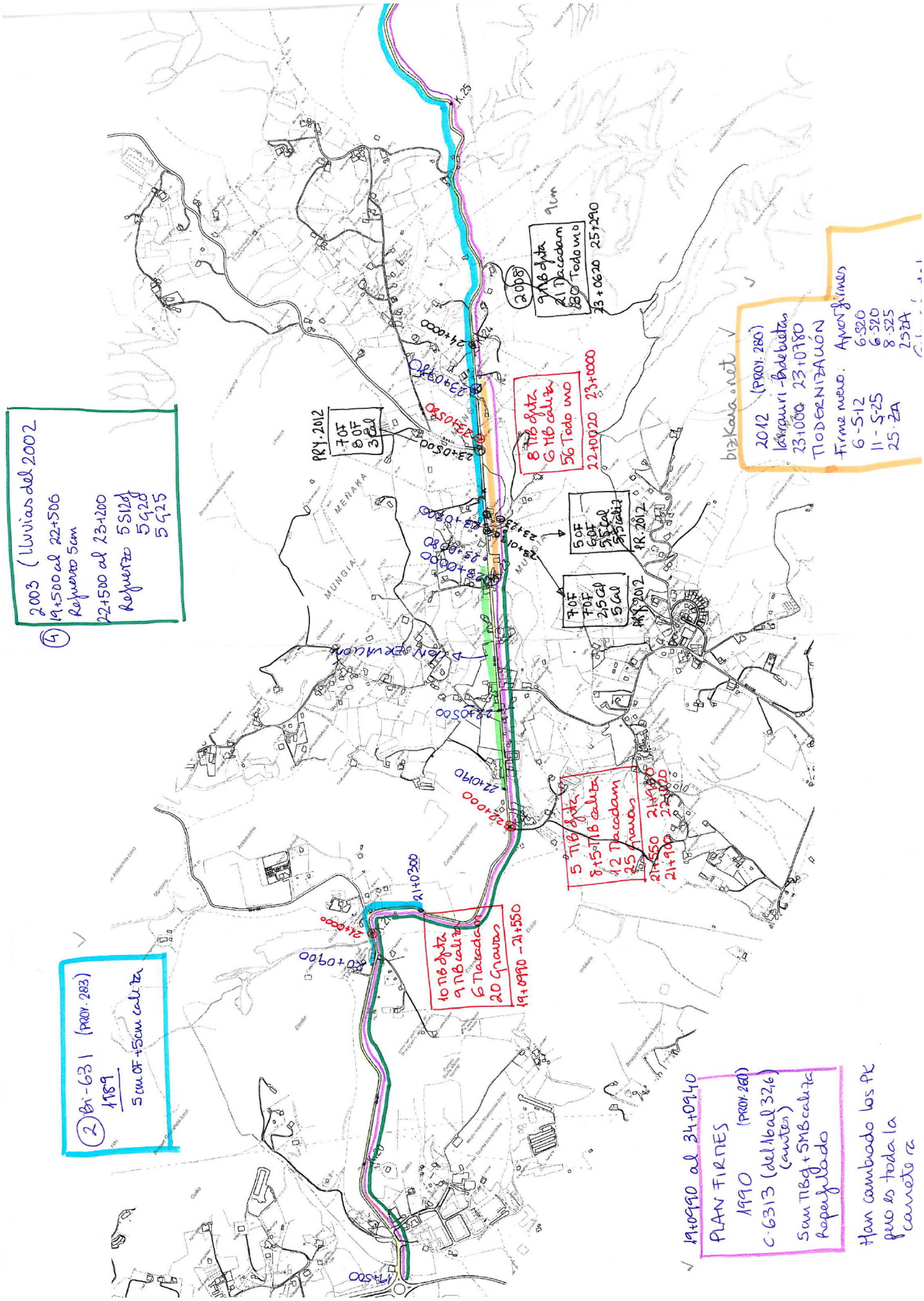


Fig. 8.3. Preliminary scheme of the road BI-631 (stretch Mungia – Bermeo) used by engineers of the UTE Agenda de Estado to observe all the projects conducted in the road.

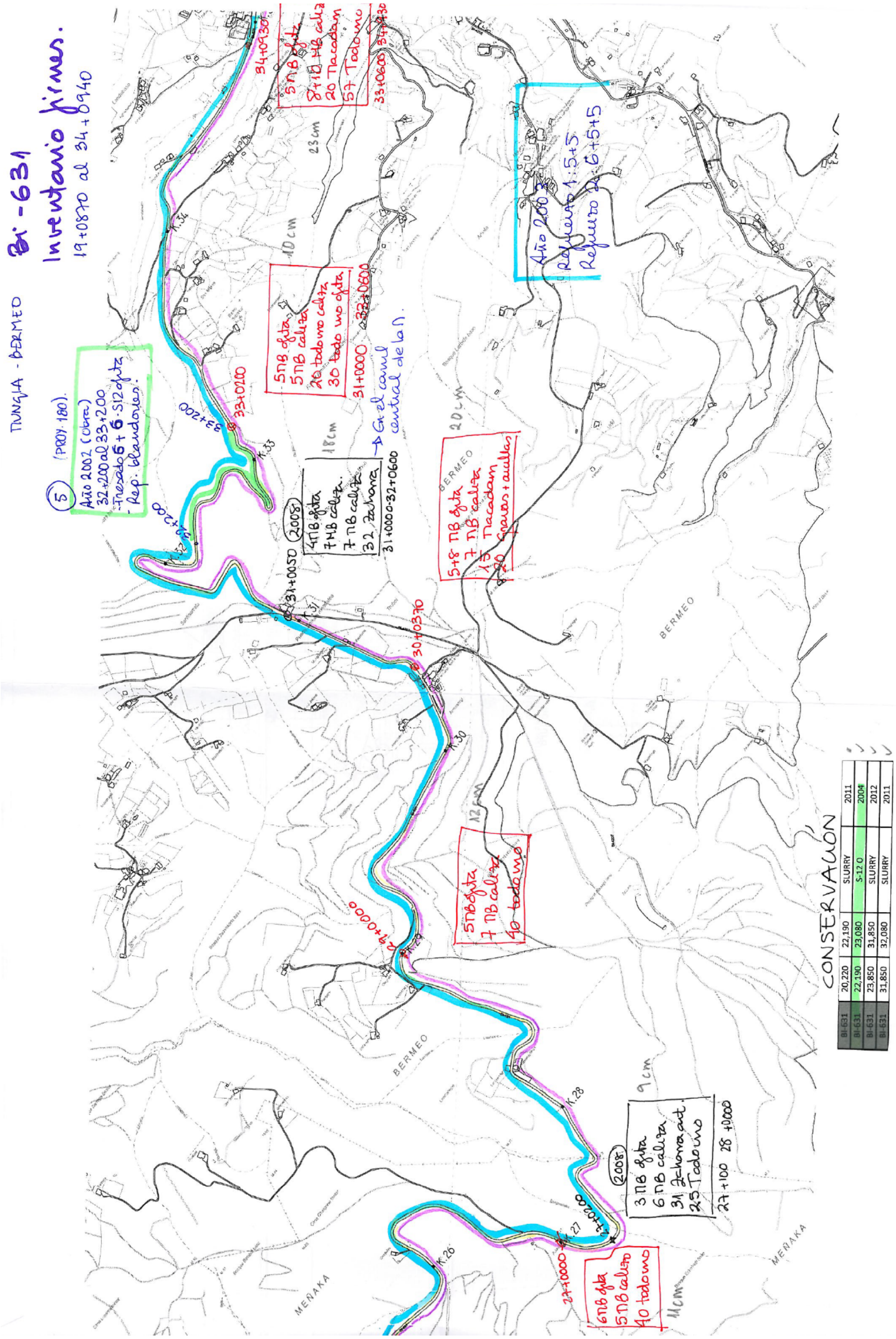


Fig. 8.4. Preliminary scheme of the road BI-631 (stretch Mungia – Bermeo) used by engineers of the UTE Agenda de Estado to observe all the projects conducted in the road (continuation).

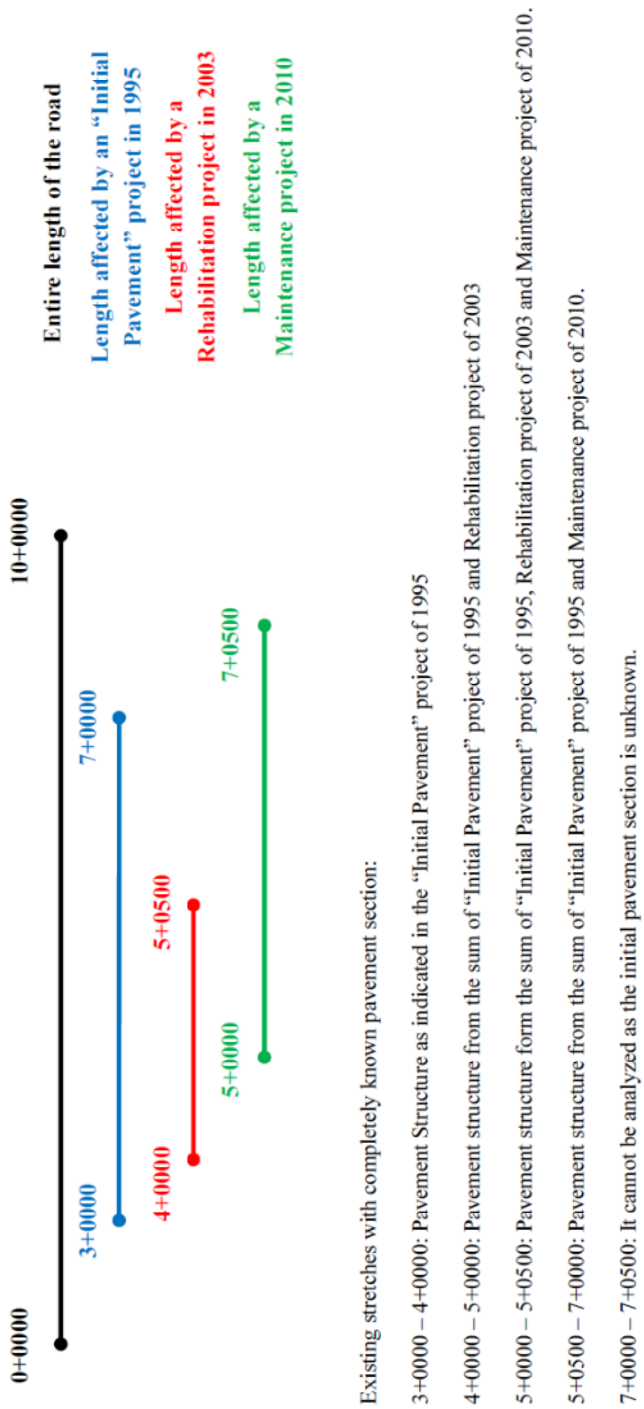


Fig. 8.511. Possible projects conducted in a road, and obtained different sections to analyze.

As commented, the draft schemes of the roads provide the clues to know the projects conducted in every road. However, those projects must be verified in the list of projects of Gestivía. First of all, it must be verified that the project was really conducted, as sometimes, a project can be ready for construction but it was not finally performed by any reason (lack of funds, substituted by another project, etc.). Then, the initial and final marker posts are checked, the type of pavement action (New outline, providing a complete pavement structure, or a Maintenance and Rehabilitation, M&R work) and the materials and thickness of the layers introduced in the database. Sometimes, values in the provisional schemes (Fig. 8.3 and 8.4) may differ from the real data included in Gestivía.

When the complete analysis of the projects conducted in each road is done, a Road file is prepared for every road in Conservation Areas 1, 2 and 3, as shown in Annex II, where individual Road files for every road are included. For each road, a predetermined file (Table 8.2) is filled, where the following information is provided:

- **Road Denomination:** the code of the road.
- **Itinerary:** The name of the road or the cities that connect
- **Marker post (KP) (Kilometric Post):** The initial and final Kilometric Post of the stretches managed by the Regional Government of Biscay are indicated.
- **Transferred stretches:** As commented, sometimes, some stretches of a road are transferred to other administration, as in the case a road crosses a village and, due to the urban nature of the stretch; it has been transferred to the local administration (town council).
- **General comments:** A brief history of the road is provided. It indicates if the roads existed before it was transferred to the Regional Government of Biscay (1983), or it has been constructed by the RGB. Indications of main projects, especially those with “Initial Pavements” are included.
- **Main actuations:** The main projects conducted in the road are displayed, ordered by date, indicating the code of Gestivía, the date when it was opened to traffic, the stretch affect (by means of Kilometric Posts) and the layers included in the projects.
- **IRI:** data from IRI are commented year by year. It shows the stretches where IRI data are used for calculation (as they come from sections with entire known structure) and the average IRI value for the considered stretch. Some notes are included in case of spurious data are observed. For example, in the case of the stretch 3+0000 to 4+0000 of the Fig. 8.5, where no projects have been conducted since 1995, if IRI data are observed to improved from 2007 to 2011 (a great improvement), it can be regarded as spurious data since some works were done in that part but the project was not included in the database.
- **Skid resistance:** Skid resistance data from 2011 and 2016 are commented. Further explanation is provided in Chapter 9.
- **Skid resistance of surface layers in 2016:** Further explanation is provided in Chapter 9.

Hence, a complete explanation of the stretches of every road included in the calculation is provided and, in case of strange results, it is possible to revise the origin of each data, individually analyze, and in case of doubts about the origin or the values, reject those data as spurious. Moreover, reasons for discarding some data are also noted in the Road files of Annex II.

Table 8.2. Example of the Road file for road BI-712, included in Annex II.

Road Denomination	**BI-712**	
Itinerary	Basauri - Bolueta	
Marker post (KP)	**From:** 387+0680	**To:** 388+0550
Transferred stretches		
382+0530 – 384+0350: Intersection with BI-625 – Basauri 384+0350 – 387+0680: San Miguel and Basauri 388+0550 – 389+0050: Bolueta		
General comments:		
This road was a part of the N-625. Nowadays it is complementary road, with the main part crosses urban areas and it is transferred to municipalities. In the part managed by the Regional Government of Biscay, the oldest project conserved is from 1993 and detailed the different bituminous layers extended over existing pavements (PROJ-400). There is a maintenance work registered in 2007 in the entire road, where slurry was extended (PROJ-2008). Later, in 2008 a layer of 5 cm of S-12 was extended in 388+0240 – 388+0420 (PROJ-2009) and in 2015. A layer of 6 cm of S-12 was extended in 388+0100 – 388+0240 (PROJ-1756).		
Main actuations:		
1993: PROJ-400 (01/06/1993) 387+0680 – 388+0100: AC 16 surf S 5 cm + AC 22 bin S 7 cm + AC 22 base G 12 cm + MG (ZA) 25 cm 388+0100 – 388+0550: AC 16 surf S2 5 cm + G20 7 cm 2007: PROY-2008 (01/06/2007) 387+0690 – 388+0550: LB2 2008: PROY-2009 (01/06/2008) 388+0240 – 388+0480: AC 16 surf S 5 cm 2015: PROY-1756 (30/10/2015) 388+0110 – 388+0240: AC 16 surf S 6 cm		
IRI:		
2000:		
Stretch 387+0680 – 388+0100: IRIm = 2,47.		
2004:		
Stretch 387+0680 – 388+0100: IRIm = 2,06. Sections with improvements are discarded. New IRIm = 2,59		
2007:		
Stretch 387+0680 – 388+0100: IRIm = 2,14 (After maintenance work)		
2011:		
Stretch 387+0680 – 388+0100: IRIm = 2,16		
2016:		
Stretch 387+0680 – 388+0100: IRIm = 2,62		
Skid resistance:		
2011:		
Stretch 387+0680 – 388+0100.		
2016:		
Stretch 387+0680 – 388+0100.		
Skid resistance of surface layers in 2016		
2016		
388+0100 – 388+0240: PROJ-1756 (30/10/2015) AC 16 surf S 6 cm 388+0240 – 388+0480: PROJ-2009 (01/06/2008) AC 16 surf S 5 cm 388+0480 – 388+0550: PROJ-2008 (01/06/2007) LB2		

8.4. Family models for IRI prediction

As observed in the models found in the literature, different models are proposed for roughness progression if the pavement section is new or it has been rehabilitated or maintained (AASHTO, 2008; 2015, George, 2000). It is logical to assume that different performances are observed if the pavement is completely new or has been partially rehabilitated. Despite removing the surface layer (and even more layers) and substituting it (or them) with the same material and thickness, a worse deterioration is registered since the remaining layers have withstood previous traffic loads. This fact can be observed in several models (European Commission, 1997; AASHTO, 2008; 2015, George, 2000; Busch *et al.*, 2010).

Therefore, it was decided to divide the models for predicting IRI evolution in two family models. On one hand, pavement classified as **New Outline**, and on the other hand, pavement classified as **Maintenance and Rehabilitation**, (**M&R**).

A pavement is classified as **New Outline** when the complete pavement structure is known and no rehabilitation and maintenance activities have been carried out. A complete section is known when the bituminous layers and the base and subbase layers (composed by unbound materials or cement-treated materials) are known, with the used materials in each layer and their thickness. No rigid pavements can be found in Biscay under the management of the RGB.

Data about the subgrade has not been considered necessary to be known, as very few projects provide some information about the subgrade. The only information that can be found in some projects is the subgrade type; E1, E2 or E3, according to the Spanish laws for pavement design (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b), and in very few cases, the layers included to create that subgrade type is registered.

Most of the times, stretches classified as New Outline come from real new outlines of roads, as by-passes of roads to avoid some cities or villages. In these cases, it is obvious that a new outline was conducted when no road existed. In few cases, a New Outline follows an existing path of a road. In this case, a complete reconstruction of the pavement section must be carried out, including both bituminous and base/subbase layers. Moreover, the section is included as “Initial Pavement” in the database and it is indicated that way in the Pavement Structure file.

A pavement is classified as **Maintained and Rehabilitated** (M&R) when additional bituminous material, in one or more layers, is extended over existing pavement. Some layers can be reclaimed before the extension, but an M&R activity is considered to be performed when at least a new layer of bituminous material, regardless of its thickness, is extended.

There is no definition to distinguish between rehabilitation and maintenance works with regard to the minimum or maximum thickness of new layers. Rehabilitation activities are generally related to structural problems of the pavement and they aim to recover the initial structural capacity by extending additional layers, usually more than one. For examples, the Spanish guide for Pavement rehabilitation (Ministerio de Fomento, 2003a) indicates that a range of 5-18 cm must be extended to improve the structural capacity of pavement when deflections are important. On the contrary, maintenance works are generally related to improve superficial characteristics of the pavement, mainly skid resistance and sometimes, roughness. They usually include a unique new layer, as surface characteristics are only related to the wearing course (surface layer). The Spanish laws for pavement design and for controlling the quality of the materials (PG-3) (Ministerio de Fomento, 2003b; 2015) indicate that for surface layers, the following thickness must be employed, according to the material (Table 8.3).

Table 8.3. Thickness of surface layers in bituminous pavements according to material types in Spanish regulations (Ministerio de Fomento, 2003b; 2015).

Material type	Denominations	Thickness (cm)
Asphalt Concrete	AC16 surf S, AC16 surf D	5
	AC22 surf S, AC22 surf D	6
Discontinuous mixes	BBTM 11A, BBTM 11B, BBTM 8A, BBTM 8B	2-3
Porous asphalt	PA 11, PA 16	4
Slurry	MICROF 11, MICROF 8, MICROF 5 (previously LB1, LB2, LB3, LB4)	< 1,5

Usually, the application of slurries is associated to maintenance activities, as the skid resistance and texture are wanted to be restored. In the next subsections, information and data included in each family model are described.

8.4.1. Data introduced in all types of pavements

For every road section, the following information is included for the analysis (Table 8.4), including two stretches of the road BI-631 in Conservation Area 1, as example)

- *Column A*: **Road denomination**: code of the road
- *Column B*, **Axle**: Main carriageway, in conventional roads, or ascending or descending carriageway in motorways and multilane highways. It could be referred to ramps, but there are not IRI data in ramps.
- *Column C*, **Initial KP**: the initial Kilometre Post of the stretch
- *Column D*, **Final KP:** the final Kilometre Post of the stretch
- *Column E*, **Exact date of data collection**: if know, the exacta data of the data collection is indicated. If the exact date is not available, information from engineers of the RGB is indicated, usually in summer
- *Column F*, **Data collection year**: Year when the IRI data collection was conducted.
- *Column G*, **Data collection year, in number**. The date of the data collection is quantified numerically. For example, if IRI data are collected in July 2004, that represents the half of the year 2004 and 2004,5 is indicated. It is used for calculating the real age of the pavement.
- *Column H*, **IRI value**. The value of the IRI collected in that stretch, according to the initial and final points of the established measuring stretch, as described in Table 6.21. Values for the right and left data are registered in the database, so two rows are created with the same data for the rest of columns except for the IRI data.

Table 8.4. Data introduced for every road stretch in the IRI analysis (with example)

A	B	C	D	E	F	G	H
Road denom	**Axle**	**Initial KP**	**Final KP**	**Exact date data collection**	**Data collection year**	**Data collection year, in number**	**IRI**
BI-631	BI-631 Unique	34+0710	34+0810	03/07/2016	2016	2016,5	1,96
BI-631	BI-631 Unique	34+0710	34+0810	03/07/2016	2016	2016,5	1,93
BI-631	BI-631 Unique	34+0810	34+0910	03/07/2016	2016	2016,5	1,18
BI-631	BI-631 Unique	34+0810	34+0910	03/07/2016	2016	2016,5	1,48

Additionally, for every stretch, both classified as New Outline or as Maintenance and Rehabilitation (M&R), the following traffic data are included (Table 8.5, which includes two stretches of the road BI-631 in Conservation Area 1, as example):

Table 8.5. Data introduced for every road stretch in the IRI analysis about traffic (with example).

T1	T2	T3	T4	T5
AADT of data collection year	**H.AADT of data collection year**	**Traffic category of data collection year**	**Total vehicles**	**Total heavy vehicles**
5.314	159	T31	2.394.000	74.866
5.314	159	T31	2.394.000	74.866

- *Column T1*, **AADT of data collection year**, the Annual Average Daily Traffic registered in the year of the data collection. It indicates the total mean traffic volume per day, considering both directions of the road, both in two-lane roads and in double carriageway motorways.
- *Column T2*, **H.AADT of data collection year**, the Annual Average Daily Traffic of heavy vehicles. A heavy vehicles is considered when its total weight is over 3.500 kg (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b). It indicates the AADT of heavy vehicles in the project lane, i.e., the lane with the highest heavy traffic. Generally, it is unknown which is the project lane and hence, it is established according to Ministerio de Obras Públicas y Urbanismo (1989a) and Ministerio de Fomento (2003b). In a two-lane road, the heavy traffic is divided equally in both directions, i.e., each lane is considered to have the half of the heavy traffic (50% - 50%). In a double carriageway highway, with two lanes in each carriageway for each direction, the external lane (the one on the right) has the total heavy traffic of that direction. If the value of the two carriageway is provided (as usual in the data of the RGB), every right lane of each carriageway receives the half of the heavy traffic. In a double carriageway highway, with three or more lane in each carriageway for each direction, the external lane (the one on the right) has the 85 % of the heavy traffic of that direction. Once again, if the value of the two carriageway is provided (as usual in the data of the RGB), every carriageway receives the half of the heavy traffic, and then, the line on the right gets the %85 of that traffic in that direction.
- *Column T3*, **Traffic category of the data collection year**, it indicates the traffic category of the road according to Ministerio de Fomento (2003b) in the year of the data collection. It is established according to Table 8.6.

Table 8.6. Heavy traffic categories in Spain (Ministerio de Fomento, 2003b).

Traffic category	H.AADT (heavy veh./day/lane)	Traffic category	H.AADT (heavy veh./day/lane)
T00	H.AADT ≥ 4.000	**T31**	200 < H.AADT ≤ 100
T0	4.000 < H.AADT ≤ 2.000	**T32**	100 < H.AADT ≤ 50
T1	2.000 < H.AADT ≤ 800	**T41**	50 < H.AADT ≤ 25
T2	800 < H.AADT ≤ 200	**T42**	25 < H.AADT

- *Column T4*, **total vehicles**, the accumulated total number of vehicles that crossed the section since the date in which the stretch was opened to traffic (in the case of New Outline stretches), or since the date in which the Maintenance and Rehabilitation activity was finished (in the case of M&R stretches) until the date of the data collection. As indicated in Chapter 7, from 2000, AADT, percentage of heavy vehicles, H.AADT and traffic category are included in the database. Data from previous years, before 2000, can be found in previous documents of the RGB. However, when data before 2000 are required it was supposed an annual increasing rate of the traffic volume of 2 %, which has been demonstrated to be a reasonable value for the decade of the 90s (Departamento de Vivienda, Obras Públicas y Transportes, 2012; Pérez-Acebo, 2018). As data of AADT are employed, this column indicates the total vehicles that crossed the section in both directions.
- *Column T5*, **total heavy vehicles**, the accumulated total number of heavy vehicles (vehicles with a total weight over 3.500 kg) that have crossed the section since the date of opening or rehabilitation (or maintenance) of the stretch until the date of the data collection. The considerations indicated for Column T4 about increasing rate for data before 2000 are also considered. Since data of H.AADT are employed, this column shows the total vehicles that crossed the section through the project lane, the lane with the highest quantity of heavy vehicles, with the assumptions of Column T2.

8.4.2. Data introduced in pavements classified as New Outline

The following data is added to the previous information in stretches classified as New Outline (Table 8.7, which includes two stretches of the road BI-631 in Conservation Area 1, as example).

- *Column I*, **date of opening**, the exact date when the road is open to traffic.
- *Column J*, **year of opening**, the year of the opening to traffic
- *Column K*, **year of opening, in number**, the quantification of the date, as explained for column G.
- *Column L*, **age,** the pavement age is calculated by subtracting column J to column F (Column L = Column F – Column J). It refers to the difference between natural year of data collection and opening to traffic.
- *Column M*, **real age**, a more accurate estimation of pavement age is obtained by subtracting column K to column G (Column M = column G – Column K). It indicates the real age of the pavement, expressed in years, in decimal fraction, where 0,5 means 6 months.
- *Column N*, **description**, the complete pavement section is described schematically, indicating the material of each layer and its thickness, starting from the surface layer and finishing in the last layer

of the subbase, the one above the subgrade. The subgrade is not introduced.

- *Column O*, **pavement type**, according to the description of previous column, the pavement is classified as flexible or semi-rigid. As commented through the thesis, no rigid pavements can be found in the network managed by the RGB: When cement or slag was employed to treat the subbase, a semi-rigid pavement is considered. Materials that can be found in semi-rigid pavements are: soil cement (*suelocemento*, in Spanish), gravel and cement (*gravacemento*), gravel and slag (*gravaescoria*) or vitrified slag (*escoria vitrificada*). In flexible pavements, the subbase is composed of unbound material, generally, crushed stone (*zahorra*) or gravel. It can be noted that the Guide for the updating of the pavement database of the State Road Network (Ministerio de Fomento, 2011a) and the Pavement Rehabilitation Guide (Ministerio de Fomento, 2003a) define a flexible pavement as the one with less than 15 cm of bituminous layers and define a semi-flexible pavement as the one with 15 cm or more of bituminous layers. This last differentiation was not considered important and pavements have been only classified as flexible or semi-rigid according to the base.
- *Column P*, **total thickness of the bituminous layers (cm)**, the thickness of all the bituminous layers that compose the pavement section, in cm.
- *Column Q*, **surface layer thickness (cm)**, the thickness of the surface layer of the pavement section, in cm.
- *Column R*, **surface layer material**, the material employed in the surface layer.
- *Column S*, **binder layer thickness (cm)**, the thickness of the bituminous binder layer of the pavement section, in cm.
- *Column T*, **binder layer material**, the bituminous material employed in the binder layer.
- Column U, **base layer thickness (cm)**, the thickness of the bituminous base layer of the pavement section, expressed in cm.
- *Column V*, **base layer material**, the bituminous material employed in the base layer. If there are only two bituminous layers in the section, they are introduced as surface and binder layer and hence, there is no base layer. If only one bituminous layer is present in the pavement structure, it is introduced as surface layer, and no data are introduced in binder and base layer.
- *Column W*, **thickness of the 1st subbase layer**, the thickness of the first subbase layer of the pavement section, expressed in cm. Generally, not bituminous layers are referred in the literature as base and subbase layers. However, in order to be distinguished from the 3rd bituminous layer, it has been preferred to call the first non bituminous layer of the section as the first subbase.
- *Column X*, **1st subbase layer material**, the non bituminous material employed in the first subbase layer.
- *Column Y*, **thickness of the 2nd subbase layer**, the thickness of the second subbase layer of the pavement section, expressed in cm. Normally, only one subbase is included, but in case a second non bituminous layer is displayed, it corresponds to the layer which is directly placed over the subgrade.
- *Column Z*, **2nd subbase layer material**, the non bituminous material employed in the second subbase layer.

Table 8.7. Data introduce in stretches classified as New Outline.

I	J	K	L	M	N	O
Date of opening	**Year of opening**	**Year of opening, in number**	**Age**	**Real Age**	**Description**	**Pavement type**
01/05/2015	2015	2015,3	1	1,2	BBTM11A 3cm + AC 16 bin S 8 cm + GC 29 cm	Semi-rigid
01/05/2015	2015	2015,3	1	1,2	BBTM11A 3cm + AC 16 bin S 8 cm + GC 29 cm	Semi-rigid

P	Q	R	S	T	U	V
Total Bituminous thickness (cm)	**Surface layer thickness (cm)**	**Surface layer material**	**Binder layer thickness (cm)**	**Binder layer material**	**Bituminous base layer thickness (cm)**	**Bituminous base layer material**
11	3	BBTM11A	8	AC 16 base S	-	-
11	3	BBTM11A	8	AC 16 base S	-	-

W	X	Y	Z
Thickness of 1st subbase layer (cm)	**1st subbase layer material**	**Thickness of 2nd subbase layer (cm)**	**2nd subbase layer material**
29	Cement Treated base (Gravel cement)		
29	Cement-Treated base (Gravel cement)		

8.4.3. Data introduced in pavements classified as Maintenance and Rehabilitation (M&R)

Apart from the identification information of the stretch, which is introduced as indicated in section 8.4.1 in Columns A to H and the traffic data (Columns T1 to T5), the following data are incorporated to the modelling in M&R stretches (Table 8.8, including two stretches of the road BI-634 in Conservation Area 1 and road BI-633 in C. Area 2, as example)

- *Column I*, **date of M&R work**, the exact date when the M&R work is finished.
- *Column J*, **year of M&R work**, the year when M&R work is finished.
- *Column K*, **year of M&R work, in number**, the quantification of the date, as explained for column G for all type of stretch.
- *Column L*, **age,** the age of the M&R work is calculated by subtracting column J to column F (*Column L = Column F – Column J*). It refers to the difference between natural year of data collection and when the M&R work finished.
- *Column M*, **real age**, a more accurate estimation of age of the M&R work is obtained by subtracting column K to column G (*Column M = Column G – Column K*). It indicates the real age of the M&R activity, expressed in years, in decimal fraction, where 0,5 means 6 months.
- *Column N*, **M&R work description**, the Maintenance or Rehabilitation activity that is carried is described schematically, indicating the material of each layer and its thickness, starting from the surface layer. It must indicate if some material is removed (milling), to know the exact final structure and thickness of the materials.

- *Column O,* **total thickness of new bituminous layers (cm)**, the thickness of the bituminous layers that are extended over existing layers (after some layers are removed, or not), in cm.
- *Column P,* **new surface layer thickness (cm)**, the thickness of the surface layer extended over existing layers (after some layers are removed, or not), in cm. When extending a layer of slurry, the thickness indicated is 0,5. Although the maximum thickness is 1,5 cm, as indicated in Table 8.3, the Pavement Management System of the RGB, does not consider it as an increment of the total thickness. It computes the new surface layer as Slurry (LB2), but if only slurry is extended over existing pavement, the same thickness is indicated, but with a slurry surface.
- *Column Q,* **new surface layer material**, the material employed in the surface layer extended over existing layers.
- *Column R,* **existing pavement description**, the existing pavement section, before the M&R work is carried out is described schematically.
- *Column S*, **pavement type**, according to the description of the previous column, the pavement is classified as flexible or semi-rigid, following the criteria indicated in Column O for New Outline stretches.
- *Column T,* **date of existing pavement**, the date when the existing pavement was carried out. It may come from a New Outline stretch and hence, the date of opening to traffic is included, or the date of the last M&R work conducted in that pavement section.
- *Column U*, **year of existing pavement**, the year of the date of the previous column.
- *Column V*, **year of existing pavement, in number**, the quantification of the date as explained for Column G for all the pavement families.
- *Column W*, **age of the previous work**, the age of the last work in that stretch (both a new construction or a M&R work) is calculated by subtracting column U to column J (*Column W* = *Column J – Column U*). It indicates the period between the last two actuations in that stretch.
- *Column X,* **real age of the previous work**, a more accurate estimation of the age of the last work in the stretch (both a new construction or a M&R work) is calculated by subtracting *column V* to *column K* (*Column X* = *Column K – Column V*). It indicates the real age of the last two actuations in that stretch, in decimal fraction, where 0,5 means 6 months.
- *Column Y,* **previous work**, the nature of the previous work, distinguishing between New Outline or M&R work.
- *Column Z,* **history of works**, it is shown the complete history of actuation and works in that stretch, including the one that is being defined and the previous one.
- *Column AA,* **total thickness of bituminous layers in existing pavement (cm)**, the thickness of all the bituminous layers that compose the existing pavement section before the M&R work, in cm.
- *Column AB,* **total thickness of subbase layers in existing pavement (cm)**, the thickness of all the non bituminous layers that compose the existing pavement section before the M&R work, in cm. It includes the two layers that may exist in the subbase.
- *Column AC,* **subbase materials in existing pavement**, the subbase materials in existing pavement section before the M&R work. It can indicate two different materials if two layers are present in existing pavement.

- *Column AD*, **description of new pavement section**, it describes the resulting pavement section after the M&R work over the existing pavement, taking into account the existing layers, the ones that can be removed and the new ones.
- *Column AE*, **total thickness of bituminous layer in new pavement (cm)**, the total thickness of all the bituminous layers that compose the new pavement section, in cm.
- *Column AF*, **surface layer thickness of new pavement section,** the thickness of the surface layer of the new pavement section, in cm. If the a slurry is extended, it does not indicate 0,5 cm as introduced in *column O* and *P*, but the thickness of the layer below the slurry.
- *Column AG*, **surface layer thickness of new pavement section**, the material of the surface layer in the new pavement section. If slurry is extended, as explained in previous column, it is indicated as Slurry, although it considers the thickness of the layer below it.
- *Column AH*, **binder layer thickness in new pavement (cm)**, the thickness of the bituminous binder layer of the new pavement section, in cm. If Slurry is displayed in the surface layer, the thickness of the binder layer is included as surface layer thickness. Therefore, in this case, binder layer represents the 3rd bituminous layer, after Slurry and another one.
- *Column AI*, **binder layer material in new pavement**, the bituminous material employed in the binder layer in the new pavement section, taking into account the indications about slurry in surface layer.
- *Column AJ*, **base layer thickness in new pavement (cm)**, the thickness of the bituminous base layer of the new pavement section, in cm, taking into account the possibility of slurry in surface layer.
- *Column AK*, **base layer material in new pavement**, the bituminous material employed in the base layer in the new pavement section. If slurry is present in surface layer, it refers to the 4th bituminous layer, if it exists.
- *Column AL*, **total thickness of subbase layers (cm)**, the total thickness of the subbase layer(s). It includes the two layers that may exist in the subbase of the new pavement section.
- *Column AM*, **subbase layer materials**, the subbase materials in existing pavement section before the M&R work. It can indicate two different materials if two layers are present in the new pavement.

Table 8.8. Data introduced in stretches classified as Maintenance and Rehabilitation (M&R).

I	J	K	L	M	N	O	P	Q
Date of M&R work	**Year of M&R work**	**Year of M&R work, in number**	**Age**	**Real age**	**Description**	**Total thickness of new bituminous layers (cm)**	**New surface layer thickness (cm)**	**New surface layer material**
01/06/2010	2010	2010,5	6	6	BBTMA 11A 3 cm + AC 16 bin S 5 cm	8	3	BBTM 11A
01/08/2014	2014	2014,6	2	1,9	Slurry	0,5*	0,5*	LB2

R	S	T	U	V	W	X
Existing pavement description	**Pavement type**	**Date of existing pavement**	**Year of existing pavement**	**Year of existing pavement, in number**	**Age of the previous work**	**Real age of the previous work**
AC 22 surf S 6 cm + AC 22 base G 11 cm + Unbound mat 25 cm	Flexible	01/06/1992	1992	1992,5	18	18
AC 16 surf S 5 cm + AC 16 surf S 6cm + AC 22 base G 11 cm + Unbound mat 25 cm	Flexible	01/01/2012	2012	2012	2	2,6

Y	Z	AA	AB	AC	AD
Previous work	**History of work**	**Total thickness of bituminous layers in existing pavement (cm)**	**Total thickness of subbase layers in existing pavement (cm)**	**Subbase materials in existing pavement**	**Description of new pavement section**
M&R	New outline – M&R – M&R	17	25	Unbound material (ZA)	BBTM 11A 3 cm + AC 16 bin S 5 cm + AC 22 surf S 6 cm + AC 22 base G 11 cm + Unbound mat 25 cm
New Outline	New outline – M&R	22	25	Unbound material (ZA	LB2 + AC 16 surf S 5 cm + AC 16 surf S 6cm + AC 22 base G 11 cm Unbound mat 25 cm

AE	AF	AG	AH	AI	AJ	AK	AL	AM
Total thickness of new bituminous layers (cm)	**Surface layer thickness of new pavement**	**Surface layer material in new pavement**	**Binder layer thickness in new pavement (cm)**	**Binder layer material in new pavement**	**Base layer thickness in new pavement (cm)**	**Base layer material in new pavement**	**Total thickness of subbase layers (cm)**	**Subbase layer materials**
25	3	BBTM 11A	5	AC 16 bin S	17	AC 22 surf S/AC 22 base G	25	MG (ZA)
22	5	LB2	6	AC 16 surf S	11	AC 22 base G	25	MG (ZA)

8.4.4. Treatment of data and available observations for each family

As commented in section 8.3.1, roads from the local network (yellow network) are not considered for IRI performance modelling because this type of road was not so exhaustively examined as the other levels due to their low importance. Efforts from engineers of the Regional Government of the Biscay were oriented to the most important network levels: the preferential interest network (red network), basic network (orange network), complementary network (blue network) and provincial network (green network) (Table 6.3).

Hence, although the 46,2 % of the total length is not included in the analysis, studied ntwork represent the 93,5 % of the mobility in the territory of Biscay, and hence, all the main roads are included.

With regard to the type of roads to be examined for IRI modelling, it was established that only two-lane roads (conventional roads, following the denomination of the Spanish Law of Roads (BOE, 2015)). In a double carriageway, as commented in 8.4.1 for *Column T2*, most of the heavy traffic circulates on the lane in the right and very few in the other(s) lanes of the carriageway. Therefore, the IRI performance of the carriageway is different to the carriageway of a two-lane road, where both lanes has the same loads. So Consequently, this PhD thesis only aims to develop IRI evolution models for two-lane roads.

Consequently, only roads from Conservation Areas 1, 2 and 3 were analyzed for IRI models. Conservation Area 4 is composed of main motorways around the metropolitan area of Bilbao, the capital of the province of Bilbao. Very few two-lane stretches exist in Area 4 (less than 5 km, without considering the N-634, in which very few stretches with the complete pavement section are known) and hence, they are not analyzed.

Once the stretches of Areas 1, 2 and 3 where the pavement structure is completely known are analyzed, it was tried to analyze all the observed individual stretches of 100 m. It was observed a very wide variance within a stretch with the same predicting values. For example, in an homogeneous stretch of 2 km with the same pavement section from the same project and the same traffic volume (both total and heavy traffic), 20 values of 100 m are available, it was noted a wide range of variance for IRI data, making impossible to correlate all the data from all the stretches with known section. Consequently, it was decided to calculate the mean IRI for each stretch with identical characteristics on pavement structure, pavement age (implying it comes from the same project) and traffic volume, from the data of the 100 m-sub-stretches in which the values are available. This procedure, the calculation of the mean value on a stretch with identical characteristics is logical and common because the deterministic models proposed in the literature try to predict the mean IRI value derived from some predicting variables and not the existing variance or range of variation within the identical stretch. This variance is also important and it is considered as one of the most important targets of the probabilistic models, which aim to forecast the percentage of the stretch in each of the predetermined states, as commented in Chapter 4. Pavements are said to probabilistic in nature (Tjan and Pitaloka, 2005; Abaza, 2016a; Li *et al.*, 1997; Hong, 2014), and this variability certifies it. However, probabilistic models are usually employed when few independent variables are available, but if detailed information is available (as in the road network of Biscay), it is recommended a deterministic analysis. Consequently, the mean value of stretches with identical characteristics is calculated and therefore, the mean IRI value will be predicted. Additionally, to complete the analysis, with each stretch, the standard deviation is calculated (Eq. 8.13). Thus, more information about the variability can be examined.

$$s = \sqrt{\sigma^2} = \sqrt{\frac{\sum_{i=1}^{N} (x_i - \mu)^2}{N}} \qquad [8.13]$$

Where x_i is each of the 100 m-stretch observation, μ is the mean IRI value for all the observation and N is the total number of observations in the considered identical stretch. In Eq. 8.13, it was selected the population standard deviation instead of the sample standard deviation (N-1 in the denominator) because the data represent the total population for that stretch with its specific characteristic. It is not a sample from a population but the total population itself.

After the mean IRI values for identical stretches are calculated, the total quantity of observations extracted from each road in Areas 1, 2 and 3 are shown in Table 8.9.

From the 186 New Outline observations in two-lane roads, 105 correspond to flexible pavements and 81 to semi-rigid pavements. The 20 new outline stretches of double carriageway have semi-rigid pavements. From the 112 M&R stretches in two-lane roads, 72 are flexible pavements and 40 are semi-rigid. The 30 M&R stretches in double carriageway motorways are semi-rigid pavements.

8.5. Proposed IRI performance models for New Outline pavements

8.5.1. Considered variables in the modelling

To describe the independent variables that are introduced in the modelling as possible influencing factors on IRI performance in the road network of Biscay, they are commented following the groups of variables usually included in IRI performance models, as listed in Table 8.1.

As exposed in section 8.2, different variables were identified as influencing factor of roughness evolutions. From the factors listed in Table 8.1, the ones that have been selected for IRI forecasting in the road network of Biscay are described, indicating which data are used, according to the available data in the pavement management database.

8.5.1.1. Age

The age of the pavement is a usual factor employed in roughness modelling and, hence, it was incorporated to the modelling in two different ways. The value of *column L*, ***Age***, indicates the difference between natural years of opening to traffic (or finishing the M&R work) and the year of data collection. As these dates can vary along the year, the *column M*, **real age (*R.Age*)**, indicates a more accurate value of the age, as the exact dates are considered. The age is expressed in years as a decimal fraction, when 0,5 represents 6 months. Moreover, when the new road or the M&R work is finished in the same year as the data collection, age indicates 0, which could indicate the value of IRI when the road is opened. Nonetheless, *column M*, *R.Age*, indicates the real age of the new or maintained pavement.

8.5.1.2. Traffic volume

In Chapter 5 and in section 8.2 it was commented the importance of the traffic volume as a factor for pavement deterioration. It represents the loads that are applied to the road structure. Generally, the Equivalent Single Axle Load (ESAL) is employed to translate the damage of the different weights of the vehicles on the road to the damage caused by a standard load. In Spain, the Equivalent Single Axle Load has a weight of 13 t. To determine the number of ESALs that crossed a section, the weights of the different vehicles must be known and a transfer function is employed to translate the caused damage, usually of the form of a fourth grade parabola. Nonetheless, the number of different vehicles that cross each section is unknown in Biscay. Traffic data only differentiate light vehicles and heavy vehicle. A heavy vehicle is considered when its total weight is over 3.500 kg (Ministerio de Obras Públicas y Urbanismo, 1989a; Ministerio de Fomento, 2003b). Some measures were made for determining the weight distribution, but they correspond to few specific roads. On those roads, those data could be employed to calculate the total number of ESALs that have crossed, but for the rest or roads, these values would have to be extrapolated. Therefore, instead of using transforming equations to the entire road network, it seems more reasonable to use directly the available data: total traffic volume and heavy traffic volume.

Table 8.9. Stretches with different values in considered information, selected for IRI modelling from conservation areas 1, 2 and 3.

	Area 1					**Area 2**					**Area 3**			
	New Outline		**M&R**			**New Outline**		**M&R**			**New Outline**		**M&R**	
Road	**2L**	**2C**	**2L**	**2C**	**Road**	**2L**	**2C**	**2L**	**2C**	**Road**	**2L**	**2C**	**2L**	**2C**
BI-631	4				BI-623	5		8	18	BI-624				
BI-634	2		2		BI-633	32		16		BI-625	14	8	13	12
BI-635	5		12		BI-638					BI-630				
BI-735	2		2		BI-732	3				BI-712	2		3	
BI-737	3				BI-2224	8		1		BI-745				
BI-2101					BI-2301	2		5		BI-2521				
BI-2120	3		2		BI-2405	8		2		BI-2522	1			
BI-2121	8		1		BI-2543	4		16		BI-2617				
BI-2122	2				BI-2632					BI-2625	3			
Bi-2153	1				BI-2636					BI-2701	7		13	
Bi-2235	6		2		N-240	18		1		BI-2757	7			
Bi-2237	2		2		N-636					BI-2794	3			
Bi-2238	6		5							N-639	12		2	
BI-2704	12		4											
BI-2713	1													
BI-2731		4												
Total	57	4	32		**Total**	80	8	49	18	**Total**	49	8	31	12

Total New Outline Two-lane roads	186
Total New Outline Double carriageway	20
Total Maintenance & Rehabiliation Two-lane roads	112
Total Maintenance & Rehabiliation Double carriageway	30
TOTAL	**358**

Note: 2L = two-lane road. 2C = double carriageway roads.

As exposed in 8.4.1, traffic data introduced in the modelling are the Annual Average Daily Traffic (***AADT***), in vehicles/day (*column T1*), Annual Average Daily Heavy Traffic (***H.AADT***) in heavy vehicles/day/lane, in the project lane (*column T2*), the traffic category according to Ministerio de Fomento (2003b) (*column T3*) in the year of the data collection; the accumulated total number of vehicles that has crossed that section, ***TotVeh***, (*column T4*) and the accumulated total number of heavy vehicles, ***TotH.Veh*** (*column T5*). For these last two columns, the exact date of opening to traffic of the stretch (or the date of finishing the M&R work) and the date of the data collection are used to calculate the data. From 2000, *AADT*, percentage of heavy vehicles, *H.AADT* and traffic category are included in the database. Data from previous years can be found in previous documents of the RGB. However, when data before 2000 are required it has been supposed an annual increasing rate of the traffic volume of 2 %, which has been demonstrated to be a reasonable value for the decade of the 90s (Departamento de Vivienda, Obras Públicas y Transportes, 2012, Pérez-Acebo, 2018). *TotVeh* and TotH.Veh variables are expressed in thousands of vehicles.

8.5.1.3. Structural parameters

As seen in Table 8.1, structural parameters are employed in IRI predictions, usually by means of the structural number or variations of it. However, this is not a parameter that is used in Spain and is not available in the database, as no test was conducted to obtain it. Therefore, the structural capacity of a pavement section to withstand the loads is introduced in the deterioration modelling by indicating all the materials and the thickness of each layer, as explained 8.4.2 and 8.4.3. For bituminous layers, three layers are estimated, surface, binder and base, following the specifications of Spanish regulations. If only two bituminous layers are present they are registered in the columns as Surface and Binder layer. For non bituminous base and subbase layers, they are referred as Subbase layer 1 and Subbase layer 2, to distinguish them from the bituminous base layer. Normally one subbase layer is displayed, as in flexible pavements, but in semi-rigid pavements, the subbase is composed of two layers. Therefore, two possible layers are included.

Total thickness of bituminous layers, ***TotBit***, is included as a parameter to evaluate the contribution of bituminous layers to the resistance of the structure. It is expressed in cm. However, it does not reflect the difference characteristics of each material, not making difference between Asphalt Concrete mixes (AC), discontinuous mixes (BBTM 11A or BBTM 11B) and porous asphalts (PA). As previously commented, the thin thickness of slurries (< 1,5 cm) is not reflected in the Pavement Management Systems as additional thickness to the pavement structure, and therefore, it is not added to the total thickness of a M&R pavement. With the aim of reflecting the variable nature of the bituminous materials, a parameter is created, Structural Strength of bituminous layers, $\boldsymbol{SS_{Bit}}$, which is calculated as the sum of the individual products of the thickness of each layer and its Young modulus, as expressed in Eq. 8.14.

$$SS_{bit} = \sum_{i=1}^{n} Bth_i \cdot E_i \qquad [8.14]$$

Where

SS_{bit} is the parameter denoted as Structural Strength of bituminous layers, which reflects the contribution of the different bituminous layers to withstands the loads

Bth_i is the thickness in cm of each bituminous layer

E_i is the modulus of Young of each bituminous layer, that can be obtained from Table 8.10.

n is the total number of bituminous layers in the section.

Table 8.10. Values of the modulus of Young for different bituminous materials (Departamento de Vivienda, Obras Públicas y Transportes, 2012)

Material	Modulus of Young (MPa)
Asphalt Concrete (AC) mixes: Dense (D) and semi-dense (S)	6.000
Asphalt Concrete (AC) mixes: (G)	5.000
Discontinuous mixes (BBTM A and B) and Porous Asphalt (PA)	4.000

In order to reflect the contribution of the subbase, a similar parameter was created *SSsub*, calculated as Eq. 8.15.

$$SS_{sub} = \sum_{i=1}^{n'} Sth_i \cdot E_i \qquad [8.15]$$

where

SS_{sub} is the parameter denoted as Structural Strength of subbaselayers, which reflects the contribution of the different subbase layers to withstands the loads

Sth_i is the thickness in cm of each non bituminous layer of the subbase

E_i is the modulus of Young of each layer, which can be obtained from Table 8.11.

n' is the total number of non bituminous layers in the section.

Table 8.11. Values of the modulus of Young for different non bituminous materials (Departamento de Vivienda, Obras Públicas y Transportes, 2012)

Material	Modulus of Young (MPa)
Soil-cement with unbound material	8.000
Soil-cement with crushed stone	12.000
Cement treated material (Gravel and cement) (Gravacemento)	22.000
Crushed stone	250*
Gravel and slag	10.000*
Vitrified slag	10.000*

* These values are approximate

To obtain a global idea of the strength of the pavement section, calculated individual contribution of bituminous and non bituminous layers are summed in an additional new parameter, Structural Strength total, ***SS**$_{tot}$*, which is calculated as:

$$SS_{tot} = SS_{bit} + SS_{sub} \qquad [8.16]$$

With regard to soil parameters, it was previously commented that very few data about the subgrade is registered in the PMS database. Therefore, any parameter of the soil is introduced as a possible influencing factor.

8.5.1.4. Other parameters

Other parameters were listed and Table 8.1 and are commented below:

- **Distresses.** The presence of distresses in a road, mainly cracks and rutting, implies a roughness increase, and can be used to predict roughness evolution. However, data about cracking and roughness are incomplete in the database, and it was decided not to include it. Moreover, an empirical model is aimed to be developed, avoiding mechanistic theories.
- **Climate.** As commented, the province of Biscay is small (2.217 km^2) and has the same climate (Oceanic climate). Therefore, small variations can be observed between the areas in which the province could be divided. They do not represent a variation higher than 10 % in rainfall data or temperatures. The introduction of the climate as an influencing factor makes sense when a global model is provided, as the HDM-III or HDM-4, or the ones provided in the MEPDG (AASHTO, 2008; 2015), where very different climates can be found. In the case of Biscay, it can be said that a performance model for a specific climate (Oceanic) is developed.
- **Initial IRI**. The initial IRI of the road is not registered in the database. Maximum initial IRI, before being a road is opened to traffic must not be achieved (Ministerio de Fomento, 2001; 2004a; 2008; 2011b; 2015), but the exact values of each project are not recorded.
- **Other parameters.** Some models included geometric data (lane width) or functional data (urban or interurban roads). As usual lane width is 3,5 m and all the roads are interurban, no other parameters have been included as influencing factors.

In the literature models for IRI performance in flexible pavement and semi-rigid pavements have been traditionally considered differentially. Consequently, an IRI performance model for flexible pavement and an IRI performance model for semi-rigid pavements are aimed to be developed.

8.5.2. IRI performance models for flexible pavement roads

The IRI evolution model for flexible pavements includes the following initial independent variables: ***Age***, ***R.Age***, ***TotBit***, $\boldsymbol{SS_{bit}}$, $\boldsymbol{SS_{tot}}$, $\boldsymbol{SUB_{cm}}$, ***AADT***, ***H.AADT***, ***TotVeh*** and ***TotH.Veh***. $\boldsymbol{SS_{sub}}$ was not considered because in all the flexible pavement section, the same unbound material is present, crushed stone (*zahorra*), and therefore, its value is proportional to SUB_{cm}. The variable SS_{tot} is included because it comes from a combination of SS_{bit}, and SS_{sub}.

Initially, an explanatory analysis of the dependent variable and the independent variables is carried out (Table 8.12 and 8.13). No one of the quantitative variables follows a normal distribution (Table 8.14).

The correlation between the independent variables and the dependent variables was carried out by means of the coefficient of Pearson, *R*, indicating the significance of that correlation (Table 8.15). Only the correlation between the dependent variable and each of the independent variables is shown, but not between independent variables

Table 8.12. Explanatory analysis for the variables in New Outline stretches with flexible pavement. First part.

		IRI	Age	R.Age	TotBit	SUBcm	SSbit	SStot
Mean		2,411	9,85	9,76	16,74	26,09	91829	98350
Standard error		0,079	0,631	0,63	0,407	0,821	2125	2106
95% CI for mean	**Lower**	2,255	8,6	8,51	15,94	24,46	87614	94173
	Upper	2,567	11,1	11,01	17,55	27,71	96043	102527
Variance		0,650	41,803	41,85	17,404	70,81	474201099	465836058
Std. deviation		0,806	6,466	6,47	4,172	8,415	21776	21583
Minimum		1,114	0	0,1	8	12	44000	49000
Maximum		5,475	26	26	33	55	192000	198250
Range		4,361	26	25,9	25	43	148000	149250
Interquartile range		1,130	9	8,9	8	0	30000	22500
Skewness		1,021	0,688	0,692	0,386	2,451	0,843	0,819
Kurtosis		0,986	-0,27	-0,268	1,379	6,301	3,653	3,923

Table 8.13. Explanatory analysis for the variables in New Outline stretches with flexible pavement. Continuation

		AADT	Age	TotVeh	TotH.Veh
Mean		5937,1	242,7	19370,7	797,1
Standard error		351,3	18,6	1731,8	77,0
95% CI for mean	**Lower**	5240,4	205,9	15936,6	644,4
	Upper	6633,8	279,5	22804,9	949,8
Variance		12960008,9	36160,3	314891873,3	622596,5
Std. deviation		3600,0	190,2	17745,2	789,0
Minimum		508	17	21,51	0,67
Maximum		20284	1268	94659,69	3787,65
Range		19776	1251	94638,18	3786,98
Interquartile range		4100	237	21563,66	1037,02
Skewness		0,889	1,805	1,622	1,498
Kurtosis		2,015	6,795	3,264	2,451

Table 8.1440. Normality tests for the dependent and independent quantitative variables in New Outline stretches with flexible pavement

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
IRI	0,114	105	0,002	0,924	105	< 0,001
Age	0,156	105	< 0,001	0,941	105	< 0,001
R.Age	0,15	105	< 0,001	0,941	105	< 0,001
TotBit	0,22	105	< 0,001	0,92	105	< 0,001
SUBcm	0,437	105	< 0,001	0,583	105	< 0,001
SSbit	0,18	105	< 0,001	0,921	105	< 0,001
SStot	0,175	105	< 0,001	0,921	105	< 0,001
AADT	0,081	105	0,085	0,94	105	< 0,001
H.AADT	0,118	105	0,001	0,865	105	< 0,001
TotVeh	0,147	105	< 0,001	0,856	105	< 0,001
TotH.Veh	0,156	105	< 0,001	0,85	105	< 0,001

[a] Lilliefors Significance correction

Table 8.15. Correlation between the dependent variable and the independent variables (coefficient of Pearson) in New Outline stretches with flexible pavement

Independent Variables	Correlation with IRI (R)	Significance of the correlation (bilateral)
Age	0,509	< 0,001
R.Age	0,512	< 0,001
TotBit	-0,531	< 0,001
SUBcm	0,319	0,001
SSbit	-0,475	< 0,001
SStot	-0,448	< 0,001
AADT	-0,024	0,810
H.AADT	-0,055	0,574
TotVeh	0,380	< 0,001
TotH.Veh	0,308	0,001

As seen, the best correlations are obtained with *Age*, *R.Age* and *TotBit*. Correlations with SUB_{cm}, SS_{bit}, SS_{tot}, *TotVeh* and *TotH.Veh* are also high. On the contrary, *AADT* and *H.ADDT* show low correlation with very low significance. In a next step, possible transformations of variables were studied. From the analysis of the curves that best fit the relationship between the dependent variables and each of the independent variables, some transformations of variables were included. Table 8.16 shows the curves that best correlates an independent variable with IRI (the dependent variable). Sometimes, a quadratic or a cubic curve fit better but they do not reproduce the pattern described in the literature. Other times, if the improvement between the linear correlation and other ones in the coefficient of determination is very low ($\Delta R^2 < 0{,}05$) a linear model has been maintained, implying that the independent variable has not been transformed.

Table 8.16. Equations that best correlate each independent variable individually with the dependent variable in New Outline stretches with flexible pavement

Independent Variable	Equation type	Resume of the model					Parameter estimates			
		R^2	F	Degrees freedom 1	Degree freedom 2	Sig.	Intercept	b1	b2	b3
Age	Potencial	0,277	39,494	1	103	< 0,001	1,778	0,026		
R.Age	Exponential	0,28	40,019	1	103	< 0,001	1,78	0,026		
TotBit	Logarithm	0,304	45,022	1	103	< 0,001	7,153	-1,702		
SUBcm	Linear	0,101	11,634	1	103	0,001	1,615	0,031		
SSbit	Logarithm	0,257	35,653	1	103	< 0,001	21,744	-1,696		
SStot	Logarithm	0,225	29,842	1	103	< 0,001	22,038	-1,711		
AADT	Inverse	0,079	8,891	1	103	0,004	2,231	527,196		
H.AADT	Inverse	0,088	9,997	1	103	0,002	2,23	17,713		
TotVeh	Quadratic	0,228	15,072	2	102	< 0,001	2,364	-1,43E-05	4,71E-10	
TotH.Veh	Quadratic	0,147	8,816	2	102	< 0,001	2,354	0	2,05E-07	

As shown in Table 8.14, different transformations can be made for improving the correlation between each independent variable and *IRI* (the dependent one). Age improves its correlation by means of a potential transformation, but *R.Age* shows a better correlation by means of an exponential relationship of the shape of Eq. 8.17. *Age* cannot be transformed by means of an exponential transformation because if the new outline is carried out in the same year that the data collection, its value is 0. Consequently, for relating the age and *IRI* it was preferred to use *R.Age*, which is a more accurate value for age.

$$IRI = 1{,}78 \cdot e^{0{,}026 R.Age} \quad [8.17]$$

For *TotBit*, SS_{bit} and SS_{tot}, a logarithm transformation is suggested to improve the value of *R*. For *AADT* and *H.AADT*, an inverse transformation is recommend, but the value that does not improve considerably and are the relationships with the lowest significance (p = 0,02 and p = 0,04). For *TotVeh* and *TotH.Veh*, a quadratic transformation is applied because observing the scatterplot of the points, it is an appropriate curve to fit the data in both cases. Hence, variables were transformed as indicated in Table 8.14. As the relationship between *IRI* and *R.Age* follows the expression of Eq. 8.17 and not to transform the independent variable, it was preferred to transform *IRI* by means of a natural logarithm, developing a relationship as indicated in Eq. 8.18.

$$LnIRI = A + B \cdot R.Age \quad [8.18]$$

Since it wanted to obtain a correlation between IRI and the independent variables following the structure of Eq. 8.17 it was necessary to transform the previously transformed variables by means of natural logarithm, in order to produce a general equation such as Eq. 8.19.

$$\mathrm{Ln}IRI = A + B \cdot \mathrm{R.Age} + C * \mathrm{Ln}(\textit{Transformed variable 1}) + D * \mathrm{Ln}(\textit{Tranformed variable 2}) \quad [8.19]$$

Once the possible equation for IRI performance is develop as indicated in Eq. 8.19, it could be transformed to a most common expression such as Eq. 8.20.

$$IRI = A' + B' * \mathrm{e}^{(B'' * R.Age)} + C' * \textit{Transformed variable 1} + D' * \textit{Transformed variable 2} \quad [8.20]$$

After applying the natural logarithm to the commented transformed variables, the obtained correlations with the new dependent variable, Ln*IRI*, are displayed in Table 8.17.

As observed, the correlation between the IRI and the independent variables improved, except for Ln(*TotVeh*^2) and Ln(*TotH.Veh*^2). Hence, it is necessary to consider the variables related to total traffic since opening without transformations.

With the transformed variables and with Ln*IRI* as dependent variable, a multiple linear regression model, by means of the functions Step by Step and Forward of the SPSS v.24 programme was calculated. It indicated an equation (Eq. 8.21) that has global significance (p-value of the F test below 0,05) and individual significance of all the variables (p-value of the t-test below 0,05).

The complete Analysis Of Variance of the suggested model is shown in Table 8.18. The coefficient of determination R^2 is 0,419, and the adjusted R^2 is 0,408.

Maintaining Ln*IRI* as the dependent variable, more possible models were tried to obtain a better coefficient of determination (R^2) as long as the relationship is significant (p-value of the *F* test below 0,05) and the coefficient of all the introduced variables are significant (p-value of the t-test below 0,05). Different combinations of available variables were checked. Other assumptions of the multiple linear regression analysis (colinearity, homocesdacity, etc.) are also checked to know the adequacy of the model. Table 8.19

presents some of the models analyzed, indicating the variables introduced, obtained R^2 and adjusted R^2 and the comments and observations about the models.

Table 8.17. Correlation between the transformed dependent variable (LnIRI) and the transformed independent variables (as suggested in Table 8.16 (coefficient of Pearson)) in New Outline stretches with flexible pavement

Independent Variables	Correlation with IRI (R)	Significance of the correlation (bilateral)
Age	0,526	< 0,001
R.Age	0,529	< 0,001
Ln(LnTotBit)	-0,550	< 0,001
Ln(SUB1cm)	0,218	0,026
Ln(LnSSbit)	-0,508	< 0,001
Ln(LnSSRtot)	-0,478	< 0,001
Ln(TotVeh^2)	0,242	0,013
Ln(TotH.Veh^2)	0,218	0,026

The dependent variables is LnIRI

$$LnIRI = Int + B \cdot R.Age + C \cdot Ln(LnTotBit) \quad [8.21]$$

Table 8.1841. Analysis of variance of the multiple linear regression model proposed by the Step by Step function of the SPSS programme in New Outline stretches with flexible pavement

Analysis of variance						
Source	**Sum of squares**	**Degrees of freedom**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**
Model	4,373	2	2,186	36,797	< 0,001	1,260
Error	6,061	102	0,059			
Corrected total	10,434	104				
Root mean square error		**R**	**R^2**	**Adj. R^2**	**Colinearity statistics**	
					Tolerance	**VIF**
0,24376		0,647	0,419	0,408	0,847	1,180
Parameter estimates						
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**	
Intercept	2,005	0,290	6,922	< 0,001	1,430	2,579
R.Age	0,018	0,004	4,533	< 0,001	0,010	0,026
Ln(LnTotBit)	-1,328	0,266	-4,993	< 0,001	-1,855	-0,800

Table 8.19. Proposed multiple linear regression models for IRI performance in New Outline stretches with flexible pavement and LnIRI as dependent variable.

Proposed model	R^2	Adj R^2	Comments and observations
LnIRI = Int + R.Age + Ln(LnSStot) + Ln(TotH.Veh^2)	0,378	0,360	Low significance for Ln(TotH.Veh^2) (p = 0,823)
LnIRI = Int + R.Age + Ln(LnSStot) + Ln(TotVeh^2)	0,378	0,359	Low significance for Ln(TotVeh^2) (p = 0,918)
LnIRI = Int + R.Age + Ln(LnSSbit) + Ln(TotH.Veh^2)	0,391	0,373	Low significance for Ln(TotH.Veh^2) (p = 0,780)
LnIRI = Int + R.Age + Ln(LnSSbit) + Ln(TotVeh^2)	0,391	0,372	Low significance for Ln(TotVeh^2) (p = 0,941)
LnIRI = Int + R.Age + Ln(LnTotBit) + Ln(TotVeh^2)	0,420	0,403	Low significance for Ln(TotH.Veh^2) (p = 0,612)
LnIRI = Int + R.Age + Ln(LnTotBit) + Ln(TotH.Veh^2)	0,423	0,406	Low significance for Ln(TotH.Veh^2) (p = 0,356)

Note: Int: Intercept.

As observed, Ln(Ln*TotBit*) provides better correlations than Ln(LnSS_{tot}) or Ln(LnSS_{bit}). Thus it is deducted that the total thickness of the bituminous layers is better correlated with the obtained IRI than the proposed variable that represents its structural capacity. Moreover, Ln(LnSS_{bit}), which accounts the total thickness of the layer of unbound material, provides worse correlation. It can be deducted that the bituminous layers are the real responsible of the structural capacity of the section, and that the unbound material provides a low quantity to it, resulting in a non influencing factor for IRI performance.

The last model of Table 8.19 indicates the best model that can be obtained despite the low significance of the variable Ln(*TotH.Veh^2*) (p = 0,356). In order to present a model in which the dependent variable is IRI, *R.Age* was transformed following the best curve, indicated in Eq. 8.17. With this transformed variable, designs as *ExpR.Age*, additional multiple linear regression models were tested, displayed in Table 8.20.

Table 8.2042. Analyzed multiple linear regression models for IRI performance in New Outline stretches with flexible pavement with IRI as dependent variable.

Proposed model	R^2	Adj R^2	Comments and observations
IRI = Int + ExpR.Age + LnSStot + TotVeh^2	0,410	0,392	All variables are significant (p < 0,05) CI = 145
IRI = Int + ExpR.Age + LnSStot + TotH.Veh^2	0,394	0,376	All variables are significant (p < 0,03) CI = 148
IRI = Int + ExpR.Age + LnSSbit + TotH.Veh^2	0,405	0,380	All variables are significant (p < 0,03) CI = 138
IRI = Int + ExpR.Age + LnSSbit + TotVeh^2	0,415	0,398	All variables are significant (p < 0,05) CI = 135
IRI = Int + ExpR.Age + LnTotbit + TotVeh	0,411	0,393	Medium significance of TotVeh (p=0,18) CI = 39
IRI = Int + ExpR.Age + LnTotbit + TotH.Veh	0,416	0,398	Medium significance of TotH.Veh (p=0,1) CI=41
IRI = Int + ExpR.Age + LnTotBit + TotVeh^2	0,442	0,425	All variables are significant (p < 0,06)
IRI = Int + ExpR.Age + LnTotBit + TotH.Veh^2	0,437	0,420	All variables are significant (p < 0,05)

Note: Int: Intercept. CI = Condition Index

From the summary of tested models of Table 8.20, it can be observed that the variable ExpR.Age is always significant; a better correlation is obtained with LnTotBit than with LnSStot or LnSSbit and the accumulated total number of vehicles and heavy vehicles since the road was opened until the moment of the IRI data collection is also significant if introduced with the quadratic transformation. Taken the first two variables as fixes, different combination of the variables related to accumulated traffic were tried and this analysis is summarized in Table 8.21.

When compared Table 8.20 and 8.21, it can be deducted that the best models are the last one of each table. The last one in Table 8.20 employs the accumulated total number of heavy vehicles that crossed the section with a coefficient of determination of 0,437 (R^2 = 0,437). The last one in Table 8.21 uses as the variable for traffic the accumulated total number of vehicles (of any type) that crossed the section, with a coefficient of determination of 0,472 (R^2 = 0,472), providing a better prediction.

Traditionally, deterministic IRI performance models include the accumulated heavy traffic as a variable for increasing the roughness as heavy vehicles are said to be responsible of pavement damaging due to its higher weight, implying greater loads over the pavement. The employment of the parameter ESAL (Equivalent Single Axle Load) tries to reflect the equivalence of each vehicle as a number of “ESALs” to calculate their influence in the pavement deterioration (Eq. 8.22).

$$ESAL_{128} = K \cdot (P_i/128)^{\alpha} \qquad [8.22]$$

Where ESAL is Equivalent Single Axle Load of 128 kN (or 13 t) and represents the number of equivalent simple axles of 128 kN (or 13 t) that would produce the same damage as an axle with a total load of P_i (in KN); Pi is the weight over each individual axle; K is 1 for single axle, 1,4 for double axles and 2,3 for triple axles and α is 4 for flexible pavement and 8 for rigid and semi-rigid.

Table 8.21. Analyzed multiple linear regression models for IRI performance in New Outline stretches with flexible pavement with IRI as dependent variable and different combination of the variables related to accumulated traffic.

Proposed model	R^2	Adj R^2	Comments and observations
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotH.Veh^2	0,442	0,420	Low significance,TotVeh^2 (p=0,33), TotH.Veh (p=0,741)
IRI = Int + ExpR.Age + LnSStot + TotVeh^2 + TotH.Veh^2	0,410	0,386	Low significance,TotVeh^2 (p=0,107), TotH.Veh (p=0,860)
IRI = Int + ExpR.Age + LnSSbit + TotVeh^2 + TotH.Veh^2	0,415	0,392	Insignificance of TotH.Veh (p=0,986)
IRI = Int + ExpR.Age + LnTotbit + SUB1cm + TotVeh^2 + TotH.Veh^2	0,444	0,415	Low significance,TotVeh^2 (p=0,554), TotH.Veh (p=0,646), SUB1cm (p=0,627)
IRI = Int + ExpR.Age + TotVeh^2 + TotH.Veh^2	0,308	0,287	Medium significance of TotH.Veh^2 (p=0,151)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotH.Veh	0,445	0,423	Low significance,TotH.Veh (p=0,407)
IRI = Int + ExpR.Age + LnTotbit + TotVeh + TotH.Veh	0,416	0,392	Low significance,TotVeh (p=0,363), TotH.Veh (p=0,874)
IRI = Int + ExpR.Age + LnTotbit + TotVeh + TotH.Veh^2	0,443	0,421	Low significance,TotVeh (p=0,303)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotH.Veh^2 + TotVeh + TotH.Veh	0,480	0,448	Low significance,TotH.Veh^2 (p=0,397), TotH.Veh (p=0,256)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotVeh + TotH.Veh	0,476	0,449	Low significance, TotH.Veh (p=0,410)
IRI = Int + ExpR.Age + LnTotbit + TotVeh^2 + TotVeh	0,472	0,451	All variables are significant (p < 0,05)

Note: Int: Intercept. CI = Condition Index

By means of the ESAL it can be deducted that the passenger cars (light vehicles) do not imply great damages on the road, as the effect of a heavy vehicles is similar to the one produced by thousands of passenger cars.

The explanation for the better correlation between IRI and total vehicles may be the high correlation between all the variables (TotVeh and TotH.Veh and their quadratic transformation) (Table 8.22) which makes that they can substitute each other in the models but, as seen in Table 8.21, if the total and heavy accumulated traffic are included, at least one of them become insignificant. This high correlation between the variables is originated because the percentage of heavy vehicles in all the roads is similar, and can be stated to be between 5 and 8 % percent of the total traffic. Furthermore, the total number of observations (105) is not high, which may be the result for this fiction correlation between IRI and total vehicles.

Table 8.22. Correlation between the variables related to the accumulated traffic (coefficient of Pearson)

	TotVeh	TotH.Veh	TotVeh^2	TotH.Veh^2
TotVeh	1	0,925	0,857	0,923
TotH.Veh	0,925	1	0,924	0,843
TotVeh^2	0,857	0,924	1	0,910
TotH.Veh^2	0,923	0,843	0,910	1

Consequently, although the equation with the accumulated total number of vehicles provides a better correlation, the one with the accumulated total number of heavy vehicles was selected for predicting IRI evolution (Eq. 8.23).

$$IRI = 0{,}680 \cdot e^{(0{,}026 \cdot R.Age)} - 1{,}411 \cdot LnTotBit + 0{,}079 \cdot 10^{-6} \cdot VehTotPes^2 \qquad [8.23]$$

Where:

IRI is the predicted mean IRI (m/km) value of the stretch with identical variable values of age, traffic and thickness of bituminous layers in flexible pavements

R.Age is the real age of the pavement, calculated from the exact date of opening to traffic until the moment it is wanted, in years in decimal fraction, when 0,5 means 6 months.

TotBit is the total thickness of the bituminous layers in the flexible pavement, in cm

VehH.Tot is the total accumulated heavy vehicles that have circulated in the period considered through the project lane (the lane with greater quantity of heavy vehicles in the section) since it was opened to traffic to the moment wanted to be calculated, in thousands of heavy vehicles. Usually, both directions are supposed to have identical heavy traffic, the half of the total.

It is thought that this type of equation can be employed in other roads of other regions and countries as it is composed of the basic factors that affect roughness evolution. On one hand, as the subbase is the same for all the flexible pavements, unbound material, the pavement strength is due mainly to the bituminous layers. Hence, the total thickness of bituminous layers has a negative effect on roughness progression. On the contrary, the traditional external factors that deteriorate the pavement are the age, computed from the moment when the road was opened to the moment wanted to be predicted, and the total number of heavy vehicles that passed over the pavement in the project lane. Tables 8.23 to 8.26 and Fig. 8.6 and 8.7 show the complete statistic analysis of the model.

Table 8.23. Analysis of variance of the model of Eq. 8.23 for New Outline stretches with flexible pavements in two-lane roads.

Analysis of variance								
Source	**Sum of squares**	**Degrees of freedom**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**	**Root mean square error**	**R**
Model	29,509	3	9,836	26,114	<0,001	1,448	0,6137	0,661
Error	38,043	101	0,377				**R^2**	**Adj. R^2**
Corrected total	67,552	104					0,437	0,420

Parameter estimates							Colinearity statistics	
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**		**Tolerance**	**VIF**
Intercept	5,353	0,968	5,531	<0,001	3,433	7,272		
ExpR.Age	0,382	0,19	2,011	0,047	0,005	0,759	0,584	1,714
LnTotBit	-1,411	0,254	-5,544	<0,001	-1,916	-0,906	0,82	1,219
TotH.Veh^2	0,079	0,031	2,568	0,012	0,018	0,141	0,685	1,459

Table 8.24. Coefficient correlations between the independent variables of the model of Eq. 8.23

		VehH.Tot^2	LnTotBit	ExpR.Age
Correlations	VehH.Tot^2	1	-0,177	-0,557
	LnTotBit	-0,177	1	0,419
	ExpR.Age	-0,557	0,419	1
Covariances	VehH.Tot^2	0,001	-0,001	-0,003
	LnTotBit	-0,001	0,065	0,02
	ExpR.Age	-0,003	0,02	0,036

Table 8.25. Colinearity diagnosis of Eq. 8.23

Dimension	**Eigenvalue**	**Correlation index**	**Proportions of the variance**			
			Intercept	**VehTot^2**	**LnTotBit**	**ExpR.Age**
1	3,29	1	0	0	0	0,02
2	0,686	2,19	0	0	0	0,67
3	0,022	12,312	0	0,51	0,11	0,2
4	0,002	36,842	1	0,49	0,89	0,11

Table 8.26. Statistics of the residuals of Eq. 8.23

	Minimum	**Maximum**	**Mean**	**Standard deviation**	**N**
Predicted value	1,201	4,044	2,411	0,533	105
Residual	-1,054	1,615	0,000	0,605	105
Standardized predicted value	-2,271	3,065	0,000	1,000	105
Standardized residual	-1,718	2,631	0,000	0,985	105

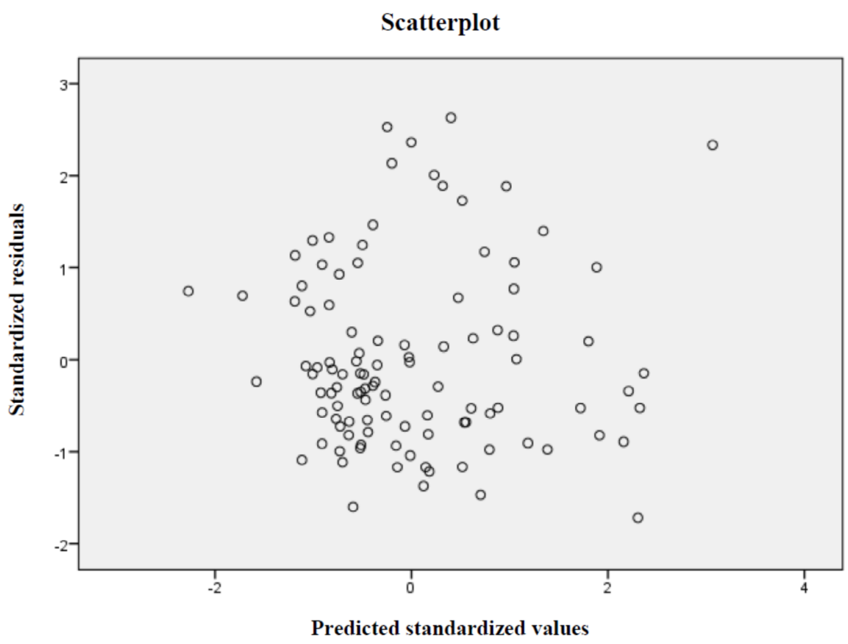


Fig. 8.6. Predicted standardized values plotted against predicted standardized residual for Eq. 8.23

Hypothesis that a multiple linear regression models must fulfilled, exposed in section 7.5.3 are checked. The adjusted R^2 is 0,420. As observed, the *F*-test provides a *p-value* < 0,001 (Table 8.23), which implies that the proposed relationship is true and the *Student's t*-test of the coefficients of the parameters shows that they are significant and different from 0 (the 95% confidence intervals for parameters do not include the zero). The Durbin-Watson statistics is 1,448, verifying the independence of errors, there is no autocorrelation. The homoscedasticity is verified in Fig. 8.6, where no patterns are detected. One Condition Index is over 30 (Table 8.25), which could show some problems of multicolinearity. Nevertheless, there are not great correlation between the variables which explain the model (Age and TotH.Veh could be related, but they only have a medium coefficient of Pearson, R = -0,557) and the Variance Inflation Factors are low (VIF > 10). Therefore, it can be stated that there is no multicolinearity. Residuals follow a normal distribution, with a mean equal to zero and the variance is near 1 (Table 8.23). Fig. 8.9 shows the plot of observed values vs. predicted values, showing that the observations are near the diagonal.

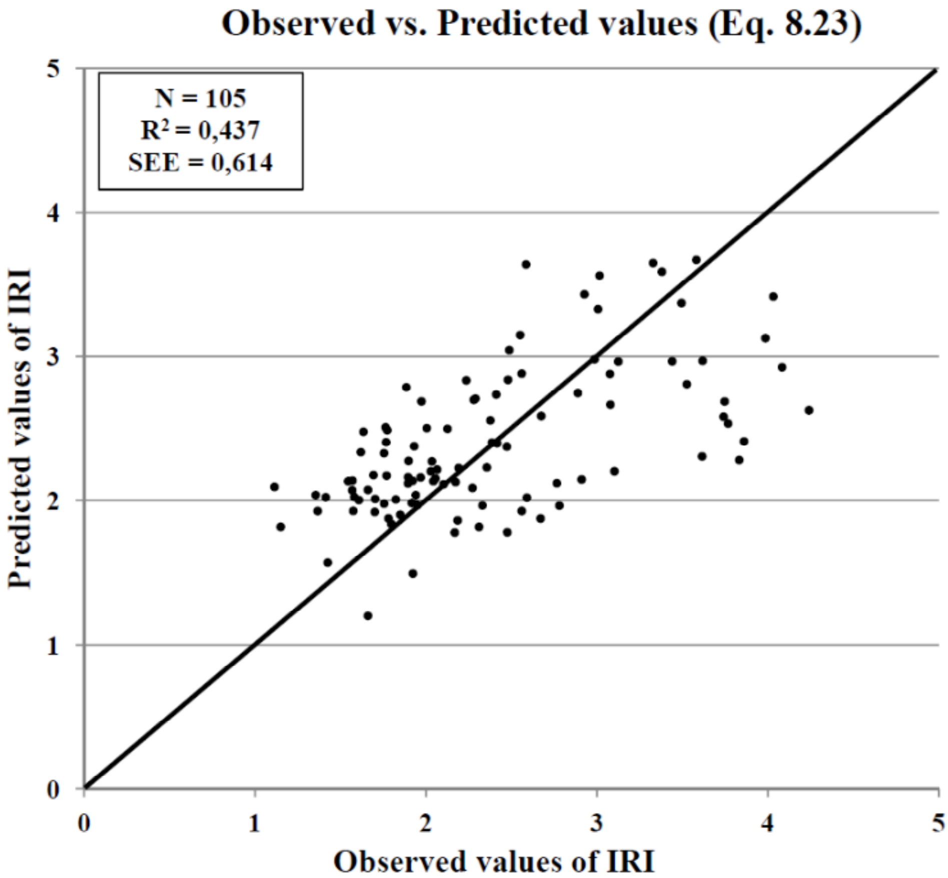


Fig. 8.7. Observed values vs. Predicted values with Eq. 8.23

8.5.3. IRI performance models for semi-rigid pavement roads

For predicting the IRI in New Outline stretches with semi-rigid pavements in two-lane roads, 81 observations are available from the Conservation Areas 1, 2 and 3 of Biscay (Table 8.9). Now, SS_{sub} is introduced instead of SUB_{cm}, because two layers can be present in a semi-rigid pavement.

First, an explanatory analysis of the variables is carried out (Table 8.27 and 8.28). Table 8.29 indicates that only IRI (the dependent variable) follows a normal distribution.

Table 8.27. Explanatory analysis for the quantitative variables (the dependent one and the independent ones) in New Outline stretches with semi-rigid pavement. First part.

		IRI	Age	R.Age	TotBit	SSsub	SSbit	SStot
Mean		2,123	7,11	6,898	13,840	398247	76840	475086
Standard error		0,057	0,603	0,601	0,376	20262	1846	20541
95% CI for mean	**Lower**	2,009	5,91	5,703	13,09	357925	73166	434208
	Upper	2,237	8,31	8,093	14,59	438569	80513	515965
Variance		0,267	29,45	29,214	11,461	33253932020	276036420	34177298690
Std. deviation		0,517	5,427	5,405	3,385	182357	16614	184871
Minimum		1,075	1	0,7	8	205000	48000	268000
Maximum		3,615	29	28,6	25	748000	131000	844000
Range		2,540	28	27,9	17	543000	83000	576000
Interquartile range		0,773	8	8,13	3	388000	26000	379000
Skewness		0,416	1,413	1,399	0,817	0,635	0,537	0,535
Kurtosis		-0,045	2,897	2,796	0,756	-1,206	0,086	-1,252

Table 8.28. Explanatory analysis for the quantitative variables (the dependent one and the independent ones) in New Outline stretches with semi-rigid pavement. Continuation

		AADT	Age	TotVeh	TotH.Veh
Mean		9378,62	581,12	18804,02	1182,07
Standard error		865,457	58,56	2127,83	148,17
95% CI for mean	**Lower**	7656,3	464,59	14570	887
	Upper	11100,93	697,66	23039	1477
Variance		60670231,49	277767,14	366739978	1778322
Std. deviation		7789,11	527,04	19150	1334
Minimum		911	19	1012,56	20,53
Maximum		36185	2533	102496,75	6957,42
Range		35274	2514	101484,18	6936,89
Interquartile range		9905	775	20912,8	1352,8
Skewness		1,405	1,195	1,914	1,888
Kurtosis		1,838	1,739	4,481	4,125

Table 8.29. Normality tests for the dependent and independent quantitative variables in New Outline stretches with semi-rigid pavement

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
IRI	0,082	81	0,200*	0,979	81	0,192
Age	0,13	81	0,002	0,878	81	< 0,001
R.Age	0,126	81	0,003	0,883	81	< 0,001
TotBit	0,215	81	< 0,001	0,91	81	< 0,001
SUB1cm	0,158	81	< 0,001	0,94	81	0,001
SSbit	0,263	81	< 0,001	0,827	81	< 0,001
SStot	0,222	81	< 0,001	0,86	81	< 0,001
AADT	0,164	81	< 0,001	0,854	81	< 0,001
H.AADT	0,177	81	< 0,001	0,862	81	< 0,001
TotVeh	0,185	81	< 0,001	0,802	81	< 0,001
TotH.Veh	0,192	81	< 0,001	0,79	81	< 0,001

[a] Lilliefors Significance correction

* This is lower bound of the true significance

Correlations between the dependent variables (IRI) and each of the independent variables was done by means of the coefficient of Pearson (Table 8.30). As in flexible pavement, *Age*, *R.Age* and *TotBit* show a good correlation with IRI. Once again, *TotBit* show better correlation than SS_{bit}, which is supposed to better reflect the difference bituminous materials extended and SS_{sub} and SS_{tot} even show a lower correlation, which is not significant (p-value > 0,05). In this case, *AADT* and *H.AADT* show good correlation with IRI and, on the contrary, *TotVeh* and *TotH.Veh* show the lowest correlations, R < 0,10.

The curves that best fit each of the independent variables were obtained, in order to know possible transformation to improve the correlations (Table 8.31).

Table 8.30. Correlation between the dependent variable and the independent variables (coefficient of Pearson) in New Outline stretches with semi-rigid pavement

Independent Variables	Correlation with IRI (R)	Significance of the correlation (bilateral)
Age	0,388	< 0,001
R.Age	0,390	< 0,001
TotBit	-0,384	< 0,001
SSbit	-0,302	0,006
SSsub	-0,134	0,231
SStot	-0,160	0,154
AADT	-0,407	< 0,001
H.AADT	-0,299	< 0,001
TotVeh	-0,072	0,525
TotH.Veh	-0,003	0,979

Table 8.31. Equations that best correlate each independent variable individually with the dependent variable in New Outline stretches with semi-rigid pavement

		Resume of the model					Parameter estimates			
Independent Variable	**Equation type**	R^2	**F**	**Degrees freedom 1**	**Degrees freedom 2**	**Sig.**	**Intercept**	**b1**	**b2**	**b3**
R.Age	Potential	0,234	24,117	1	79	< 0,001	1,668	0,133		
TotBit	Linear	0,148	13,671	1	79	< 0,001	2,934			
SSsub	Linear	0,018	1,455	1	79	0,231	2,275	-3,81E-07		
SSbit	Linear	0,091	7,918	1	79	0,006	2,844	-9,38E-06		
SStot	Linear	0,026	2,07	1	79	0,154	2,335	-4,47E-07		
AADT	Linear	0,166	15,72	1	79	< 0,001	2,376	-2,70E-05		
H.AADT	Linear	0,089	7,743	1	79	0,007	2,293	0		
TotVeh	Quadratic	0,026	1,042	2	78	0,358	2,075	7,37E-09	-1,27E-16	
TotH.Veh	Quadratic	0,003	0,132	2	78	0,877	2,096	5,19E-05		

Transformations indicated in Table 8.31 were carried out. *R.Age* was transformed by means of a potential equation, as shown in Eq. 8.24.

$$PotR.Age = 1{,}668 \cdot R.Age^{0{,}133} \qquad [8.24]$$

With the new variable *PotR.Age* and the transformed $TotVeh^2$ and $TotH.Veh^2$ a multiple linear regression model was tried by means of the Step by Step function of SPSS v24. Obtained solution included the variables of Eq. 8.25. Values of the model are displayed in Table 8.32.

$$IRI = PotR.Age + VehTot + TotBit \qquad [8.25]$$

Table 8.32. Analysis of variance of the model of Eq. 8.25 for New Outline stretches with semi-rigid pavements in two-lane roads

Analysis of variance								
Source	**Sum of Squares**	**Degrees of freedom**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**	**Root mean square error**	**R**
Model	8,089	3	2,696	15,657	< 0,001	1,602	0,415	0,616
Error	13,261	77	0,172				**R^2**	**Adj. R^2**
Corrected total	21,350	80					0,379	0,355
Parameter estimates							**Colinearity statistics**	
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**		**Tolerance**	**VIF**
Intercept	0,090	0,567	0,158	0,875	-1,040	1,219		
PotR.Age	1,256	0,235	5,345	< 0,001	0,788	1,724	0,652	1,534
TotVeh^2	-8,622E-9	0,000	-2,804	0,006	0,000	0,000	0,621	1,610
LnTotBit	-0,032	,015	-2,118	0,037	-0,062	-0,002	0,820	1,219

Additional multiple linear regression models were tested to obtain a better coefficient of determination (R^2), but checking that the relationship is significant, the coefficients of the variables introduced are significant and all the assumptions of the multiple linear model (Chapter 7) are verified. Table 8.33 shows a summary of the most interesting models that were checked.

Table 8.33. Proposed multiple linear regression models for IRI performance in New Outline stretches with semi-rigid pavement

Proposed model	**R^2**	**Adj R^2**	**Comments and observations**
IRI = Int + PotR.Age + SSbit + SSsub + TotH.Veh + TotVeh	0,387	0,346	Low significance, SSbit (p=0,13), SSsub (p=0,32)
IRI = Int + PotR.Age + TotH.Veh^2 + SSsub + TotBit	0,324	0,288	Low significance, TotH.Veh^2 (p=0,34), SSsub (p=0,94)
IRI = Int + PotR.Age + TotH.Veh^2 + TotBit	0,324	0,297	Low significance, TotH.Veh (p=0,34)
IRI = Int + PotR.Age + TotH.Veh + TotBit + SSsub	0,325	0,289	Low significance, TotH.Veh (p=0,31), SSsub (p=0,93)
IRI = Int + PotR.Age + LnTotBit + TotH.Veh	0,318	0,292	Low significance, TotH.Veh (p=0,328)
IRI = Int + PotR.Age + LnTotBit + TotVeh	0,371	0,347	Low significance, Int(p=0,82), IC=43
IRI = Int + PotR.Age + SSbit + TotVeh	0,361	0,337	Medium significance, SSbit(p=0,137)
IRI = Int + PotR.Age + TotBit + TotVeh	0,384	0,360	All variables are significant (p<0,05)
IRI = Int + PotR.Age + TotBit + TotH.Veh	0,330	0,304	Low significance, TotH.Veh (p=0,289)

Note: Int: Intercept.

As observed, it seems that the proposed variables SS_{bit}, SS_{sub} and SS_{tot} do not reflect the different qualities of the layers. Modulus of Young may be not the best parameter to calculate a parameter similar to SNC or perhaps the values were not totally corrected. However, possible layers in semi-rigid pavements have differences properties and this must affect the IRI performance. Therefore, some General Linear Models were tested, when the possible layers of the subbase were introduced as factors. It cannot be introduced as a unique factor because two layers could be extended in the subbase Consequently, the following levels for two possible factors, SUB1 and SUB2, were created, according to existing materials in the database (Table 8.34).

Table 8.34. Possible layers in the first subbase layer (SUB1) and in the second layer (SUB2) in semi-rigid pavements

First subbase layer (SUB1)	Second subbase layer (SUB2)
Cement treated material (Gravel and cement) (12)	Crushed stone (11)
Soil-cement (13)	Soil-cement (13)
Gravel and slag (14	Vitrified slag (15)
	Nothing (20)

Table 8.35 presents a summary of the General Linear Models, which can include different combination of factors and even, the factors introduced can be combined as needed. The programme SPSS v24 allows choosing the specific combination of factors and covariables. The sum of squares is Type III. The thickness of each of the subbase layers, *SUB1cm* and *SUB2cm* for the first and the second layer, respectively, are also included as quantitative variables.

Table 8.35. Proposed General Linear Models for IRI performance in New Outline stretches with semi-rigid pavement

Proposed model	R^2	Adj R^2	Comments and observations
IRI = Int + PotR.Age + TotBit + TotVeh + SUB1(f)	0,422	0,384	Low significance, TotBit (p=0,775)
IRI = Int + PotR.Age + TotBit + TotVeh + SUB1(f) + SUB2(f)	0,464	0,404	Low significance, TotBit (p=0,998), SUB2 (p=0,143)
IRI = Int + PotR.Age + TotVeh + SUB1(f) + SUB2(f)	0,464	0,413	Low significance, SUB2 (p=0,134), Int(p=0,1)
IRI = Int + PotR.Age + TotVeh + TotBit + SUB1(f) + SUB2(f) + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,470	0,376	Low significance, TotBit (p=0,89), SUB1 (p=0,96), SUB2 (p=0,28) and products
IRI = Int + PotR.Age + TotVeh + TotBit + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,460	0,391	Low significance, TotBit (p=0,69), and products (p>0,12)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,459	0,398	Low significance, SUB2*SUB2cm (p=0,38), and Int (p=0,41)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm + SUB2(f)	0,463	0,404	Low significance, SUB2(f) (p=0,29), and Int (p=0,285)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,466	0,381	Low significance, + SUB1(f)*SUB1cm (p=0,983) SUB2(f)*SUB2cm (p=0,673)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB2(f)	0,465	0,405	Medium significance, Int (p=0,146)
IRI = Int + PotR.Age + TotVeh + TotBit + SUB1(f)*SUB2(f)	0,465	0,397	Low significance, TotBit (p=0,995), Int (p=0,257)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm*SUB2(f)*SUB2cm	0,386	0,337	Low significance, product (p=0,271)
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB1cm*TotBit + SUB2(f)*SUB2cm*TotBit	0,459	0,399	Low significance, SUB2(f)*SUB2cm*TotBit (p=0,184), Int (p=0,426)
IRI = Int + PotR.Age + TotVeh^2 + SUB1(f)*SUB1cm*TotBit + SUB2(f)*SUB2cm*TotBit	0,438	0,375	Int could be zero (p=1,00)
IRI = Int + PotR.Age + TotH.Veh + SUB1(f)*SUB1cm*TotBit + SUB2(f)*SUB2cm*TotBit	0,394	0,327	Low significance, TotH.Veh (p=0,39), Int (p=0,563)

As observed in Table 8.35, when introducing the thickness of the subbase layers, those variables become insignificant in the model. They thickness and the type of material was even combined with TotBit but other variables obtained low significance. As in flexible pavement, different combination of variables related to traffic volume were tried. These attempts are summarized in Table 8.36.

Table 8.36. Proposed General Linear Models for IRI performance in New Outline stretches with semi-rigid pavement for different combinations of variables related to traffic volumes.

Proposed model	R^2	Adj R^2	Comments and observations
IRI = Int + PotR.Age + TotVeh + SUB1(f)*SUB2(f)*TotBit	0,464	0,396	Low significance of the Intercept (p=0,243)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f)	0,503	0,441	Low significance of the Intercept (p=0,318)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f)*TotBit	0,498	0,427	Low significance of the Intercept (p=0,274)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + TotBit + SUB1(f)*SUB2(f)	0,504	0,433	Low significance, Intercept (p=0,382), TotBit (p=0,884)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + LnTotBit + SUB1(f)*SUB2(f)	0,505	0,435	Low significance, Intercept (p=0,344), TotBit (p=0,602)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + LnTotBit + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,498	0,426	Low significance, Intercept (p=0,850), SUB2(f)*SUB2cm (p=0,480), LnTotBit (p=0,989)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB1cm + SUB2(f)*SUB2cm	0,498	0,434	Low significance, Intercept (p=0,801), SUB2(f)*SUB2cm (p=0,464)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB1cm + SUB2(f)	0,500	0,436	Low significance, Intercept (p=0,616), SUB2(f) (p=0,423)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)	0,481	0,446	Low significance of the Intercept (p=0,446)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f) + SUB2(f)	0,501	0,446	Low significance, Intercept (p=0,125), SUB2(f) (p=0,409)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f) + SUB2(f)*SUB2cm	0,500	0,444	Low significance, Intercept (p=0,199), SUB2(f)*SUB2cm (p=0,440),
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f) + SUB1(f)*SUB2(f)*LnTotBit	0,514	0,419	The products are insignificant (p > 0,7)
IRI = Int + PotR.Age + TotVeh + TotH.Veh + SUB1(f)*SUB2(f)*LnTotBit	0,504	0,434	Low significance of the Intercept (p=0,338)

As seen in Table 8.36, the best correlation equations with all the variables significant are the second one and the last one. In both equations, the age of the pavement, the accumulated total number of vehicles and heavy vehicles that circulated through the stretch and a factor that combines the non bituminous materials included in the subbase layers. The second model (the last equation of Table 8.36) also includes the Total thickness of bituminous layers combined with the factor of the combination of the non bituminous materials of the subbase layers. However, although including an additional variable both models have similar coefficient of determination (0,503 and ,504). Nevertheless, as the first one has one variable less, the adjusted R^2 (Eq. 7.41) is higher in the first one. Additionally, the significance of the coefficients of the variables are higher in the first model. Consequently, the first model (second equation of Table 8.36) is suggested as the model for new semi-rigid pavement is two-lane roads. It is defined by Eq. 8.26.

$$IRI = 0{,}217 + 2{,}237 \cdot R.Age^{0{,}133} - 2{,}208 \cdot 10^{-5} \cdot TotVeh + 1{,}738 \cdot 10^{-4} \cdot TotH.Veh + SUB_f \qquad [8.26]$$

Where

IRI is the predicted mean International Roughenss Index value (m/km) of the stretch with identical variable values of age, traffic and subbase composition.

R.Age is the real age of the pavement, calculated from the exact date of opening to traffic until the moment it is wanted to be predicted, in years in decimal fraction, when 0,5 means 6 months.

TotVeh is the total accumulated heavy vehicles that have circulated through the section since it was opened to traffic until the moment wanted to be predicted in both direction of the two-lane road, in thousands of vehicles.

VehH.Tot is the total accumulated heavy vehicles that have circulated in the period considered through the project lane (the lane with greater quantity of heavy vehicles in the section) since it was opened to traffic until the moment wanted to be calculated, in thousands of heavy vehicles. Usually, both directions are supposed to have identical heavy traffic, the half of the total.

SUBf is afactor that takes into account the possible combination of non bituminous materials that can be extended in the subbase. Its value is obtained from Table 8.37.

Table 8.37. Values for factor SUBf to be introduced in Eq. 8.26, according to the non bituminous materials included in the subbase layers

		Materials in subbase layer 1		
		Cement treated base (gravel and cement) (gravacemento)	**Soil-cement**	**Gravel and slag (Gravaescoria)**
Materials in the subbase layer 2	**Crushed stone**	-0,367	-	-0,198
	Soil-cement	-0,532	-	-
	Vitrified slag	-	-	0,038
	Nothing (no 2^{nd} layer)	-0,330	-0,649	0

Only the combinations that have a value in Table 8.37 can be deployed. If there is no value, that combination was not found.

Table 8.38 shows the test of Between-Subjects effect for the model of Eq. 8.26, where the significance of individual variables can be seen. The intercept have low significance (p-value = 0,318). However, it was preferred to be maintained because if eliminated the coefficient of determination decreased. Table 8.39 presents the estimations of the parameters (coefficients) of the model of Eq. 8.39.

Table 8.38. Test of Between-Subjects effects for model of Eq. 8.26.

Origin	**Type III Sum of Squares**	**Degree of freedom**	**Mean Square**	**F**	**Sig.**	**Partial eta-squared**	**Non centrality parameter**	**Observed Power**
Corrected model	10,749	9	1,194	7,999	<0,001	0,503	71,993	1,000
Intercept	0,151	1	0,151	1,012	0,318	0,014	1,012	0,168
PotR.Age	4,127	1	4,127	27,643	<0,001	0,28	27,643	0,999
TotVeh	2,541	1	2,541	17,017	<0,001	0,193	17,017	0,983
TotH.Veh	0,825	1	0,825	5,522	0,022	0,072	5,522	0,640
SUB1(f)*SUB(2)	3,126	6	0,521	3,49	0,004	0,228	20,938	0,929
Error	10,601	71	0,149					
Total	386,44	81						
Corrected total	21,35	80						

Table 8.39. Parameter estimates for model of Eq. 8.26.

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	U`pper			
Intercept	-0,217	0,518	-0,419	0,677	-1,25	0,816	0,002	0,419	0,07
PotR.Age	1,341	0,255	5,258	0	0,832	1,849	0,28	5,258	0,999
TotVeh	-2,21E-05	5,35E-06	-4,125	0	-3,28E-05	-1,14E-05	0,193	4,125	0,983
TotH.Veh	1,74E-04	7,40E-05	2,35	0,022	2,63E-05	3,21E-04	0,072	2,35	0,64
[SUB1=12] * [SUB2=11]	-0,367	0,235	-1,565	0,122	-0,835	0,101	0,033	1,565	0,339
[SUB1=12] * [SUB2=13]	-0,532	0,305	-1,744	0,086	-1,14	0,076	0,041	1,744	0,405
[SUB1=12] * [SUB2=20]	-0,33	0,133	-2,49	0,015	-0,595	-0,066	0,08	2,49	0,69
[SUB1=13] * [SUB2=20]	-0,649	0,166	-3,91	0	-0,98	-0,318	0,177	3,91	0,971
[SUB1=14] * [SUB2=11]	-0,198	0,126	-1,566	0,122	-0,449	0,054	0,033	1,566	0,339
[SUB1=14] * [SUB2=15]	0,038	0,243	0,156	0,876	-0,447	0,523	0	0,156	0,053
[SUB1=14] * [SUB2=20]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

Fig. 8.8 presents the scatterplots by levels and provides graphic information about the variance homogeneity and allows detecting the possible existence of any type of relation between the size of the means and the size of the variances. As the variances are not equal, points in Fig. 8.8a and 8.8b are not horizontally aligned

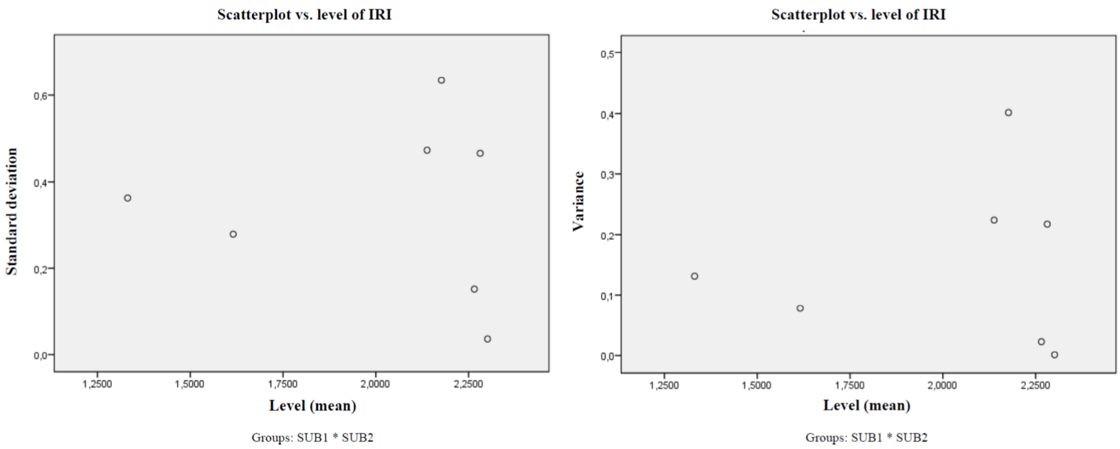


Fig. 8.8. Scatterplots by levels of Eq. 8.26 a) Standard deviation, b) Variance

The plot of residuals of Fig. 8.9 allows observing that they are random and independent between them. As the plot of predicted values vs standardized residuals is random (there are not any pattern), the residuals are independent. The residual variances are homogeneous since the dispersion of the standardized residuals is similar along all the values of predicted values. Predicted and observed values show a linear pattern, which is reflected by the coefficient of determination.

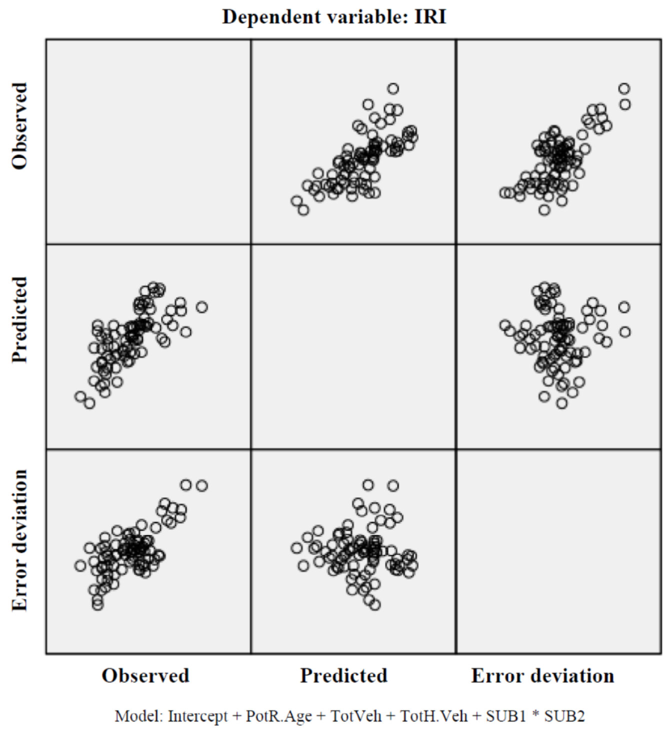


Fig. 8.9. Plot of residuals (standardized), observed and predicted values of the model of Eq. 8.26.

In Fig. 8.10 a more detailed plot of observed values vs. predicted values for Eq. 8.26 is shown.

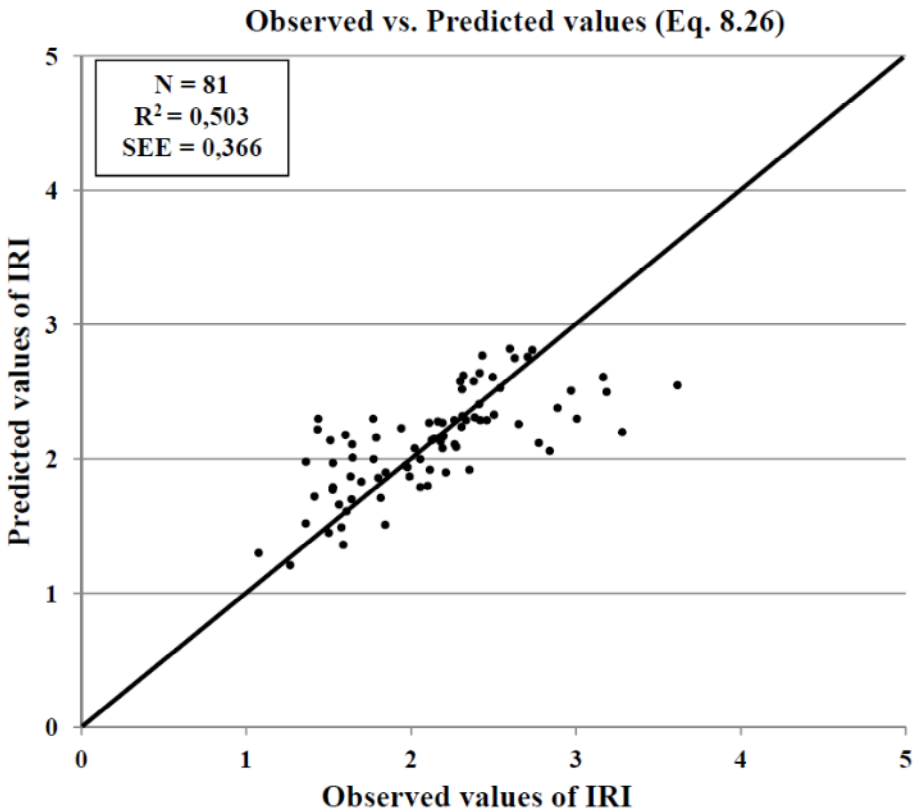


Fig. 8.10. Observed vs. predicated values with Eq. 8.26.

The proposed model for IRI prediction in new two-lane roads with semi-rigid pavement includes the age of the pavement, the total accumulated number of total vehicles and heavy vehicles and a factor to take into consideration the different materials that can be introduced in the one or two layers that can compose the subbase of the semi-rigid pavement. In this model, the loads over the pavement are represented by a combination of the accumulated total vehicles and heavy vehicles, fact that can be interpreted as a higher influence of heavy vehicles, as it could be expected. For taking into account the strength of the semi-rigid pavement structure a factor that considers the materials of the subbase is needed. It does not need the thickness of these layers, since they are usually designed following the standard in force. It could be included in the model a factor that considered the materials of the layers of the subbase and the thickness of the bituminous layers. However, the model does not show any improvement and, since pavement design are conducting according to the standards, it was estimated that it was not necessary

8.6. Proposed IRI performance models for pavements classified as Maintenance and Rehabilitation

Due to the low quantity of pavements classified as "Maintenance and Rehabilitated", both in flexible pavements and in semi-rigid pavements, it has considered not to be enough to develop a reliable model. Therefore, inclusion of stretches of double carriageway roads would be necessary to have enough quantity of available data. This idea is included as future line of research (section 10.2).

8.7. Conclusions

IRI performance models for new two-lane roads with flexible and semi-rigid pavements for the road network of Biscay were developed. Individual models for flexible and semi-rigid pavements were considered due to the different performance.

For flexible pavement, a multiple linear regression was adopted, which included the age of the pavement since it was constructed, the total thickness of the bituminous layers and the accumulated total number of heavy vehicles as the necessary variables, which are usual factors in the literature.

For new two-lane roads with semi-rigid pavements, the proposed equation for IRI prediction include the age of the pavement, the total accumulated number of total vehicles and heavy vehicles and a factor to take into consideration the different materials that can be introduced in the one or two layers that can compose the subbase of the semi-rigid pavement.

Chapter 9. SCRIM Coefficient performance models for pavement roads in Biscay

9.1. Introduction

In this chapter, SCRIM Coefficient performance models for the road network of Biscay are developed. Researches about pavement friction have demonstrated that, after a new road is opened to traffic, an initial friction improvement appears due to the elimination of the bituminous film and aggregates are exposed to traffic. Then, immediately after the peak (Fig. 5.31), the surface is said to be polished quickly, firstly with a high rate of loss of skid resistance and later a slower rate until an equilibrium point. In the equilibrium phase, the values are said to be constant and only seasonal variation are registered, with higher values in winter and lower ones in summer.

There have been proposed some models to quantify that seasonal variation, but it seems more interesting to predict the friction values at their minimum, to know the situation at its worst. As observed in the literature (section 5.5), the skid resistance models of pavements in situ depends on two variables: the heavy traffic volume (expressed according to varying definitions) and the polished stone value (PSV) of the aggregates (Szatkowski and Hosking, 1972; Roe and Hartshorne, 1998; Transit New Zealand, 2002a; NZTA, 2013a). The last one indicates the resistance of the aggregate to be polished, while the first one represents the loads that are applied that polish the aggregates. Laboratory tests have shown the same tendency, quick polishing initially, later a lower rate and finally, an equilibrium point where more and more cycles of polishing do not decrease the skid resistance. That curve can be calculated from aggregate properties like gradation, size, etc. However, those values are not measured for field measurements or as required characteristics for aggregates to be used in real work projects.

Firstly, an initial skid resistance model for new roads in Biscay was developed with the SCRIM data collected in winter. Secondly, different models were aimed to be proposed with summer friction data. Pavement families presented in Chapter 8, New outline pavement and rehabilitated or maintained pavements, are considered as a first approach to start the analysis.

9.2. Skid resistance model for new roads in Biscay with winter data

Before obtaining the skid resistance (and IRI) data from 2016, which were received in 2017, a skid resistance model was developed by means of the data collected in winter 2011. As commented in Chapter 6, the RGB collected pavement condition data in 2000, 2002 (partially), 2004, 2007 and 2011. The date of the skid resistance data collections in the years 2000, 2004 and 2007 is not registered. They are said to be collected in summer, but no specific data is recorded in the database, so they cannot be employed for calculation reliablely. Data from 2011 was recorded to be collected in February and March 2011, in winter, when friction values can be said to be at their maximum (Burchett and Rizengers 1980; Echaveguren and Solminihac, 2011).

When developing this prediction model, in order to avoid interferences from previous road structure or rehabilitation and maintenance works, 16 stretches of new highways (interurban), constructed in the last 25 year, were selected for the analysis (Pérez-Acebo *et al.*, 2017a). They have no rehabilitation or maintenance works until 2011. Table 9.1 shows the selected road stretches with their main characteristics. All roads are conventional roads, i.e., two-lane roads.

Table 9.1. Selected roads for the skid resistance model with winter data (Pérez-Acebo et al., 2017a).

Road denomination (code)	**Road network**	**Surface layer**	**Pavement structure [a]**	**Length (km)**	**Date of opening to traffic**
N-240 (I)	Preferential Interest	AC 16 surf S	SR	0,70	24/05/2002
BI-625	Basic	AC 16 surf S	FL	1,04	01/10/1994
BI-633 (I)	Basic	AC 16 surf S	FL	1,34	08/01/2003
BI-633 (II)	Basic	AC 16 surf S	FL	2,535	07/08/2002
BI-647	Basic	AC 16 surf S	FL	1,40	01/06/2007
BI-2121	Provincial	AC 16 surf S	FL	0,975	01/07/2000
BI-2238 (I)	Provincial	AC 16 surf S	SR	1,02	01/06/2009
BI-2238 (II)	Provincial	AC 16 surf S	SR	1,55	23/10/2009
BI-2224	Provincial	AC 16 surf S	FL	0,08	01/06/2007
BI-2405	Provincial	AC 16 surf S	SR	1,570	23/10/2009
BI-2522	Provincial	AC 16 surf S	SR	1,300	01/06/1997
BI-2713 (I)	Provincial	AC 16 surf S	SR	0,08	01/11/2003
BI-732	Complementary	BBTM 11A	SR	0,70	01/10/2005
BI-2713 (II)	Provincial	BBTM 11A	SR	2,260	03/08/2005
N-240 (II)	Preferential Interest	PA 11	SR	1,71	19/07/2002
BI-633 (III)	Basic	PA 11	FL	1,42	18/09/2000

[a] Pavement structure: FL, flexible ; SR, semi-rigid

Some stretches, with the same transverse section, were divided into various sub-stretches according to traffic volumes, due to some connections along their layout. Therefore, 23 different sections with different traffic volumes were studied.

There are three types of surface bituminous layers: discontinuous (BBTM 11A), Asphalt Concrete semi-dense (AC surf S) and porous asphalt (PA 11). With regard to pavement structure, flexible (FL) and semi-rigid (SR) pavements are included.

The following variables were included as possible factor for skid resistance prediction.

- ***Age*.** The years since construction of the road to 2011 were incorporated as affecting factor. It was calculate as the difference between 2011 and the year of construction. Since each road was constructed at a different moment in time, the data collections were carried out at different pavement ages.
- **Total thickness of the bituminous layers (*TotBit*).** The thickness of the bituminous layers was also included in cm, as possible independent variable. Since all the pavement sections were included as new outline pavements, the complete structure is known in all of them.

- **Average Annual Daily Traffic (*AADT*).** The annual average daily traffic, which counts all the vehicles that have crossed the section per day on average, was also introduced in the calculation.
- **Heavy Average Annual Daily Traffic (*H.AADT*).** The traffic volume of heavy vehicles was also included. In Spain, as commented in Chapter 8, a vehicle is classified as a heavy vehicle when its weight is over 3.500 kg (Ministerio de Fomento, 2003b). Each road is classified by means of H.AADT (heavy vehicles/day/lane) in the project lane when the road is opened to traffic (Table 9.2) (Ministerio de Fomento, 2003b).

Table 9.2. Heavy traffic categories in Spain (Ministerio de Fomento, 2003b).

Traffic category	H.AADT (heavy veh./day/lane)	Traffic category	H.AADT (heavy veh./day/lane)
T00	H.AADT ≥ 4.000	**T31**	200 < H.AADT ≤ 100
T0	4.000 < H.AADT ≤ 2.000	**T32**	100 < H.AADT ≤ 50
T1	2.000 < H.AADT ≤ 800	**T41**	50 < H.AADT ≤ 25
T2	800 < H.AADT ≤ 200	**T42**	25 < H.AADT

- **Required Polished Stone Value (*PSV_{req}*).** As shown in Chapter 5, the PSV has been included in skid resistance prediction models for real roads as a key factor, which represents the resistance of the aggregates to be polished (Szatkowski and Hosking, 1972; Roe and Hartshorne, 1998; Transit New Zealand, 2002a; NZTA, 2013a). Nevertheless, the exact PSV of aggregates in each highway is unknown for roads in Biscay. Spanish regulations do indicate the minimum PSV required for each asphalt concrete and traffic category. At the time of the projects of the selected roads (Table 9.1), for AC 16 surf S mixes a minimum value of 0,50 for T0 and T1 categories was indicated, with a figure of 0,45 for T2 and 0,40 or the T31-42 categories. For discontinuous and porous mixes, 0,50 was established for the T0 – T2 range and 0,45 for the T31-T42 range (Ministerio de Fomento, 2001, 2004a; Ministerio de Obras Públicas y Urbanismo, 1989b). At present, the regulation indicates that the PSV is expressed in a scale from 0 to 100 (Ministerio de Fomento, 2015) and in this scale was introduced in the modelling.

Regarding the dependent variable, the skid resistance value, obtained by means of the SCRIM Coefficient, is employed. The average value for each of the 23 stretches used in the studied is calculated. But these data indicate the maximum value that skid resistance can achieve in winter. It seems more interesting to know skid resistance values in summer, when it is at its minimum and variations are at the least, as the **Mean Summer SCRIM Coefficient (*MSSC*)** employed by the British Highway Agency, representing the mean of three Side-force coefficient (SFC) measured in summer (section 5.4.5.5) (Hosking and Woodford, 1976b). With regard to the data of Biscay, supposing that data in 2000, 2004 and 2007 were collected in summer, the seasonal variation for each surface layer type can be calculated (Table 9.3).

Table 9.3 SFC variation range in Biscay and in Gipuzkoa (Navarro et al., 2011).

Surface layer	Variation range in Biscay	Average SFC in Biscay	Variation range in Gipuzkoa	Average SFC in Gipuzkoa	Proposed reduction
BBTM 11A	14,7	65	18	55	13,5
PA-11	11,3	60,1	17	54	13
AC16 surf S	17,1	54,7	12,5	50	9

These range of variations (from winter to summer) were compared to the values obtained in a research carried out in Gipuzkoa (Spain), where SCRIM values were collected for previously rehabilitated roads every 3 months over the course of 2 years (Navarro *et al.*, 2011). Gipuzkoa is another province of the Basque Country, also located at the seaside, with similar extent and oceanic climate as Biscay. Navarro *et al*. (2011) observed an initial decrease during the first 12 months and subsequent seasonal variations, whit the lowest value in June (since there were no data in July or August) (Table 9.3).

As observed, the average range variation registered in Biscay approximates to those obtained in Gipuzkoa (Navarro *et al*., 2011). There is not a perfect correspondence because not all the stretches of the study in Biscay have been measured in all data collections (2000, 2004 and 2007). These seasonal variations are within the range obtained by Echaveguren and Solminihac (2011) for asphalt concrete, between 8 and 25. The reduction proposed for each surface, from winter value to mean summer minimum value, is shown in the last column of Table 9.3. It is 3/4 of the registered variation of Gipuzkoa, as these data include the initial decrease of friction value (Fig. 5.38), because they were recorded from the beginning of the roads' service. Therefore, the average value of the selected roads stretches in winter 2011 is reduced to the quantity indicated in the last column of Table 6.3, according to the bituminous mixing type in the surface layer, to obtain the Mean Summer SCRIM Coefficient (MSSC).

Then, a multiple linear regression was performed between the MSSC, as a dependent variable (VD) and the possible independent variables (VI) that can affect the value: *AADT*, *H.AADT*, *Age*, total bituminous thickness (*TotBit*) in cm and required PSV (PSV_{req}). The influence of each variable was assessed using forward stepwise regression analysis, by means of Version 24 of the SPSS software. When performing a regression analysis some assumptions are made, summarized in chapter 7.

Some transformations were conducted to obtain a normal distribution of the variables and to have a greater linearity between the dependent and independent variables. Specifically, although there is correlation between *MSSC* and *AADT* and *H.AADT* (Table 9.4), the last ones were transformed by applying natural logarithm, square root and inverse, following the ideas from commented researches (Eq. 5.69 and Eq. 5.71) and applying similar techniques as other authors when analysing frictional data (Ongel *et al*., 2009), in order to obtain a normal distribution of the data. Table 9.4 shows the Pearson coefficient, *R*, between variables, indicating those with a high level of significance.

After the multiple regression analysis, it appeared that the best result could be obtained excluding *AADT*, *Age* and *TotBit*. As a result, Eq. 9.1 was proposed for the prediction of skid resistance with a 95% of confidence level (Pérez-Acebo *et al*., 2017a).

$$MSSC = 30{,}19 - 0{,}82 \cdot \sqrt{H.AADT} + 0{,}76 \cdot PSV_{req} \qquad [9.1]$$

Where *H.AADT* is expressed in heavy vehicles/day/lane and PSV_{req} is expressed on a scale from 0 to 100. *MSSC* is also expressed on a scale from 0 to 100.

As seen, the only parameters employed in the model are the square root of *H.AADT* and the required *PSV*. As shown in Table 9.5, the determination coefficient (R^2) of the model is 0,696, indicating that the formula can explain 70% of the total variance of the 23 sections. The Durbin-Watson statistic is 1,499, certifying the independence of errors. Fig. 9.1a shows that the residuals follow a normal distribution and Fig. 9.1b shows that the residuals do not follow any pattern, certifying the homoscedasticity. The Variance Inflation Factor (*VIF*) has a value of 1,17; certifying no problem of colinearity. An *F* test shows that the correlation is true ($p < 0{,}01$) and an analysis using *Student's t* test of the coefficient shows that they are real, and different from 0 (Table 9.5)

The proposed model does not consider the age of the pavement, which showed a low correlation with MSSC ($R = 0{,}06$), corroborating the ideas from the TRL (Hosking and Woodford, 1976b), and the shape of Fig. 5.31 was verified. In equilibrium phase, the only variations were seasonal, and the minimum value remains constant through years. Therefore, this certifies the expressions developed by Rezaei and Masad (2013) and Kassem *et al*. (2013), underlining that, after a number of polishing cycles, an asymptotic value was reached. If the pavement surface is more than 2 years old, age must not be regarded as an influencing factor.

Table 9.4. Correlations among variables (coefficient of Pearson, R) (Pérez-Acebo et al., 2017a)

	MSSC	AADT	H.AADT	Ln(AADT)	Ln(H.AADT)	Inv(AADT)	Inv(H.AADT)	Sqrt(AADT)	Sqrt(H.AADT)	PSV_{req}	Age	Tot Bit
MSSC	1,00											
AADT	-0,66**	1,00										
H.AADT	-0,75**	0,70**	1,00									
Ln(AADT)	-0,63**	0,90**	0,65**	1,00								
Ln(H.AADT)	-0,73**	0,75**	0,89**	0,86**	1,00							
Inv(AADT)	0,47*	-0,63**	-0,48*	-0,89**	-0,79**	1,00						
Inv(H.AADT)	0,49*	-0,59**	-0,56**	-0,86**	-0,85**	0,98**	1,00					
sqrt(AADT)	-0,66**	0,98**	0,69**	0,97**	0,82**	-0,76**	-0,726**	1,00				
sqrt(H.AADT)	-0,77**	0,74**	0,98**	0,75**	0,96**	-0,62**	-0,693**	0,77**	1,00			
PSV_{req}	0,02	0,22	0,41	0,25	0,37	-0,37	-0,409	0,22	0,38	1,00		
Age	-0,06	-0,13	0,05	0,00	0,16	-0,12	-0,225	-0,07	0,10	-0,12	1,00	
Tot Bit	-0,33	0,09	0,47**	0,29	0,59**	-0,41	-0,527**	0,18	0,55**	0,20	0,18	1,00

*p < 0.05; ** p < 0.001

Table 9.5. Analysis of variance of the model (Pérez-Acebo et al., 2017a)

Analysis of variance						
Source	**Degrees of freedom**	**Sum of squares**	**Mean Square**	***F* value**	***p-value***	**Durbin-Watson**
Model	2	985,122	492,561	22,932	< 0.001	1,499
Error	20	429,581	21,479			
Corrected total	22	1414,704				
Root MSE*	**Dependent mean**	**Coeff. var.**	***R***	***R^2***	**Adj. *R^2***	***VIF***
6,69166	51,1484	13,08	0,834	0,696	0,666	1,172
Parameters estimates						
Variable	**Parameter estimate**	**Standard error**	***t* value**	***p-value***	**95 % confidence limits**	
Intercept	30,188	12,394	2,436	0,024	4,335	56,042
Sqrt(H.AADT)	-0.824	0,122	-6,771	< 0,001	-1,077	-0,570
PSV	0,759	0,280	2,713	0,0013	0,175	1,342

* Root mean square error

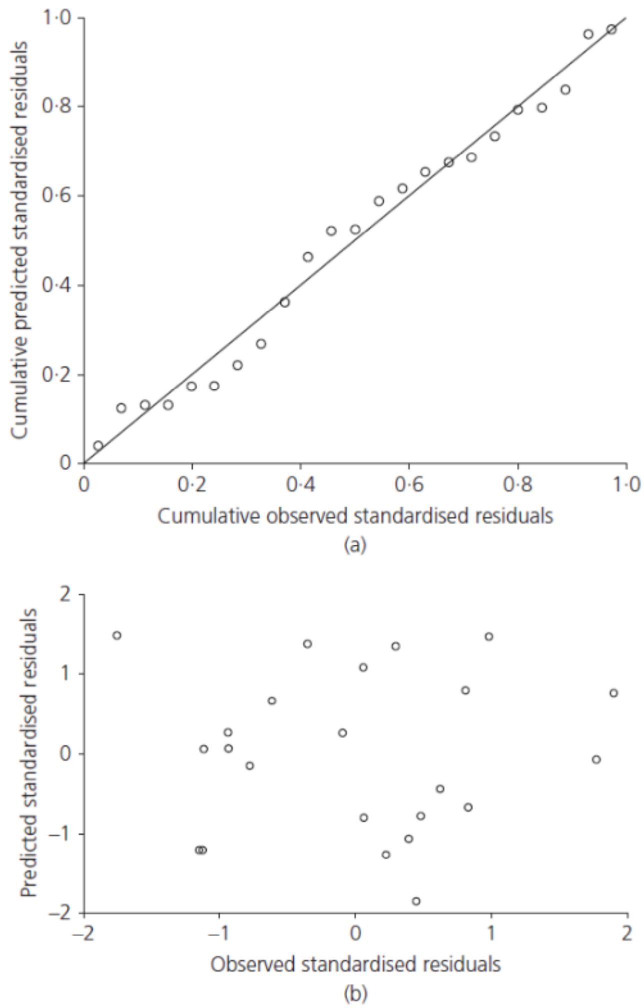


Fig. 9.1. Analysis of the residuals of the proposed model: a) adjustment of residuals to standardized normal distribution; b) observed standardized residuals plotted against predicted standardized residuals (Pérez-Acebo et al., 2017a)

Furthermore, the total thickness of the bituminous layers does not influence the skid resistance, as it could be anticipated, since friction is a surface characteristic. The correlation with *MSSC* could be considered as not very low (R = - 0,33) (Table 9.4), but the reason is that pavements with higher heavy traffic volumes are designed with thicker bituminous layers and the regression analysis excluded this variable from the model. Additionally, the surface layer material (porous asphalt, discontinuous mixes and asphalt concrete mixes with semi-dense gradation) does not reflect a great importance. It must be considered to estimate the difference between the peak and the lower values, but not to define the low value, which was demonstrated to be only dependent on heavy traffic and required *PSV*. The pavement structure type, flexible or semi-rigid, did not show variations in the proposed model. When gathering pavement structures according to their type, similar models were obtained.

As any other model, Eq. 9.1 allows predicting the minimum SFC obtained in a required PSV. For example, if the RGB establishes a higher threshold for skid resistance than the value predicted for that heavy traffic volume, a higher value of *PSV* must be required in the aggregates used in the surface layer (Fig. 9.2). It has to be mentioned that the model was only valid for *H.AADT* values under 1.400 heavy vehicles/day/lane; the approximate maximum figures for two-lane roads.

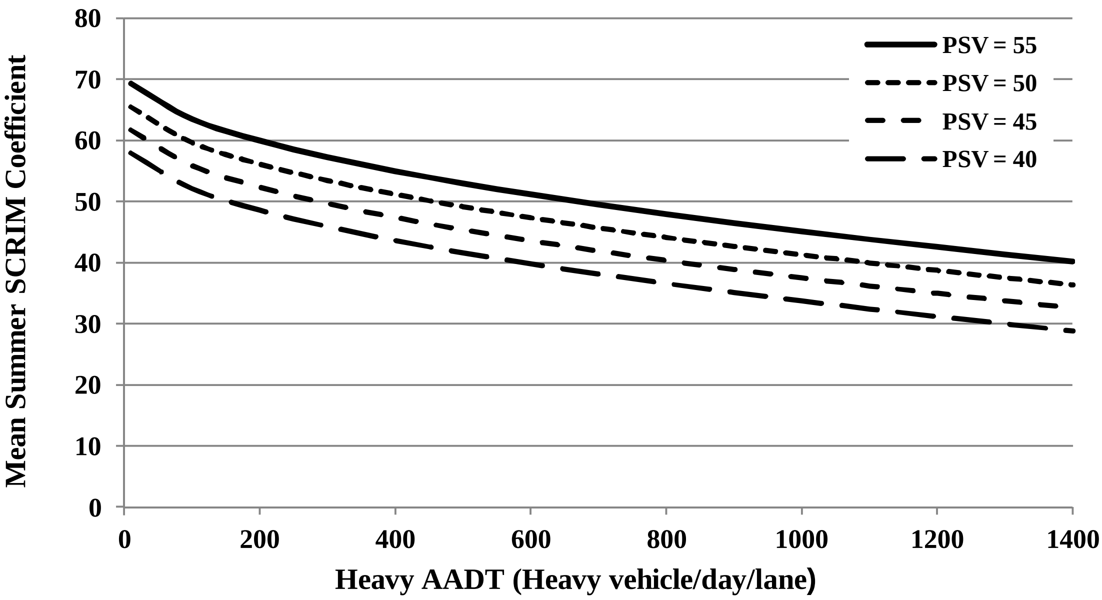


Fig. 9.2. Prediction of Mean Summer SCRIM Coefficient (MSSC) by means of H.AADT and PSV_{req} (Pérez-Acebo et al., 2017a)

9.3. Skid resistance models for roads in Biscay with summer data

In the previous section, a skid resistance model was developed to predict the Mean Summer SCRIM Coefficient (*MSSC*) with winter data. In that case, a seasonal variation had to be estimated, and subtract it to winter data of friction, which are said to be at their maximum levels.

In order to develop a more accurate skid resistance model, real summer data from 2016 were employed. As commented, skid resistance is said to reach the minimum value in summer due to some reasons explained in section 5.4.5.5 and, additionally, its variance is lower (Cenek *et al.*, 1999). Consequently, as adopted by the

British road agency friction data are recommended to be collected in summer (Britain, 2015; Highway Agencies, 2015). Therefore, the RGB collected data in 2016 in summer and these data are employed for obtaining a more accurate prediction model for skid resistance in the roads of Biscay.

9.3.1. Skid resistance prediction model with completely known sections

For developing the skid prediction models for the road network of Biscay, the analysis of the roads conducted for the roughness performance, where the entire pavement section is known, was deployed again. It was followed the project analysis carried out to identify the completely registered pavement structures in the data set of the PMS and the analysis of IRI data was considered, which could lead to discard some stretches if they have better IRI values and no rehabilitation or maintenance works are registered. Adopting these criteria, stretches with completely known pavement structure were identified, with initial pavement and possible rehabilitation and maintenance works. Hence, similarly to the works exposed for each road in the road files of Annex II in section “IRI”, in section “Skid resistance”, the stretches where the complete section is known are identified, indicating their initial and final Marker Posts (Kilometre Point, KP). There are two parts. Stretches with a section completely identified in 2016 are indicated in section 2016. Additionally, as the date of the data collection in 2011 is known, mainly in winter 2011 (in February and March 2011), it was decided to also indicate the stretches with a known pavement structure in 2011 in order to compare the minimum skid resistance values (in Summer, with data from 2016), with the maximum skid resistance values (in Winter, with data from 2011). However, for a real comparison, the same structure must be conserved from 2011. As the main attention is on data from 2016, only stretches that had in 2011 the same section as in 2016 are included and hence, their number is considerably lower. These stretches from 2011 are indicated in section “2011”.

The reason for selecting stretches that had a completely know pavement section was that it was wanted to be know if the pavement structure, flexible or semi-rigid, the quantity of layers, their thickness and materials employed, really influence or not in the available friction. Previous work with data from 2011 (Pérez-Acebo *et al*., 2017a) and literature review (Flora, 2009; Prang *et al*. 2012) indicate that there is not a correspondence between pavement structure and skid resistance. Then, if the results show that the structural properties of the pavement are not influencing skid resistance data, a different point of view for the research will be adopted.

Therefore, the information employed for the skid resistance analysis is described below (Table 9.6), which shares some data with the IRI analysis:

- *Identification of the stretch*: The same columns as for IRI analysis, described in section 8.4.1, were employed: columns A to G from Table 8.4. They indicate the road denomination (column A), the axle (column B), the intial and final KP (column C and D, respectively) and Column E to G to introduce the exact date of the data collection, the year and the year in numer to calculate the real age of the pavement.
- *Skid resistance data*: **SCRIM Coefficient** (*column S1*) for the considered stretch of 20 m, as explained in 6.6.2, expressed from 0 to 100; and the **Texture** (*column S2*), in mm, representing the Mean Profile Depth, obtained by a laser-base device (Table 9.6).

Table 9.6. Example of some stretches of road BI-631 with stretch indentification and skid resistance data.

A	B	C	D	E	F	G	S1	S2
Road denom	**Axle**	**Initial KP**	**Final KP**	**Exact date data collection**	**Data collection year**	**Data collection year, in number**	**SCRIM Coefficient**	**Texture (mm)**
BI-631	BI-631 Unique	23+0334	23+0353	30/06/2016	2016	2016,5	38	0,42
BI-631	BI-631 Unique	23+0354	23+0373	30/06/2016	2016	2016,5	33	0,39
BI-631	BI-631 Unique	23+0374	23+0393	30/06/2016	2016	2016,5	38	0,45
BI-631	BI-631 Unique	23+0394	23+0413	30/06/2016	2016	2016,5	38	0,45

- *Traffic data*: the same information as for IRI data are incorporated, as shown in Table 8.5: data from the year of the data collection, generally 2016, including Annual Average Daily Traffic (AADT) (*column T1*), Heavy Annual Average Daily Traffic (H.AADT) (*column T2*) following the lane considerations exposed in Ministerio de Fomento (2003b) and traffic category, (*column T3*), the accumulated total vehicles that crossed the section since the data in which the stretch was opened to traffic (in the case of New Outline stretches) or since the data in which the M&R activity was finished) until the date of the data collection (*column T4*) and the accumulated total number of heavy vehicles since the data or opening or rehabilitation of the stretch untul the date of the data collection (*column T5*). For data from years previous to 2000, as indicated in section 8.4.1, an increasing rate of 2% each year was considered for total traffic and heavy traffic volumes.
- *Rainfall data*: In section 5.4.5.6 it was commented that previous rainfalls may affect available skid resistance in the pavement (Bird and Scott, 1936b, Henry 1981, 2000; Hill and Henry, 1981). As the RGB has included in the PMS the information of the rainfall data during the previous 15 days to the friction data collection in the area of the road from the nearest meteorological station, these data, **Rainfall data**, in mm, was also incorporated (*column S3*) as a possible influencing factor for friction prediction.
- *Polished Stone Value*. Other authors have included the Polished Stone Value of the used aggregates as an influencing factor. As commented in section 9.2, the exact value of the employed aggregates in each project is not available. However, it is known the minimum required value for each project according to the heavy traffic category in the year that the road is opened to traffic (or the M&R work is finished). Consequently, a column with the H.AADT of the year when the new outline or the M&R work was finished (*column S*4), the corresponding heavy traffic category according to Ministerio de Fomento (2003b) (*column S5*), and finally the required minimum PSV (*column S6*) (Table 9.7). The value of the required PSV is a function of the heavy traffic categories, the employed material in the surface layer and the laws and regulations in force in that moment of opening to traffic. Table 9.8 shows the minimum required PSV for each surface material layer (Asphalt concrete, discontinuous mixes, porous asphalt or slurries), the traffic category and the different regulations in force in each period.

Table 9.7. Example of some stretches of road BI-631 with data about stretch identification (not completely), skid resistance, rainfall and required Polised Stone Value.

A	C	D	S1	S2	S3	S4	S5	S6
Road denom	**Initial KP**	**Final KP**	**SCRIM Coefficient**	**Texture (mm)**	**Rainfall data (mm)**	**H.AADT in year open to traffic**	**Traffica category in year open to traffic**	**Required Polished Stone Value**
BI-631	23+0334	23+0353	38	0,42	34,3	249	T2	50
BI-631	23+0354	23+0373	33	0,39	34,3	249	T2	50
BI-631	23+0374	23+0393	38	0,45	34,3	249	T2	50
BI-631	23+0394	23+0413	38	0,45	34,3	249	T2	50

- *Structural data*: The guideline commented for IRI data analysis was followed. Data commented in section 8.4.2 were introduced for stretches classified as New Outline (Table 8.7, *columns I* to *Z*) and for stretches classified as “Maintenance & Rehabilitation”, data exposed in section 8.4.3 were included for friction analysis (*columns I* to *AM* of Table 8.8).

Initially, stretches from two-lane roads were only analysed, similarly to the work presented in 9.2. As the fact that some lanes are available for the traffic may provide a different polishing action on pavement, it was preferred to check the action in conventional roads, i.e. unique carriageway and double direction, as it is sure that the specified traffic volume goes through the lane in each direction. Later, after observing polishing effects on two-lane roads, it may be extended to double carriageway highways.

Consequently, as roads of Area 4 mainly consists of double carriageway roads, since motorways and multilane highways of the metropolitan area of Bilbao are included there, only two-lane roads from Areas 1, 2 and 3 are considered in this first step of the analysis. Existing double carriageway stretches in areas 1, 2 and 3 are discarded for this initial analysis.

Furthermore, similarly to IRI analysis, in a first approach it was tried to analyze the friction data of all the individual stretches of 20 m, as attempted to do with 100 m-stretches for IRI data. Once again, it was noted a wide variance within a stretch with the same predicting values. For example, for a stretch of 500 m with the same data characteristics (traffic volumes, age, and pavement section), 25 values of 20 m are available, it was observed a wide range of variation for friction data, which makes impossible to correlate all the data from all the stretches with known section. Consequently, once more, it was calculated the mean SCRIM Coefficient and texture for the stretches with similar characteristics (pavement structure, pavement age and traffic volumes) from the data of the 20 m-sub-stretches in which the information is presented. This type of analysis, the calculation of the mean value of a stretch with similar characteristics, is logical because the models proposed in the literature calculate the mean friction value obtained from some predicting variables and not the existing variance or range of variation. The analysis of the variability, as explained in the IRI analysis (section 8.4.4) is conducted in probabilistic models, which predict the percentage of the stretch that is in each predetermined state (Chapter 4). Therefore, a deterministic model, as the ones developed in this thesis, must provide the predicted mean value from the variables that are estimated to influence. To complete the variability analysis, within each stretch in which the mean value is calculated, the standard deviation is calculated (Eq. 9.2). Thus, it is possible to know the existing variance around the mean value.

Table 9.8. Required minimum Polished Stone Value for Asphalt Concrete mixes in Spain in different periods.

Materials	Regulation	Standard PSV test	Time in force	Heavy traffic categories	Required min. PSV	Required minimum SCRIM Coefficient
Asphalt Concrete (AC) mixes and Porous Asphalt (PA)	Art. 542. (MOP, 1976b)	NLT 174/72, NLT 175/73	1976 – 1989	Heavy traffic roads	0,45	-
				Rest of roads	0,40	
	Art 542. OC 299/1989 (MOPU, 1989b)	NLT 174/72	1989 - 2001	T0 and T1	0,50	0,65
				T2	0,45	
				T3 and T4	0,40	
	Art. 542. OC 5/2001 (MFOM, 2001)	NLT 174/72	2001 – 2004	T00	0,55	Porous asphalt, 60 Asphalt Concrete, 65
				T0, T1	0,50	
				T2	0,45	
				T3, T4, shoulders	0,40	
	Art. 542. FOM/891/2004 MFOM, 2004a)	Une 146136, Annex D	2004 – 2008	T00	0,55	Porous asphalt, 60 Rest, 65
				T0 and T1	0,55	
				T2	0,45	
				T3 , T4, shoulders	0,40	
Asphalt Concrete (AC) mixes	Art. 542. OC 24/2008 MFOM, 2008)	UNE-EN-1097-8	2008 – 2014	T00 and T0	56	65
				T1 – T31	50	
				T32, T4, shoulders	44	
	Art. 542 FOM/2523/2014 (MFOM, 2015)	UNE-EN-1097-8	2014 - present	T00 and T0	56	65
				T1 – T31	50	
				T32, T4, shoulders	44	
Discontinous mixes (BBTM A and B)	Art. 543. (MOP, 1976b)	NLT 174/72, NLT 175/73	1976 – 1989	Heavy traffic roads	0,45	-
				Rest of roads	0,40	
	Art 543. OC 299/1989 (MOPU, 1989b)	NLT 174/72	1989 - 2001	T0, T1	0,50	0,65
				T2	0,45	
				T3 and T4	0,40	
	Art. 543. OC 5/2001 (MFOM, 2001)	NLT 174/72	2001 – 2004	T00	0,55	BBTM A, 65 BBTM B, 60
				T0, T1 and T2	0,50	
				T3, T4, shoulders	0,45	
	Art. 543. FOM/891/2004 (MFOM, 2004a)	UNE 146136, Annex D	2004 – 2008	T00	0,55	BBTM A, 65 BBTM B, 60
				T0, T1 and T2	0,50	
				T3 , T4, shoulders	0,45	
Discontinuous mixes (BBTM A and B) and Porous Asphalt (PA)	Art. 543. OC 24/2008 (MFOM, 2008)	UNE-EN-1097-8	2008 – 2014	T00 and T0	56	BBTM A, 65 BBTM B and PA, 60
				T1 – T31	50	
				T32, T4, shoulders	44	
	Art. 543 FOM/2523/2014 (MFOM, 2015)	UNE-EN-1097-8	2014 - present	T00 and T0	56	BBTM A, 65 BBTM B, PA, 60
				T1 – T31	50	
				T32, T4, shoulders	44	
Slurries	Art. 540. (MOP, 1976b)	NLT 174/72, NLT 175/73	1976 – 1988	Any road	0,40	-
	Art 540. OC 297/1988 (MOPU, 1989b)	NLT 174/7, NLT 175/75	1988 - 2001	T0, T1 and T2	0,50	LB1, LB2, 0,65 LB3 0,60, LB4, 0,55
				T3, T4, shoulders	0,45	
	Art. 540. OC 5/2001 (MFOM, 2001)	NLT 174	2001 – 2004	T0, T1 and T2	0,50	LB1, LB2, 65 LB3, 60, LB4, 55
				T3, T4, shoulders	0,45	
	Art. 540. FOM/891/2004 (MFOM, 2004a)	UNE 146136, Annex D	2004 – 2011	T0, T1 and T2	0,50	LB1, LB2, 65 LB3, 60; LB4, 55
				T3 , T4	0,45	
				Shoulders of T3 and T4	0,40	
	Art. 540. OC 29/2011 (MFOM, 2011b)	UNE-EN-1097-8	2011 – 2014	T00	56	MICROF 11, 8: 65 MICROF 5: 60
				T1 – T31	50	
				T32, T4, shoulders	44	
	Art. 540 FOM/2523/2014 (MFOM, 2015)	UNE-EN-1097-8	2014 - present	T00	56	MICROF 11, 8: 65 MICROF 5: 60
				T1 – T31	50	
				T32, T4, shoulders	44	

Note: MOPU = Ministerio de Obras Públicas, MFOM = Ministerio de Fomento

$$s = \sqrt{\sigma^2} = \sqrt{\frac{\sum_{i=1}^{N}(x_i - \mu)^2}{N}} \qquad [9.2]$$

Where x_i is each SCRIM Coefficient of the 20 m-stretch observation, μ is the mean value for all the SCRIM Coefficient observation and N is the total number of observations.

Once again, as for IRI data, it has been selected the population standard deviation instead of the sample standard deviation (N-1 in the denominator) because the data represent the total population for that stretch with its specific characteristic, it is not a sample from a population but the total population itself.

Once the mean values of the stretches with similar characteristics in pavement section, age and traffic volumes have been calculated, 127 two-lane stretches are available from conservation areas 1, 2 and 3, including those from 2011 collected in summer (Table 9.9). Moreover, 11 stretches from double carriageway roads with completely known pavement structure were available.

The 15 values collected in summer in 2011 have been compared with those collected in summer in 2016. Table 9.10 shows the values for the same stretches and characteristics.

Table 9.9. Stretches with different values in considered variables, selected for SCRIM Coefficient analysis from conservation areas 1, 2 and 3.

	Area 1				Area 2				Area 3		
Road	**2L 2016**	**2C 2016**	**2011**	**Road**	**2L 2016**	**2C 2016**	**2011**	**Road**	**2L 2016**	**2C 2016**	**2011**
BI-631	4			BI-623	5	6		BI-624			
BI-634	2			BI-633	12			BI-625	9	3	
BI-635	4			BI-638				BI-630			
BI-735	1		1	BI-732	1			BI-712	1		
BI-737	1		1	BI-2224	5			BI-745			
BI-2101				BI-2301	2			BI-2521			
BI-2120	3			BI-2405	4			BI-2522			
BI-2121	3			BI-2543	8			BI-2617			
BI-2122	2		2	BI-2632				BI-2625	1		
Bi-2153	1			BI-2636				BI-2701	4		1
Bi-2235	5			N-240	15			BI-2757	3		3
Bi-2237	1			N-636				BI-2794	2		1
Bi-2238	5							N-639	5		4
BI-2704	2		2								
BI-2713	1										
BI-2731		2									
Total	35	2	6	**Total**	52	6	0	**Total**	25	3	9
			TOTAL 2016 Two-lane roads					112			
			TOTAL 2016 Double carriageway roads					11			
			TOTAL 2011					15			

Table 9.10. Comparison between SCRIM Coefficient values collected in summer in 2011 and in summer in 2016 for the same stretch.

Road	Initial KP	Final KP	AADT in 2011	H.AADT in 2011	SCRIM Coff in 2011	AADT in 2016	H.AADT in 2016	SCRIM Coef in 2016
BI-2122	24+0500	24+0600	12333	309	63,5789	12286	147	36,2000
BI-2122	27+0000	27+0090	8099	162	60,9333	9598	134	45,4000
BI-2701	25+0900	26+0200	4667	164	66,1636	4493	171	53,4375
BI-2704	8+0500	9+0300	12888	303	72,1596	12052	229	48,2105
BI-2704	23+0400	23+0700	5214	126	86,5566	5273	119	44,7143
BI-2757	0+0000	1+0120	3411	197	64,4468	3718	279	49,8214
BI-2757	2+0820	4+0700	1302	27	72,8235	2243	56	49,3511
BI-2757	4+0700	6+0840	1302	27	60,4000	2243	56	62,3714
BI-2794	23+0850	24+0780	3931	443	58,7500	4175	376	55,2093
BI-735	9+0960	10+0300	15897	636	85,5676	15385	423	46,7778
BI-737	13+0990	14+0150	11355	540	72,8000	12046	301	44,8000
N-639	21+0360	22+0790	4407	177	63,4545	4392	209	61,5135
N-639	22+0790	23+0760	4407	177	63,5789	4392	209	53,4894
N-639	23+0760	23+0860	4407	177	60,9333	4392	209	54,1667
N-639	23+0860	24+0170	4407	177	66,1636	4392	209	44,2000

As observed, following the ideas of seasonal variation commented in Chapter 5, a big variance is detected between values in 2011 and 2016, supposing that those of 2011 were also collected in summer. Their range variation is similar to the one exposed in Table 9.3 and also similar to the range observed between same stretches with data from winter 2011 and summer 2016. Therefore, although it is registered in the PMS that data were collected in summer 2011, they are rejected as spurious due to the big difference with summer values of 2016, which leads to think that they were collected in winter as the rest of the friction data of 2011

Consequently, 112 observations of completely known pavement structures are analyzed to develop a skid resistance prediction model. From those 112 observations, 49 correspond to stretch classified as New Outline, and 63 to stretches classified as Maintenance and Rehabilitation.

The variables introduced in the model are the following:

- The predicted variable, or dependent variable, is the SCRIM Coefficient, indicating its minimum value, obtained in summer. It is referred as **Mean Summer SCRIM Coefficient** (*MSSC*), following the terminology of the Transportation Research Laboratory (chapter 5).
- Among the possible predicting variables, or independent variables, some are quantitative and other qualitative. The quantitative independent variables are:
  - **Annual average daily traffic**, *AADT,* of the year of the data collection
  - **Annual average daily traffic of heavy vehicles** (> 3.500 kg), *H.AADT*, in the project lane (the lane with more heavy traffic) in the year of the data collection.
  - **Age**, Age, the difference between the year of the data collection and the year when the new outline of the rehabilitation or maintenance work was finished.
  - **Real age**, *R.Age*, the difference between the year of data collection and the year when the new outline of the rehabilitation or maintenance work was finished, expressing them in decimal format. For example, if the data collection is carried out in the final days of June

2016, it is computed as 2016,5. Thus, it is possible to know a more accurate age of the pavement.

- **Total vehicles**, *TotVeh*. It indicates the total number of vehicles that have crossed the section since it was first constructed (in case of new outlines) or when it was rehabilitated or maintained until the data collection, in thousand of vehicles.
- **Total heavy vehicles**. *TotHVeh*. Similarly to Total Vehicles but referred to the heavy traffic in the project lane of the stretch, in thousand of heavy vehicles.
- **Required Polished Stone Value**, PSV_{req}, it indicates the required PSV of the aggregates in the surface layer according to the surface layer material, the heavy traffic category of the stretch (Ministerio de Fomento, 2003b) and the year when it was finished (Table 9.8).
- **Rainfall data,** *Rain15*, the rainfall data, in mm, in the 15 days before the data collection in the nearest meteorological station to the road.
- **Total thickness of bituminous layers**, *TotBit*, indicating the total sum of the thickness of the bituminous layers in the pavement section, in cm.

The qualitative predicting variables are:

- **Pavement type**, *PaveType*, distinguishing between the two possibilities, flexible pavement (1) or semi-rigid pavement (2). It must be said that more information about the complete pavement structure is known, but initially an analysis observing the influence of the pavement type is only going to be conducted.
- **Surface denomination**, *SurfDen,* and **surface type**, *SurfType*, distinguishing between the following possible surface layer denomination and if, gathered according to similar properties, the possible surface layer type (Table 9.11).

Table 9.11. Possible surface layer types and their surface groups

Surface denomination (SurfDen)	Surface type (SurfType)
AC 16 surf S (1)	Asphalt concrete (AC) (1)
AC 22 surf S (2)	Asphalt concrete (AC) (1)
AC 16 surf D (3)	Asphalt concrete (AC) (1)
AC 22 surf D (4)	Asphalt concrete (AC) (1)
BBTM 11A (5)	Discontinuous mixing and porous asphalt (BBTM&PA) (2)
BBTM 11B (6)	Discontinuous mixing and porous asphalt (BBTM&PA) (2))
PA 11 (7)	Discontinuous mixing and porous asphalt (BBTM&PA) (2))
LB2 (8)	Slurry (3)

The different surface layer denomination (SurfDen), column in the left of Table 9.11, were grouped as indicated in the column in the right because, in some analysis, few data exist in some of the surface denomination and it has been preferred to group them according to similar characteristics. For example, although discontinuous mixings and porous asphalts are different, they share some characteristics; they have the ability to drain rain water and are defined in the same article (543) of the Spanish regulations (Table 9.8). The same occur with asphalt concrete mixes; they have different gradation, Semi-dense, (S), or Dense (D), they share some attributes. Consequently, they were grouped to avoid groups with only one or two data.

9.3.1.1. Analysis of data for New Outline stretches

There are 49 observation with completely know pavement structure. 19 observations correspond to flexible pavements and 30 to semi-rigid. With regard to the surface layer, 27 observations include Asphalt Concrete, AC, (mainly AC 16 surf S) and 20 include discontinuous mixing or porous asphalt. It has decided to anlayze the influence of surface layers according to the categories of SurfType (Table 9.11).

Firstly, an explanatory analysis of the data is carried out. The main statistics of each variable are conducted so as to see its distribution. Table 9.12 shows the main statistics for the quantitative variables. The Normal distribution of the variables has been checked by means of the statistical of Shapiro-Wilk and the statistical of Kolmogorov-Smirnov, which is said to be more adequate for samples of more than 30 observations (Table 9.13). As observed, MSSC is the only variable that follows a normal distribution, verified by the Kolmogorov-Smirnov and the Shapiro-Wilks statistics.

Table 9.12. Exploratory analysis for the quantitative variables in the stretches classified as New Outline

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Mean		50,39	8123,4	463,8	8,9	8,75	19535,99	1289,17	47,18	31,06	16,88
Standard error		1,09	1019,1	63,14	0,99	0,99	2785,77	200,07	0,59	1,92	0,81
95% CI for mean	**Lower**	48,2	6074,3	336,82	6,9	6,77	13934,8	886,92	46	27,2	15,24
	Upper	52,58	10172,5	590,73	10,9	10,74	25137,15	1691,43	48,37	34,91	18,51
Variance		58,11	50891283	195360	48,47	47,72	380264605	1961249	17,07	179,76	32,44
Std. deviation		7,62	7133,8	442	6,962	6,9	19500,4	1400,45	4,13	13,41	5,696
Minimum		27,33	669	23	1	0,8	1453,5	43,01	40	2,1	8
Maximum		64,62	31299	1408	29	28,6	98892	4447,04	50	45,8	33
Range		37,29	30630	1385	28	27,8	97438,5	4404,03	10	43,7	25
Interquartile range		9,4	7767	804	9	8,66	20228,3	2213,96	6	4,3	8
Skewness		-0,36	1,62	0,75	1,18	1,21	2,18	1,02	-0,96	-1,48	0,69
Kurtosis		0,69	2,53	-0,96	0,91	0,91	5,85	-0,22	-0,84	0,52	0,25

Table 9.13. Normality tests for the dependent and independent quantitative variables in the stretches classified as New Outline.

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
MSSC	0,089	49	0,200*	0,971	49	0,270
AADT	0,383	49	< 0,001	0,675	49	< 0,001
H.AADT	0,192	49	< 0,001	0,827	49	< 0,001
Age	0,242	49	< 0,001	0,816	49	< 0,001
R.Age	0,405	49	< 0,001	0,655	49	< 0,001
TotVeh	0,22	49	< 0,001	0,87	49	< 0,001
TotH.Veh	0,212	49	< 0,001	0,867	49	< 0,001
PSV	0,177	49	0,001	0,78	49	< 0,001
Rain 15	0,239	49	< 0,001	0,809	49	< 0,001
TotBit	0,14	49	0,018	0,936	49	0,01

[a] Lilliefors Significance correction

* This is lower bound of the true significance

The correlation between the independent variables and the dependent variable is carried out by means of the coefficient of Pearson, *R*, indicating the significance of that correlation (Table 9.14). As it is an explanatory analysis, only the coefficient of correlation between the dependent variable and each independent variable is shown, but not between the independent variables.

Table 9.14. Correlation between the dependent variable and the independent variables (coefficient of Pearson) in the stretches classified as New Outline

Independent Variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,464	0,001
H.AADT	-0,36	0,011
Age	0,218	0,132
R.Age	0,212	0,144
TotVeh	-0,183	0,209
TotH.Veh	-0,169	0,247
PSV	-0,479	< 0,001
Rain 15	-0,15	0,279
TotBit	-0,429	0,002

Figure 9.3 shows the correlation between the dependent and the independent variables graphically. From Table 9.14 it can be observed that the independent variables with the best correlation with *MSSC* are PSV_{req}, *AADT*, *TotBit* and *H.AADT*. All these variables show a great significance in the correlation (p-value < 0,05). Fig. 9.3 shows graphically the correlation between the dependent variable and each of the considered independent quantitative variables and the best curve in each correlation that fits the data. For *AADT* and *H.AADT*, a logarithm function improves the correlations. For these variables, a cubic equation produces a slightly better R^2, but the shape of the curve is not logical according to previous studies, so it has been discarded. It has been considered that this curve fits better for these specific data, but it is not the pattern found in the literature (Szatkowski and Hosking, 1972; NZTA, 2013a). For PSV_{req}, although slight improvements, a linear relationship has been maintained. For *Age*, *R.Age*, *TotVeh* and *TotH.Veh* any kind of function shows similar low values in the coefficient of determination, not increasing significantly the correlation, always lower than 0,1 (R^2 < 0,12); implying that these variable do not influence the available friction. Hence, the seasonal variations (Fig. 5.33 and Fig. 5.34) and the usual skid resistance performance with time (Fig. 5.31) are verified. The rainfall data, *Rain15*, is not influencing the friction too.

The correlation between the dependent variable (*MSSC*) and the qualitative independent variables (*PaveType* and *SurfType*) has also been checked. Firstly, an explanatory analysis of the data according to the qualitative variables, grouping the data according to the considered levels, has been conducted (Table 9.15). Then a normality test for data groups is performed (Table 9.16).

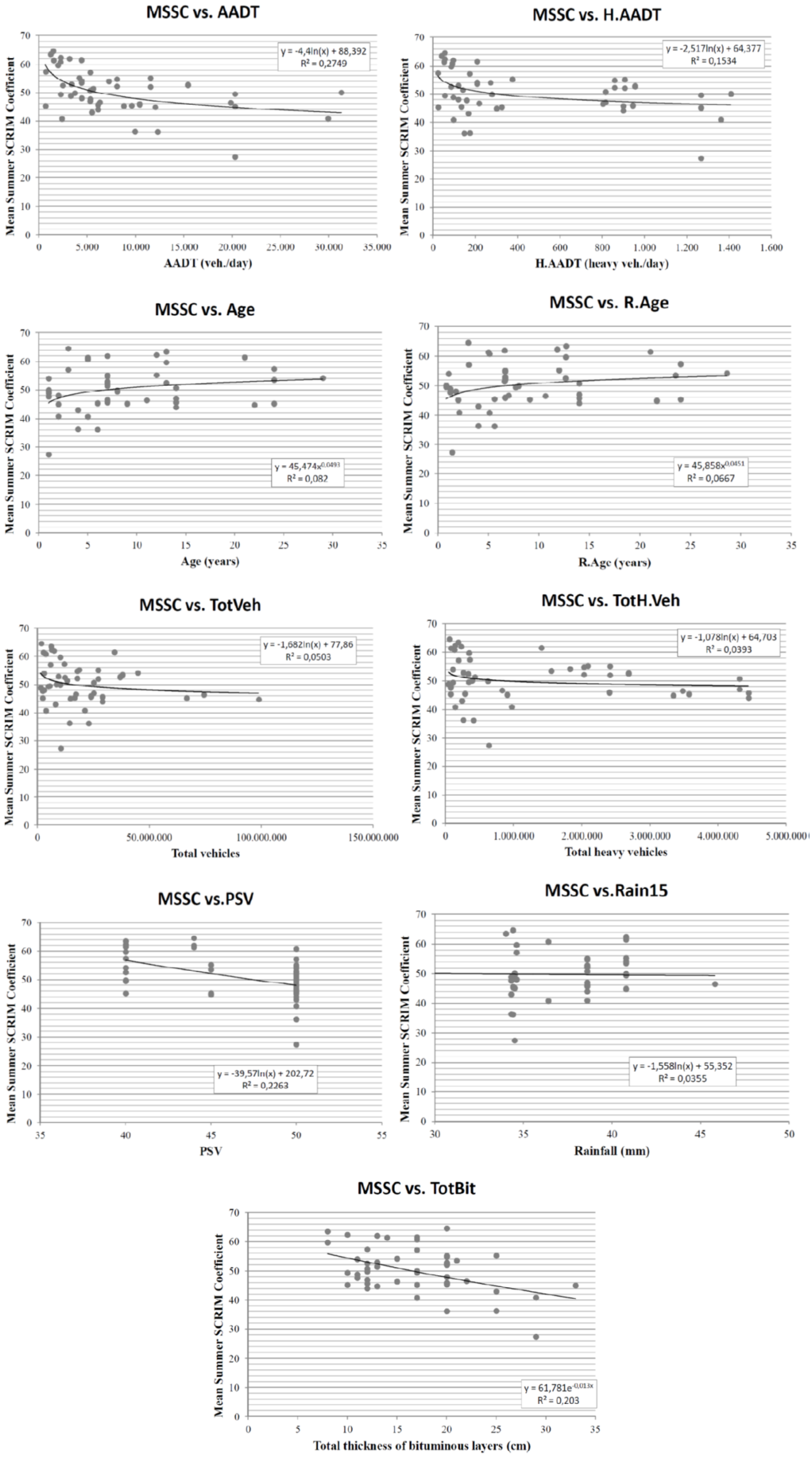


Fig. 9.3. Plot of the dependent variable (MSSC) vs. each of the independent variables and best fitted curve in the stretches classified as New Outline

Table 9.15. Exploratory analysis for the qualitative variables in the stretches classified as New Outline, grouped by levels for the dependent variable (MSSC)

Qualitative variables		**PaveType**		**SurfType**	
Leveles		**Flexible (1)**	**Semi-rigid (2)**	**AC (1)**	**BBTM&PA (2)**
N		19	30	27	20
Mean		49,227	51,124	51,413	49,131
Standard error		2,266	1,068	1,851	0,835
95% CI for mean	**Lower**	44,467	48,940	47,609	47,394
	Upper	53,987	53,307	55,217	50,868
Variance		97,542	34,194	92,460	15,342
Standard deviation		9,876	5,848	9,616	3,917
Minimum		27,333	36,300	27,333	40,833
Maximum		64,625	63,500	64,625	55,102
Range		37,292	27,200	37,292	14,268
Interquartile range		14,389	7,223	15,583	6,674
Skewness		-0,272	0,157	-0,620	-0,172
Kurtosis		-0,309	0,849	-0,090	-0,816

Table 9.16. Normality tests for independent qualitative variables in the stretches classified as New Outline, grouped by levels for the dependent variables (MSSC)

Qualitative independent Variables	**Levels of the variable**	**Kolmogorov-Smirnov[a]**			**Shapiro-Wilk**		
		Statistical	**Degree of freedom**	**Significance**	**Statistical**	**Degree of freedom**	**Significance**
PaveType	Flexible	0,131	19	0,200*	0,953	19	0,452
	Semi-rigid	0,115	30	0,200*	0,957	30	0,261
SurfType	Asphalt Concrete	0,102	27	0,200*	0,948	27	0,191
	BBTM & PA	0,133	22	0,200*	0,958	22	0,441

a Lilliefors Significance correction

* This is lower bound of the true significance

Afterwards, a Levene test is conducted to contrast the hypothesis of that the groups defined by the factor variable (qualitative) come from population with the same variance. The significance associated to the Levene statistic allows checking the of variance homogeneity: if the significance is below 0,05 the hypothesis of homogeneous variance is rejected. This test is complemented with a *t* test to compare mean values of the levels obtained applying a factor (qualitative variable) by means of the difference between their means. A *t* test is applied as there are only two possible groups in each of the qualitative variables (flexible and semi-rigid pavements for *PaveType*, and AC and BBTM & PA for *SurfType*). If more there are more than two groups, an ANOVA (ANalysis Of VAriance) is conducted.

As seen, although the variance is not homogeneous for flexible or semi-rigid pavements, the mean are identical (p-value > 0,05) (Table 9.17). It can be also observed that the difference between them can range from positive to negative values in interval with 95% of confidence in Table 9.17. Therefore, it could be said that this variable does not influence the friction. For surface type, similar conclusions can be obtained: the Levene statistic shows that the variance of the two populations is not homogeneous, but there is not a difference of means with a 95% of significance (Table 9.18).

Table 9.17. Levene statistic for homogeneous variances and t-test for equality of means for levels of PaveType for MSSC in the stretches classified as New Outline.

Levene statistic			**t-test for equality of means**						
F	**Sig.**		**t**	**Degrees of freedom**	**Significance (bilateral)**	**Mean difference**	**Standard error**	**Interval of 95% of confidence for the difference**	
								Lower	**Upper**
8,909	0,004	Equal variances assumed	-0,846	47	0,402	-1,896	2,241	-6,407	2,613
		Equal variances not assumed	-0,757	26,082	0,456	-1,897	2,504	-7,045	3,251

Table 9.18. Levene statistic for homogeneous variances and t-test for equality of means for levels of SurfType for MSSC in the stretches classified as New Outline

Levene statistic			**t-test for equality of means**						
F	**Sig.**		**t**	**Degrees of freedom**	**Significance (bilateral)**	**Mean difference**	**Standard error**	**Interval of 95% of confidence for the difference**	
								Lower	**Upper**
13,721	0,001	Equal variances assumed	1,043	47	0,302	2,282	2,187	-2,119	6,682
		Equal variances not assumed	1,124	35,828	0,268	2,282	2,030	-1,836	6,400

Some variables have been transformed, according to the conclusions of Fig. 9.3. More specifically, natural logarithms have been applied to *AADT* and *H.AADT*. Some multiple linear regression models were tried to develop, only for checking that it has a global significance (the *F* statistic for it has a p-value < 0,05) and that all the introduced variables have individual significance (by means of a t-test to show that with a 95% of significance they were different from 0). The aim was to know which variables are introduced in the model to predict the *MSSC*. The best model developed only included *LnAADT* (the natural logarithm of *AADT*) as the unique variable, with a coefficient of determination, R^2 = 0,275 and an adjusted R^2 = 0,259, implying that only the 25% of the variability of MSSC was possible to be explained by the model.

Then, some General Linear Model (GLM), as defined in section 7.5.5, were tried, including qualitative variables, and hence, it can be observed if the qualitative variables are significant or not. For example, it was tried a GLM with *LnH.AADT*, PSV_{req} and Ln*AADT* as quantitative variables (the first two following the formula of Szatkowski and Hosking (1972), and the last one as it was the unique variable in a multiple linear regression model) and *PaveType* and *SurfType* as qualitative variables. Table 9.19 shows the mean and the standard deviation for the groups produced by the two qualitative variables. The model has the structure of the Eq. 9.3.

Table 9.19. Mean and standard deviation for the levels of the two qualitative variables in the stretches classified as New Outline for the dependent variables MSSC

SurfType	PaveType	Mean	Standard deviation	N
Asphalt concrete (AC) (1)	Flexible (1)	49,955	10,807	15
	Semi-rigid (2)	53,235	7,961	12
	Total	51,413	9,616	27
Discontinuous and Porous Asphalt (BBTM&PA) (2)	Flexible (1)	46,497	5,261	4
	Semi-rigid (2)	49,716	3,475	18
	Total	49,131	3,917	22
Total	Flexible (1)	49,227	9,876	19
	Semi-rigid (2)	51,124	5,848	30
	Total	50,388	7,623	49

$$MSSC = Intercept + LnAADT + LnH.AADT + PSV_{req} + SurfType +$$

$$+ PaveType + SurfType * PaveType \qquad [9.3]$$

The analysis of the effect inter-subject and the estimation of the coefficients for this model are shown in Table 9.20 and table 9.21, respectively. The model has a R^2 = 0,378 and an adjusted R^2 = 0,289. The complete analysis is not shown as this is only a provisional model, employed to detect significant and non significant variables.

As observed, Ln*H.AADT* has low significance, based on the *F* statistic and the *t* test for its individual significance. *PaveType* also shows a low significance in both statistics. Therefore, these variables must be discarded from the model. A model with Ln*AADT*, PSV_{req} and *SurfType* as the only variables (Eq. 9.4) produces the results displayed in Table 9.22 and Table 9.23.

$$MSSC = Intercept + LnAADT + PSV_{req} + SurfType \qquad [9.4]$$

Table 9.20. Test of Between-Subjects effects for model of Eq. 9.3 for the stretches classified as New Outline

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	1054,51	6	175,75	4,255	0,002	0,378	25,531	0,961
Intercept	2432,38	1	2432,38	58,89	< 0,001	0,584	58,89	1
LnAADT	87,23	1	87,23	2,112	0,154	0,048	2,112	0,295
PSVreq	126,97	1	126,97	3,074	0,087	0,068	3,074	0,403
LnH.AADT	0,59	1	0,59	0,014	0,905	< 0,001	0,014	0,052
SurfType	56,62	1	56,62	1,371	0,248	0,032	1,371	0,208
PaveType	41,59	1	41,59	1,007	0,321	0,023	1,007	0,165
SurfType * PaveType	9,87	1	9,87	0,239	0,628	0,006	0,239	0,077
Error	1734,74	42	41,30					
Total	127199,61	49						
Corrected total	2789,25	48						

Table 9.21. Parameter estimates for model of Eq. 9.3 for the stretches classified as New Outline

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	Lower			
Intercept	115,871	15,352	7,548	< 0,001	84,889	146,853	0,576	7,548	1,000
LnAADT	-3,602	2,479	-1,453	0,154	-8,605	1,400	0,048	1,453	0,295
PSVreq	-0,631	0,360	-1,753	0,087	-1,358	0,095	0,068	1,753	0,403
LnH.AADT	-0,246	2,057	-0,120	0,905	-4,398	3,906	< 0,000	0,120	0,052
[SurfType=1]	-4,766	3,839	-1,242	0,221	-12,513	2,981	0,035	1,242	0,228
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.
[PaveType=1]	-3,412	3,751	-0,910	0,368	-10,983	4,158	0,019	0,910	0,144
[PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=1] * [PaveType=1]	2,380	4,869	0,489	0,628	-7,447	12,206	0,006	0,489	0,077
[SurfType=1] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

Table 9.22. Test of Between-Subjects effects for model of Eq. 9.4 for the stretches classified as New Outline

Origin	Type III Sum o Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	1011,033	3	337,011	8,528	< 0,001	0,362	25,585	0,990
Intercept	2814,599	1	2814,599	71,227	< 0,001	0,613	71,227	1,000
LnAADT	308,031	1	308,031	7,795	0,008	0,148	7,795	0,780
PSVreq	164,534	1	164,534	4,164	0,047	0,085	4,164	0,515
SurfType	159,316	1	159,316	4,032	0,051	0,082	4,032	0,502
Error	1778,217	45	39,516					
Total	127199,61	49						
Corrected total	2789,249	48						

The model has a R^2 of 0,362 and an adjusted R^2 of 0,320. It has a worse R^2 but a better adjusted R^2 and all the introduced variables are significant. Other possibilities mixing different variables from those with better correlation may show better R^2 values but some of the variables are not significant. *PaveType* has verified not to be an influencing variable; as the means in the different groups are equal and it has low significance in General Linear Models. On the contrary, *SurfType*, which has also produce groups with the same mean, has shown a high significance, especially when not included with *PaveType* and *LnH.AADT*.

Table 9.23. Parameter estimates for model of Eq. 9.4 for the stretches classified as New Outline

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	Upper			
Intercept	116,527	14,27	8,166	< 0,000	87,785	145,269	0,597	8,166	1,000
LnAADT	-3,897	1,396	-2,792	0,008	-6,708	-1,086	0,148	2,792	0,780
PSV	-0,635	0,311	-2,041	0,047	-1,261	-0,008	0,085	2,041	0,515
[SurfType=1]	-4,61	2,296	-2,008	0,051	-9,234	0,014	0,082	2,008	0,502
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

9.3.1.1.1. Analysis of data for New Outline stretches with an real age over two years

Kokkalis (1998) showed that the equilibrium phase or stationary period of pavement friction is achieved after 2 years. Navarro *et al.* (2011) also exposed that, when analyzing new pavement surfaces, the phases described in Fig. 5.38 take places in Gipuzkoa, with a similar climate to Biscay, and long for about 2 years. Therefore, in order to discard sections which had not achieved that stationary phase and could bias the previous analysis; sections with a real age lower than 2 years were discarded from the modelling. A total of 42 observations with *R.Age* ≥ 2 years were analyzed. Followed analysis structure is similar to the previous subsection. Table 9.24 shows the explanatory analysis of the quantitative variables for the 42 observations. Once again, only the dependent variable (*MSSC*) follow a normal distribution but no one of the independent variables do (Table 9.25).

Table 9.24. Exploratory analysis for the quantitative variables in the stretches classified as New Outline with a real age not lower than 2 years.

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Mean		51,04	7283,74	431,81	10,19	10,01	21980,1	1464,2	46,71	31,26	16,93
Standard error		1,13	943,98	62,68	1,03	1,03	3088,6	221,96	0,66	2,12	0,87
95% CI for mean	**Lower**	48,75	5377,33	305,23	8,11	7,93	15742,5	1016,0	45,38	26,98	15,18
	Upper	53,33	9190,15	558,39	12,27	12,09	28217,6	1912,5	48,05	35,54	18,68
Variance		53,91	37426356	164993	44,7	44.41	400656120	2069107	18,4	188,3	31,6
Std. deviation		7,34	6117,71	406,19	6,69	6,66	20016,4	1438	4,29	13,72	5,62
Minimum		36,2	669	23	2	2	1760	60,11	40	2,1	8
Maximum		64,62	29916	1361	29	28,6	98892	4447,04	50	45,8	33
Range		28,43	29247	1338	27	26,6	97132	4386,93	10	43,7	25
Interquartile range		10,33	7978	775	7	7,4	19780	2175	7	4,4	8
Skewness		0,04	1,69	0,71	1,24	1,25	2,07	0,80	-0,72	-1,43	0,64
Kurtosis		-0,66	3,60	-1,02	0,86	0,81	5,23	-0,63	-1,28	0,35	0,36

With regard to the correlations between the dependent variable and the quantitative variables (Table 9.26), they show higher coefficient of Pearson, R, than the correlations of Table 9.14.

Table 9.25. Normality tests for dependent and independent quantitative variables in the stretches classified as New Outline with a real age not lower than 2 years

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
MSSC	0,116	42	0,175	0,969	42	0,296
AADT	0,182	42	0,001	0,848	42	< 0,001
H.AADT	0,248	42	< 0,001	0,814	42	< 0,001
Age	0,231	42	< 0,001	0,853	42	< 0,001
R.Age	0,222	42	< 0,001	0,849	42	< 0,001
TotVeh	0,171	42	0,003	0,799	42	< 0,001
TotH.Veh	0,215	42	< 0,001	0,844	42	< 0,001
PSV	0,373	42	< 0,001	0,694	42	< 0,001
Rain 15	0,365	42	< 0,001	0,674	42	< 0,001
TotBit	0,139	42	0,042	0,939	42	0,027

[a] Lilliefors Significance correction

Table 9.26. Correlation between the dependent variable and the independent variables (coefficient of Pearson) in the stretches classified as New Outline with a real age not lower than 2 years

Independent Variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,493	0,001
H.AADT	-0,322	0,038
Age	0,152	0,336
R.Age	0,149	0,345
TotVeh	-0,274	0,079
TotH.Veh	-0,257	0,100
PSV	-0,491	0,001
Rain 15	-0,138	0,385
TotBit	-0,362	0,018

Observing the curves that best fit the data in the plot variable dependent (*MSSC)* vs. variables independent individually, the selected equations for each variable is exposed in Table 9.27. The best curve has not been always selected, as usually, quadratic and cubic curves fit better but they do not reproduce the pattern described in the literature. Other times, if the improvement between the linear correlation and other ones in the coefficient of determination is very low ($\Delta R^2 < 0{,}05$) a linear model has been maintained, implying that the independent variable has not been transformed.

Explanatory analyses of the qualitative independent variables were carried out (Table 9.28).

Table 9.27. Equations that best correlate each independent variable individually with the dependent variable for stretches classified as New Outline with real age not lower than 2 years.

Independent Variable	Equation type	Resume of the model					Parameter estimates			
		R^2	F	Degrees of freedom 1	Degree of freedom 2	Sig.	Intercept	b1	b2	b3
AADT	Logarithm	0,284	15,828	1	40	< 0,001	88,501	-4,382	-	-
H.AADT	Logarithm	0,141	6,591	1	40	0,014	63,84	-2,33	-	-
Age	Linear	0,023	0,950	1	40	0,336	49,337	0,167	-	-
R.Age	Linear	0,022	0,914	1	40	0,345	49,392	0,165	-	-
TotVeh	Logarithm	0,126	5,776	1	40	0,021	77,916	-2,794	-	-
TotH.Veh	Inverse	0,105	4,716	1	40	0,036	49,169	609,596	-	-
PSV	Linear	0,241	12,706	1	40	0,001	90,295	-0,84	-	-
Rain15	Linear	0,019	0,771	1	40	0,385	53,342	-0,074	-	-
TotBit	Linear	0,131	6,044	1	40	0,018	59,048	-0,473	-	-

Table 9.28. Exploratory analysis for the qualitative variables in the stretches classified as New Outline, grouped by levels for the dependent variables (MSSC), with real age not lower than 2 years.

		PaveType		SurfType	
		Flexible (1)	Semi-rigid (1)	AC (1)	BBTM&PA (2)
N		17	25	25	17
Mean		50,588	51,349	52,513	48,876
Standard error		2,138	1,266	1,723	1,040
95% CI for mean	**Lowe**	46,055	48,737	48,956	46,671
	Upper	55,120	53,961	56,070	51,081
Variance		77,712	40,038	74,254	18,389
Standard deviation		8,815	6,328	8,617	4,288
Minimum		36,200	36,300	36,200	40,833
Maximum		64,625	63,500	64,625	55,102
Range		28,425	27,200	28,425	14,268
Interquartile range		15,078	8,259	15,852	6,971
Skewness		0,120	0,042	-0,367	-0,076
Kurtosis		-1,391	0,412	-0,950	-1,200

Both levels of the variable *PaveType* and both levels of *SurfType* follow a normal distribution (Table 9.29). Levels according to each factor, *PaveType* and *SurfType* (qualitative independent variables) have non homogeneous variances, but they have equal means (Table 9.30 and 9.31, respectively). However, when the type of pavement (flexible or semi-rigid) maintained a similar significance for equality of means, in the case of surface type, distinguishing between asphalt concrete mixes and discontinuous and porous asphalts have show a trend to reduce the significance of the equality of means when the "*young*" section data were eliminated, i.e. a greater difference between these two groups, indicating that it may be a influencing factor.

Table 9.29. Normality tests for independent qualitative variables in the stretches classified as New Outline, grouped by levels for the dependent variables (MSSC), with a real age not lower than 2 years

Qualitative Variables	Levels of the factor	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
		Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
PaveType	Flexible (1)	0,204	17	0,059	0,923	17	0,168
	Semi-rigid (2)	0,123	25	0,200*	0,958	25	0,384
SurfType	AC (1)	0,118	25	0,200*	0,94	25	0,150
	BBTM & PA (2)	0,203	17	0,061	0,92	17	0,147

[a] Lilliefors Significance correction

* This is a lower bound of the true significance

Table 9.30. Levene statistic for homogeneous variances and t-test for equality of means for levels of PaveType for MSSC in the stretches classified as New Outline, with a real age not lower than 2 years.

Levene statistic		t-test for equality of means							
F	Sig.	Mean difference	t	Degrees of freedom	Significance (bilateral)	Mean difference	Standard error	Interval of 95% of confidence for the difference	
								Lower	Upper
6,749	0,013	Equal variances assumed	-0,326	40	0,746	-0,761	2,334	-5,478	3,955
		Equal variances not assumed	-0,306	26,968	0,762	-0,761	2,485	-5,860	4,337

Table 9.31. Levene statistic for homogeneous variances and t-test for equality of means for levels of SurfType for MSSC in the stretches classified as New Outline with a real age not lower than 2 years

Levene statistic		t-test for equality of means							
F	Sig.	Mean difference	t	Degrees of freedom	Significance (bilateral)	Mean difference	Standard error	Interval of 95% of confidence for the difference	
								Lower	Upper
8,720	0,005	Equal variances assumed	1,606	40	0,116	3,637	2,265	-0,941	8,215
		Equal variances not assumed	1,807	37,25	0,079	3,637	2,013	-0,441	7,715

A new attempt to find a multiple linear regression model, by the functions Step by Step and Forward, shows that, once again the unique independent variable in the model is Ln*AADT*, the one with the higher correlation, not being possible to introduce more variables if the significance of the coefficient is required. The statistics of the model are shown in Table 9.32.

Again, General Linear Models were tried including the two qualitative variables (*PaveType* and *SurfType*) and the independent quantitative variables that best correlate with MSSC (dependent variables), according to the coefficient of Pearson and the multiple linear regression analysis. A first attempt tried to develop a model as the one indicated in Eq. 9.3. Obtained values are exposed in Table 9.33 and 9.34. A value of R^2 of 0,354 was obtained and an adjusted R^2 of 0,243.

Table 9.32138. Analysis of variance of the multiple linear regression model proposed for the stretches classified as New Outline with a real age not lower than 2 years.

Analysis of variance						
Source	**Degrees of freedom**	**Sum of squares**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**
Model	1	626,632	626,632	15,828	< 0,001	1,641
Error	40	1583,556	39,589			
Corrected total	41	2210,188				
Root mean square error	**Coefficient of variation**	**R**	**R^2**	**Adj. R^2**	**Colinearity statistics**	
					Tolerance	**VIF**
6,292	15,828	0,532	0,284	0,266	1,000	1,000
Parameter estimates						
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**	
Intercept	88,501	9,465	9,350	< 0,001	69,370	107,631
Ln*AADT*	-4,382	1,101	-3,979	< 0,001	-6,608	-2,156

Table 9.33. Test of Between-Subjects effects for model of Eq. 9.3 for the stretches classified as New Outline with a real age not lower than 2 years

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	781,381	6	130,23	3,19	0,013	0,354	19,141	0,871
Intercept	2037,321	1	2037,321	49,906	< 0,001	0,588	49,906	1
LnAADT	104,558	1	104,558	2,561	0,119	0,068	2,561	0,344
PSVreq	79,283	1	79,283	1,942	0,172	0,053	1,942	0,273
LnH.AADT	7,805	1	7,805	0,191	0,665	0,005	0,191	0,071
SurfType	2,115	1	2,115	0,052	0,821	0,001	0,052	0,056
PaveType	12,375	1	12,375	0,303	0,585	0,009	0,303	0,083
SurfType * PaveType	5,688	1	5,688	0,139	0,711	0,004	0,139	0,065
Error	1428,806	35	40,823					
Total	111628,3	42						
Corrected total	2210,188	41						

The variables *PaveType*, *SurfType* and Ln*H.AADT* show low significance. After several trials, combining these variables in a General Linear Model, the best R^2 obtained is not over 0,35, and not all the variables are statistically significant. As example, it can be presented the model as presented in Eq. 9.4 (Table 9.35 and Table 9.36).

Table 9.34. Parameter estimates for model of Eq. 9.3 for the stretches classified as New Outline with a real age not lower than 2 years

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	Upper			
Intercept	108,63	16,116	6,741	< 0,001	75,914	141,347	0,565	6,741	1,000
LnAADT	-4,489	2,805	-1,6	0,119	-10,184	1,205	0,068	1,6	0,344
PSVreq	-0,52	0,373	-1,394	0,172	-1,278	0,238	0,053	1,394	0,273
LnH.AADT	1,19	2,722	0,437	0,665	-4,336	6,716	0,005	0,437	0,071
[SurfType=1]	-1,877	5,024	-0,374	0,711	-12,076	8,322	0,004	0,374	0,065
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.
[PaveType=1]	-2,3	4,175	-0,551	0,585	-10,777	6,176	0,009	0,551	0,084
[PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=1] * [PaveType=1]	1,959	5,249	0,373	0,711	-8,697	12,615	0,004	0,373	0,065
[SurfType=1] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.

a Set to zero because this parameter is redundant.

Table 9.35. Test of Between-Subjects effects for model of Eq. 9.4 for the stretches classified as New Outline with a real age not lower than 2 years

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	743,708	3	247,903	6,424	0,001	0,336	19,271	0,952
Intercept	2115,631	1	2115,631	54,821	< 0,001	0,591	54,821	1,000
LnAADT	205,648	1	205,648	5,329	0,027	0,123	5,329	0,614
PSVreq	96,711	1	96,711	2,506	0,122	0,062	2,506	0,339
SurfType	51,129	1	51,129	1,325	0,257	0,034	1,325	0,202
Error	1466,479	38	38,592					
Total	111628,3	42						
Corrected total	2210,188	41						

Table 9.36. Parameter estimates for model of Eq. 9.4 for the stretches classified as New Outline with a real age not lower than 2 years

Parámetro	B	Std. Error	t	Sig.	95% confidence interval		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	Upper			
Intercept	108,271	15,286	7,083	< 0,001	77,325	139,216	0,569	7,083	1,000
LnAADT	-3,682	1,595	-2,308	0,027	-6,91	-0,453	0,123	2,308	0,614
PSVreq	-0,514	0,325	-1,583	0,122	-1,171	0,143	0,062	1,583	0,339
[SurfType=1]	-2,949	2,562	-1,151	0,257	-8,137	2,238	0,034	1,151	0,202
[SurfType=2]	0[a]	.	.	.	.	.	.	.	.

a Set to zero because this parameter is redundant.

After analysis the different combination, it can conclude that Ln*AADT* is almost always a significant variables, PSV_{req} is near the 90% of confidence to be a true variable (different from zero) and *SurfType* could be introduced despite its low significance (p=0,257).

9.3.1.2. Analysis of data for Maintenance and Rehabilitation (M&R) sections

For analysis of M&R section there are 63 stretches. Three of them have a real age lower than 2 years and hence, it is assumed that they are not in the stationary phase of the polishing performance. Consequently, they are discarded from the analysis, following the criteria of Kokkalis (1998) and the results from Navarro et al. (2011), ideas adopted in the previous analysis of stretches considered as New Outline. A similar explanatory analysis was conducted with the 60 observations with a real age not below two years (Table 9.37). The normality test (Table 9.38) indicates that only *MSSC* follows a normal distribution. Table 9.39 shows the correlation between the dependent variable and the independent variables.

Table 9.37. Exploratory analysis for the quantitative variables in stretches classified as Maintenance & Rehabilitation

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Mean		54,80	6978,65	277,28	5,83	5,85	13413,0	589,0	47,70	21,36	19,52
Standard error		1,08	637,04	26,73	0,37	0,37	1659,5	84,6	0,49	2,00	0,71
95% CI for mean	**Lower**	52,64	5703,9	223,80	5,09	5,10	10092,48	419,63	46,72	17,36	18,11
	Upper	56,96	8253,4	330,76	6,58	6,59	16733,59	758,35	48,68	25,35	20,93
Variance		69,71	24349274	42857	8,24	8,30	165227148	429814	14,25	239,15	29,81
Std. deviation		8,35	4934,50	207,02	2,87	2,88	12854	656	3,78	15,46	5,46
Minimum		34,27	665	23	2	2	1818	14	40	2,1	12
Maximum		73,32	19050	956	12	12,1	70547	3647	50	45,8	37
Range		39,05	18385	933	10	10,1	68729	3633	10	43,7	25
Interquartile range		12,858	5281,0	273,0	5	4,97	13258	633,97	5	29,7	10
Skewness		-0,127	0,620	1,305	0,44	0,443	2,181	2,640	-1,261	-0,036	0,318
Kurtosis		-0,204	-0,193	2,301	-1,05	-1,06	6,506	8,845	-0,044	-1,717	0,252

Table 9.38. Normality tests for dependent and independent quantitative variables in stretches classified as M&R

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
MSSC	0,078	60	0,200*	0,984	60	0,61
AADT	0,24	60	< 0,001	0,84	60	< 0,001
H.AADT	0,126	60	0,019	0,93	60	0,002
Age	0,127	60	0,018	0,889	60	< 0,001
R.Age	0,429	60	< 0,001	0,622	60	< 0,001
TotVeh	0,238	60	< 0,001	0,897	60	< 0,001
TotH.Veh	0,195	60	< 0,001	0,906	60	< 0,001
PSV	0,117	60	0,04	0,918	60	0,001
Rain 15	0,184	60	< 0,001	0,79	60	< 0,001
TotBit	0,202	60	< 0,001	0,722	60	< 0,001

[a] Lilliefors Significance correction

* This is a lower bound of the true significance

Table 9.39. Correlation between the dependent variable and the independent variables (coefficient of Pearson) in stretches classified as Maintenance & Rehabilitation

Independent variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,443	<0,001
H.AADT	-0,436	<0,001
Age	0,023	0,862
R.Age	0,021	0,874
TotVeh	-0,436	< 0,001
TotH.Veh	-0,426	0,001
PSV	-0,500	<0,001
Rain 15	0,109	0,407
TotBit	-0,059	0,654

As in previous analysis, *AADT* and *H.AADT* have a good correlation with *MSSC*, and *Rain15*, *Age* and *R.Age* do not show significant correlation. On the contrary, *TotVeh* and *TotH.Veh* show good correlations with the dependent variable. As done in the previous analysis, the curves that best fit the MSSC vs. each of the independent variables are calculated (Table 9.40), and taking those equations into consideration, some transformations of cariables are suggested. For *AADT*, *H.AADT*, *TotVeh* and *TotH.Veh* a logarithmic transformation is carried out. In variables PSV_{req}, *Age*, *R.Age* changing from a linear correlation to another one does not imply an improvement over 0,05 in R^2. For *Rain15*, a cuadratic transformation is conducted.

Table 9.40.. Equations that best correlate each independent variable individually with the dependent variable for stretches classified as Maintenance & Rehabilitation

Independent Variable	Equation type	Resume of the model					Parameter estimates			
		R^2	F	Degrees of freedom 1	Degree of freedom 2	Sig.	Intercept	b1	b2	b3
AADT	Logarithm	0,318	27,067	1	58	<0,001	93,268	-4,542	-	-
H.AADT	Logarithm	0,273	21,824	1	58	<0,001	79,914	-4,745	-	-
Age	Linear	0,001	0,03	1	58	0,862	54,411	0,067	-	-
R.Age	Linear	0	0,025	1	58	0,874	54,445	0,061	-	-
TotVeh	Logarithm	0,327	28,243	1	58	<0,001	131,715	-4,813	-	-
TotH.Veh	Logarithm	0,291	23,857	1	58	<0,001	108,421	-4,197	-	-
PSV	Linear	0,25	19,319	1	58	<0,001	107,541	-1,106	-	-
Rain15	Cuadratic	0,107	3,404	2	57	0,040	49,549	0,863	0,019	-
TotBit	Lineal	0,003	0,203	1	58	0,654	56,561	-0,09	-	-

Similar analysis as in New Outline stretches were conducted for rehabilitated and maintained sections with qualitative variables (Table 9.41). In this case, there are stretches with Slurry, introducing it as a surface layer (level) in factor *SurfType*. Distribution in each group produced by the levels follow a normal distribution (Table 9.42)

Table 9.41. Exploratory analysis for the qualitative variables in stretches classified as Maintenance & Rehabilitation, grouped by levels, for the dependent variables (MSSC).

		PaveType		SurfType		
		Flexible (1)	**Semi-rigid (2)**	**AC (1)**	**BBTM& PA (2)**	**Slurry (3)**
N		41	19	17	8	35
Mean		56,057	52,086	56,547	47,094	55,712
Standard error		1,363	1,587	2,834	1,155	1,044
95% CI for mean	**Lower**	53,302	48,751	50,539	44,363	53,590
	Upper	58,811	55,421	62,555	49,825	57,834
Variance		76,163	47,879	136,541	10,670	38,165
Standard deviation		8,727	6,919	11,685	3,267	6,178
Minimum		36,000	34,273	34,273	43,576	43,600
Maximum		73,318	60,674	73,318	53,400	65,980
Range		37,318	26,401	39,046	9,824	22,380
Interquartile range		14,325	9,800	16,353	4,852	8,567
Skewness		-0,131	-0,953	-0,489	0,913	-0,410
Kurtosis		-0,595	0,855	-0,491	0,827	-0,699

Table 9.42. Normality tests for independent qualitative variables in stretches classified as Maintenance & Rehabilitation, grouped by levels for the dependent variables (MSSC,)

Qualitative Variables	Levels of the factor	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
		Statistical	**Degree of freedom**	**Significance (p-value)**	**Statistical**	**Degree of freedom**	**Significance (p-value)**
PaveType	Flexible (1)	0,108	41	0,200*	0,975	41	0,505
	Semi-rigid (2)	0,171	19	0,147	0,916	19	0,096
SurfType	AC (1)	0,139	17	0,200*	0,95	17	0,459
	BBTM & PA (2)	0,164	8	0,200*	0,916	8	0,401
	Slurry (3)	0,124	35	0,194	0,956	35	0,168

[a] Lilliefors Significance correction

* This is a lower bound of the true significance

The Levene statistic show that the levels from *PaveType* have homogeneous variance and that there is not significantly difference between means of those levels (Table 9.43).

Table 9.43. Levene statistic for homogeneous variances and t-test for equality of means for levels of PaveType for MSSC in stretches classified as Maintenance & Rehabilitation

Levene statistic		t-test for equality of means							
F	**Sig.**	**Mean difference**	**t**	**Degrees of freedom**	**Significance (bilateral)**	**Mean difference**	**Standard error**	**Interval of 95% of confidence for meanSCR difference**	
								Lower	**Upper**
2,724	0,104	Equal variances assumed	1,743	58	0,087	3,970	2,278	-0,590	8,531
		Equal variances not assumed	1,898	43,647	0,064	3,970	2,092	-0,247	8,188

For *SurfType*, as there are three levels (AC, BBTM&PA, and Slurry) an Analysis of Variance (ANOVA) was conducted. The Levene statistic (Table 9.44) indicate that the null hypothesis is rejected ($p < 0{,}05$), and hence, levels do not have homogeneous variances. The robust tests for equality of means show that there is not equality of means between all the levels in SurfType (Table 9.45) obtaining a similar conclusion from the ANOVA test (Table 9.46). Moreover, the multiple comparison between levels by means of some statistics (Tamhane's T2 and Dunnett's T3 for not homogeneous variances) indicate that there is mean difference between BBTM&PA and the other two levels (AC and Slurry), but not between AC and Slurry (Table 9.47).

Table 9.44. Test of homogeneity of variances for levels in SurfType in stretches classified as Maintenance & Rehabilitation, for the dependent variable (MSSC)

Levene Statistic	Degrees of Freedom 1	Degrees of Freedom 2	Significance
8,833	2	57	< 0,001

Table 9.45. Robust test of equality of means for levels in SurfType in stretches classified as Maintenance & Rehabilitation, for the dependent variable (MSSC).

Test	Statistic	Degrees of freedom 1	Degrees of freedom 2	Significance
Welch	16,160	2	24,371	< 0,001
Brown-Forsythe	4,520	2	24,477	0,021

Table 9.46. ANOVA for levels in SurfType levels in stretches classified as Maintenance & Rehabilitation for the dependent variable (MSSC).

Variable (SurfType)	Sum of Squares	Degrees of freedom	Mean Square	F	Significance
Between groups	556,014	2	278,007	4,455	0,016
Within groups	3556,976	57	62,403		
Total	4112,989	59			

For model developing, initially a multiple linear regression was conducted to observe the quantitative variables that best correlates with the dependent variables. Independent variables were transformed according to previous results (Table 9.40) and conducting a Forward and Step by Step analysis with IBM SPSS v.24, the only variable included was Ln*AADT* (Table 9.48).

Then, some general linear models were tried, including qualitative variables. Once again, a model with the variables indicated in Eq. 9.3 was analyzed (Table 9.49 and Table 9.50).

Table 9.47. Multiple comparison for levels SurfType levels in stretches classified as Maintenance & Rehabilitation for the dependent variable (MSSC)

Statistic	SurfType (I)	SurfType (J)	Mean Differences (I-J)	Std Error	Sig	95% confidence interval	
						Lower	Upper
Tamhane's T2	AC (1)	BBTM&PA (2)	9,453	3,060	0,017	1,498	17,408
		Slurry (3)	0,835	3,020	0,990	-7,016	8,686
	BBTM&PA (2)	AC (1)	-9,453	3,060	0,017	-17,408	-1,498
		Slurry (3)	-8,618	1,557	0,000	-12,667	-4,568
	Slurry (3)	AC (1)	-0,835	3,020	0,990	-8,686	7,016
		BBTM&PA (2)	8,618	1,557	0,000	4,568	12,667
Dunnett's T3	AC (1)	BBTM&PA (2)	9,453	3,060	0,017	1,529	17,376
		Slurry (3)	0,835	3,020	0,989	-6,985	8,655
	BBTM&PA (2)	AC (1)	-9,453	3,060	0,017	-17,376	-1,529
		Slurry (3)	-8,618	1,557	0,000	-12,651	-4,584
	Slurry (3)	AC (1)	-0,835	3,020	0,989	-8,655	6,985
		BBTM&PA (2)	8,618	1,557	0,000	4,584	12,651

Table 9.48. Analysis of variance of the multiple linear regression model proposed for stretches classified as Maintenance & Rehabilitation

Analysis of variance						
Source	**Degrees of freedom**	**Sum of squares**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**
Model	1	1308,686	1308,686	27,067	< 0,001	1,037
Error	58	2804,303	48,35			
Corrected total	59	4112,989				
Root mean square error	**Coefficient of variation**	**R**	**R^2**	**Adj. R^2**	**Colinearity statistics**	
					Tolerance	**VIF**
6,953		0,564	0,318	0,306	1,000	1,000
Parameter estimates						
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**	
Intercept	93,268	7,448	12,522	< 0,001	78,358	108,178
LnAADT	-4,542	0,873	-5,203	< 0,001	-6,290	-2,795

As seen in Table 9.49, Ln*H.AADT* is the variables with lowest significance. Removed from the model, PSV_{req} have the lowest significance (p-value = 0,204). Removed from the analysis, a new General Linear Model including Ln*AADT*, *SurfType* and *PaveType* as factors was analyzed. *PaveType* and the combination of factors *PaveType***SurfType* were variables not statistically significant (p-value = 0,070 and p-value = 0,160, respectively). After eliminating *PaveType* from the analysis, a model with the variables of Eq. 9.5 was tested. It has all the variables statistically significant (Table 9.51 and 9.52. The model has a R^2 of 0,503 and an adjusted R^2 of 0,476.

$$MSSC = Intercept + LnAADT + SurfType \qquad [9.5]$$

Table 9.49. Test of Between-Subjects effects for model of Eq. 9.3 for stretches classified as Maintenance & Rehabilitation

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	2267,057	7	323,865	9,123	< 0,001	0,551	63,863	1,000
Intercept	2063,186	1	2063,186	58,12	< 0,001	0,528	58,12	1,00
LnAADT	91,143	1	91,143	2,567	0,115	0,047	2,567	0,35
LnH.AADT	12,683	1	12,683	0,357	0,553	0,007	0,357	0,09
PSVreq	63,133	1	63,133	1,778	0,188	0,033	1,778	0,258
SurfType	939,544	2	469,772	13,234	< 0,001	0,337	26,467	0,996
PaveType	171,325	1	171,325	4,826	0,033	0,085	4,826	0,578
SurfType * PaveType	58,89	1	58,89	1,659	0,203	0,031	1,659	0,244
Error	1845,932	52	35,499					
Total	184291,279	60						
Corrected total	4112,989	59						

Table 9.50. Parameter estimates for model of Eq. 9.3 for stretches classified as Maintenance & Rehabilitation

Parameters	B	Std. Error	t	Sig.	95% CI Lower	95% CI Upper	Partial eta-squared	Non centrality parameter	Observed Power
Intercept	121,270	15,42	7,861	<0,001	90,314	152,226	0,543	7,861	1,000
LnAADT	-4,281	2,672	-1,602	0,115	-9,643	1,080	0,047	1,602	0,350
PSVreq	1,338	2,238	,598	0,553	-3,153	5,828	0,007	0,598	0,090
LnH.AADT	-0,746	0,559	-1,334	0,188	-1,867	0,376	0,033	1,334	0,258
[SurfType=1]	-13,403	3,815	-3,513	0,001	-21,058	-5,747	0,192	3,513	0,932
[SurfType=2]	-8,963	2,626	-3,413	0,001	-14,233	-3,693	0,183	3,413	0,918
[SurfType=3]	0[a]	.	.	.	.	.	.	.	.
[PaveType=1]	2,816	2,445	1,152	0,255	-2,090	7,721	0,025	1,152	0,204
[PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=1] * [PaveType=1]	5,732	4,450	1,288	0,203	-3,198	14,662	0,031	1,288	0,244
[SurfType=1] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.
[SurfType=2] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=3] * [PaveType=1]	0[a]	.	.	.	.	.	.	.	.
[SurfType=3] * [PaveType=2]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

Table 9.51139. Test of Between-Subjects effects for model of Eq. 9.5 for the stretches classified as Maintenance & Rehabilitation

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	2069,161	3	689,72	18,898	<0,001	0,503	56,694	1,000
Intercept	5807,797	1	5807,797	159,131	<0,001	0,74	159,131	1,000
LnAADT	1513,148	1	1513,148	41,46	<0,001	0,425	41,46	1,000
SurfType	760,475	2	380,237	10,418	<0,001	0,271	20,837	0,984
Error	2043,828	56	36,497					
Total	184291,279	60						
Corrected total	4112,989	59						

Table 9.52. Parameter estimates for model of Eq. 9.5 for stretches classified as Maintenance & Rehabilitation

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	Upper			
Intercept	113,185	8,984	12,598	<0,001	95,187	131,182	0,739	12,598	1
LnAADT	-6,488	1,008	-6,439	<0,001	-8,507	-4,47	0,425	6,439	1
[SurfType=1]	-8,644	2,315	-3,735	<0,001	-13,281	-4,008	0,199	3,735	0,956
[SurfType=2]	-7,409	2,375	-3,12	0,003	-12,166	-2,651	0,148	3,12	0,866
[SurfType=3]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

9.3.1.3. Conclusions from analysis of completely known stretches grouped according to New Outline or Maintenance and Rehabilitation

After developing some models for completely known pavement sections grouped according to New Outline or Rehabilitation and Maintenance, it can be observed that the main variables affecting the skid resistance, by means of MSSC, are the Annual Average Daily Traffic (*AADT*) (especially when calculating its natural logarithm), the Annual Average Daily Heavy Traffic (*H.AADT*) and the required PSV of the aggregates (PSV_{req}). The last two ones were identified in the literature as the main ones. It has been demonstrated that *AADT* become the main variable and the other ones sometimes are not significant when added to it. The other quantitative variables, *Age*, Real Age (*R.Age*), Total thickness of bituminous layers (*TotBit*), and rainfall data of 15 days before the data collection (*Rain15*) always showed low correlations and were not included in the models. The total vehicles that have crossed the section (*TotVeh*) and the total heavy vehicles that crossed the section (*TotH.Veh*) show an enough good correlation ($0,38 < R < 0,20$).

With regard to the qualitative variables, although sometimes, depending on the groups (New Outline or Rehabilitation or Maintenance) the variance of *PaveType* levels was homogeneous or not, there was no mean difference, indicating that this variable did not influence. Moreover, when introduced in a GLM, it was not statistically significant, and hence, it was always discarded. On the contrary, *SurfType* showed non homogeneous variances and no mean difference in New Outlines but mean difference in rehabilitated or maintained stretches; but when included in GLM showed high significance, and improve the prediction of the model.

Since the affecting variables are mainly the same in all the cases, and in order to avoid the low quantity of observation in both cases, it is conducted an analysis including all the completely known pavement structures, regardless their classification. Nonetheless, “New Outline” and “Rehabilitation and Maintenance” stretches are examined as levels of a qualitative variables called Type of Work performed (*WorkType*), to analysis its (possible) influence.

9.3.1.4. Global consideration of completely known sections

The analysis of the known pavement sections is conducted with all the observations together. From the 112 observation from 2016, the ones with a real age below 2 years were discarded, following the ideas of the previous analysis and those from Kokkalis (1998) and Navarro *et al.* (2011). Consequently, 102 observations were available (42 + 60). An explanatory analysis of the data is presented in Table 9.53. The normality test for quantitative variables (Table 9.54) indicates that *MSSC* is the only one with normal distribution.

Table 9.53. Exploratory analysis for the quantitative variables in all the stretches with known pavement structure and real age not lower than 2 years.

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15	TotBit
Mean		53,25	7104,27	340,91	7,63	7,56	16940,6	949,4	47,29	25,43	18,45
Standard error		0,80	537,24	30,97	0,52	0,52	1648,3	112,0	0,40	1,53	0,56
95% CI for mean	**Lower**	51,66	6038,54	279,47	6,60	13670,9	13670,9	727,3	46,51	22,39	17,34
	Upper	54,85	8170,01	402,35	8,66	20210,4	20210,4	1171,4	48,08	28,48	19,56
Variance		66,06	29439479	97854	27,62	27,13	277114620	1278385	16,03	240,12	31,89
Std. deviation		8,13	5425,8	312,8	5,26	5,21	16647	1131	4,00	15,50	5,65
Minimum		34,27	665	23	2	2	1760	14	40	2,1	8
Maximum		73,32	29916	1361	29	28,6	98892	4447	50	45,8	37
Range		39,05	29251	1338	27	26,6	97132	4433	10	43,7	29
Interquartile range		13,65	6694	345	5	5,18	16898	1018	5	33,7	9
Skewness		0,011	1,239	1,288	1,873	1,874	2,318	1,691	-1,004	-0,503	0,395
Kurtosis		-0,384	2,263	0,835	4,146	4,105	7,136	2,063	-0,705	-1,506	0,056

Table 9.54 Normality tests for dependent and independent quantitative variables in all the stretches with known pavement structure and real age not lower than 2 years

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
MSSC	0,069	102	0,200*	0,985	102	0,286
AADT	0,258	102	<0,001	0,803	102	<0,001
H.AADT	0,118	102	0,001	0,905	102	<0,001
Age	0,197	102	<0,001	0,833	102	<0,001
R.Age	0,407	102	<0,001	0,655	102	<0,001
TotVeh	0,207	102	<0,001	0,811	102	<0,001
TotH.Veh	0,198	102	<0,001	0,814	102	<0,001
PSV	0,127	102	<0,001	0,949	102	0,001
Rain 15	0,181	102	<0,001	0,776	102	<0,001
TotBit	0,237	102	<0,001	0,749	102	<0,001

[a] Lilliefors Significance correction

* This is a lower bound of the true significance

Correlations are shown in Table 9.55 and the equations for each independent variable to fit the dependent one are displayed in Table 9.56.

Levels of the different qualitative variables (*PaveType*, *SurfType* and *WorkType*) were also analysed (Table 9.57)

Table 9.55. Correlation between dependent variables and independent variables (coefficient of Pearson) for all the stretches with known pavement structure and real age not lower than 2 years

Independent variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,452	<0,001
H.AADT	-0,382	<0,001
Age	-0,015	0,88
R.Age	-0,013	0,893
TotVeh	-0,381	<0,001
TotH.Veh	-0,356	<0,001
PSV	-0,448	<0,001
Rain 15	-0,052	0,606
TotBit	-0,115	0,251

Table 9.14056. Equations that best correlate each independent variable individually with the dependent variable for all the stretches with known pavement structure and real age not lower than 2 years

Independent Variable	Equation type	Resume of the model					Parameter estimates			
		R^2	F	Degrees of freedom 1	Degree of freedom 2	Sig.	Intercept	b1	b2	b3
AADT	Logarithm	0,300	42,767	1	100	<0,001	92	-4,558	-	-
H.AADT	Logarithm	0,210	26,554	1	100	<0,001	72,557	-3,592	-	-
Age	Linear	< 0,001	0,023	1	100	0,880	53,431	-0,023	-	-
R.Age	Linear	< 0,001	0,018	1	100	0,893	53,411	-0,021	-	-
TotVeh	Logarithm	0,274	37,787	1	100	<0,001	122,137	-4,251	-	-
TotH.Veh	Logarithm	0,212	26,865	1	100	<0,001	92,738	-3,02	-	-
PSV	Linear	0,201	25,088	1	100	<0,001	96,247	-,909	-	-
Rain 15	Linear	0,003	0,268	1	100	0,606	53,942	-,027	-	-
TotBit	Linear	0,013	1,333	1	100	0,251	56,298	-,165	-	-

Table 9.57141. Exploratory analysis for the qualitative variables in all the stretches with known pavement structure and real age not lower than 2 years, grouped by levels, for the dependent variables (MSSC).

		PaveType		SurfType			WorkType	
		Flexible	**Semi-rigid**	**AC**	**BBTM&PA**	**Slurry**	**New Outline**	**Maintenance**
N		58	44	42	25	35	42	60
Mean		54,45	51,67	54,15	48,31	55,71	51,04	54,80
Standard error		1,19	0,98	1,55	0,80	1,04	1,13	1,08
95% CI for mean	**Lower**	52,08	49,68	51,02	46,65	53,59	48,75	52,64
	Upper	56,83	53,65	57,27	49,96	57,83	53,33	56,96
Variance		81,57	42,53	100,77	16,09	38,17	53,91	69,71
Standard deviation		9,03	6,52	10,04	4,01	6,18	7,34	8,35
Minimum		36,00	34,27	34,27	40,83	43,60	36,20	34,27
Maximum		73,32	63,50	73,32	55,10	65,98	64,63	73,32
Range		37,32	29,23	39,05	14,27	22,38	28,43	39,05
Interquartile range		15,34	8,92	16,82	7,00	8,57	10,33	12,86
Skewness		-0,07	-0,41	-0,23	0,21	-0,41	0,04	-0,13
Kurtosis		-0,82	0,30	-0,63	-1,09	-0,70	-0,66	-,20

The Levene statistic shows that the levels for *PaveType* have not homogeneous variance and the *t*-test shows that there is not mean difference (Table 9.58).

Table 9.58142. Levene statistic for homogeneous variances and t-test for equality of means for levels of PaveType for MSSC in all the stretches with known pavement structure and real age not lower than 2 years

Levene statistic		**t-test for equality of means**							
F	**Sig.**	**Mean difference**	**t**	**Degrees of freedom**	**Significance (bilateral)**	**Mean difference**	**Standard error**	**Interval of 95% of confidence for the difference**	
								Lower	**Upper**
8,875	0,004	Equal variances assumed	1,732	100	0,086	2,786	1,609	-0,406	5,979
		Equal variances not assumed	1,809	99,791	0,073	2,786	1,540	-0,270	5,842

The Levene statistic for *WorkType* shows that null hypothesis is accepted (p-value > 0,05) and the variances are homogeneous and the t-test indicates that there is equality of means (p < 0,05) with a 95 of confidence level (Table 9.59).

Table 9.59. Levene statistic for homogeneous variances and t-test for equality of means for levels of WorkType for MSSC in all the stretches with known pavement structure and real age not lower than 2 years

Levene statistic		**t-test for equality of means**							
F	**Sig.**	**Mean difference**	**t**	**Degrees of freedom**	**Significance (bilateral)**	**Mean difference**	**Standard error**	**Interval of 95% of confidence for the difference**	
								Lower	**Upper**
0,717	0,399	Equal variances assumed	-2,349	100	0,021	-3,758	1,600	-6,932	-0,584
		Equal variances not assumed	-2,403	94,827	0,018	-3,758	1,564	-6,863	-0,654

The Levene statistic for *SurfType* shows that null hypothesis is rejected (p-value < 0,05) (Table 9.60) and the variances are not homogeneous. The robust tests for equality of means (Table 9.61) and the ANOVA (Table 9.62) indicate that there is not mean equality. Additionally the multiple comparison between levels by means of Tamhane's T2 and Dunnett's T3 statistics indicate that the difference appears between AC and BBTM&PA, and between BBTM&PA and LB2, but not between AC and LB2 (Table 9.63).

A multiple linear regression model was tried with all the quantitative variables (and their transformations) to observe the ones that best correlate with *MSSC*. The Forward and Step by Step functions of IBM SPSS v24 programme were employed and Ln*AADT* was the only variable in the model with global and individual significance over 95% (Table 9.64).

Table 9.60 Test of homogeneity of variances for levels in SurfType for the dependent variable (MSSC) in all the stretches with known pavement structure with age not lower than 2 years

Levene Statistic	**Degrees of Freedom 1**	**Degrees of Freedom 2**	**Significance**
10,982	2	99	< 0,001

Table 9.61. Robust test of equality of means for levels in SurfType for the dependent variable (MSSC) in all the stretches with known pavement structure with age not lower than 2 years

Test	Statistic	Degrees of freedom 1	Degrees of freedom 2	Significance
Welch	17,342	2	65,934	< 0,001
Brown-Forsythe	8,881	2	84,391	< 0,001

Table 9.62. ANOVA for levels in SurfType for the dependent variable (MSSC) in all the stretches with known pavement structure with age not lower than 2 years

Variable (SurfType)	Sum of Squares	Degrees of freedom	Mean Square	F	Significance
Between groups	856,900	2	428,450	7,294	0,001
Within groups	5815,240	99	58,740		
Total	6672,140	101			

Table 9.63. Multiple comparison for SurfType for the dependent variable (MSSC) in all the stretches with known pavement structure with age not lower than 2 years

Statistic	SurfType (I)	SurfType (J)	Mean Differences (I-J)	Std Error	Sig	95% confidence interval	
						Lower	Upper
HSD Tukey	AC	BBTM&PA	5,840	1,744	0,004	1,553	10,127
		Slurry	-1,566	1,868	0,789	-6,136	3,005
	BBTM&PA	AC	-5,840	1,744	0,004	-10,127	-1,553
		Slurry	-7,406	1,317	<0,001	-10,644	-4,167
	Slurry	AC	1,566	1,868	0,789	-3,005	6,136
		BBTM&PA	7,406	1,317	0,000	4,167	10,644
Bonferroni	AC	BBTM&PA	5,840	1,744	0,004	1,557	10,123
		Slurry	-1,566	1,868	0,786	-6,132	3,001
	BBTM&PA	AC	-5,840	1,744	0,004	-10,123	-1,557
		Slurry	-7,406	1,317	<0,001	-10,641	-4,171
	Slurry	AC	1,566	1,868	0,786	-3,001	6,132
		BBTM&PA	7,406	1,317	<0,001	4,171	10,641

Various General Linear Models were examined. Firstly, a model of the form of Eq. 9.6 was tried to observe the significance of the qualitative variables (R^2 = 0,508 and adjusted R^2 = 0,448) (Table 9.65).

$$MSSC = Intercept + LnAADT + LnH.AADT + PSV_{req} + SurfType + PaveType +$$

$$+ WorkType + SurfType * WorkType + SurfType * PaveType + WorkType * PaveType +$$

$$+ SurfType * WorkType * PaveType \qquad [9.6]$$

Table 9.64. Analysis of variance of the multiple linear regression model proposed for all the stretches with known pavement structure with age not lower than 2 years

			Analysis of variance			
Source	**Degrees of freedom**	**Sum of squares**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**
Model	1998,686	1	1998,686	42,767	<0,001	1,265
Error	4673,453	100	46,735			
Corrected total	6672,140	101				
Root mean square error	**Coefficient of variation**	**R**	**R^2**	**Adj. R^2**	**Colinearity statistics**	
					Tolerance	**VIF**
6,836		0,547	0,300	0,293	1,000	1,000
			Parameter estimates			
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**	
Intercept	92,000	5,964	15,427	<0,001	80,168	103,831
LnAADT	-4,558	,697	-6,540	<0,001	-5,940	-3,175

Table 9.65143. Test of Between-Subjects effects for model of Eq. 9.6 for all the stretches with known pavement structure with age not lower than 2 years.

Origin	**Type III Sum of Squares**	**Degree of freedom**	**Mean Square**	**F**	**Sig.**	**Partial eta-squared**	**Non centrality parameter**	**Observed Power**
Corrected model	3391,539	11	308,322	8,458	<0,001	0,508	93,043	1,000
Intercept	4590,841	1	4590,841	125,94	<0,001	0,583	125,945	1,000
LnAADT	219,266	1	219,266	6,015	0,016	0,063	6,015	0,68
LnH.AADT	17,923	1	17,923	0,492	0,485	0,005	0,492	0,107
PSVreq	144,623	1	144,623	3,968	0,049	0,042	3,968	0,504
SurfType	890,11	2	445,055	12,21	<0,001	0,213	24,419	0,995
WorkType	29,284	1	29,284	0,803	0,372	0,009	0,803	0,144
PaveType	42,722	1	42,722	1,172	0,282	0,013	1,172	0,188
SurfType * WorkType	5,912	1	5,912	0,162	0,688	0,002	0,162	0,068
SurfType * PaveType	76,323	2	38,161	1,047	0,355	0,023	2,094	0,228
WorkType * PaveType	124,477	1	124,477	3,415	0,068	0,037	3,415	0,448
SurfType * WorkType * PaveType	0	0	.	.	.	0	0	.
Error	3280,601	90	36,451					
Total	295919,58	102						
Corrected total	6672,14	101						

As observed, variables *WorkType* and *PaveType* (and their combination) show low signification. *WorkType* has been discarded, demonstrating that it is not an influencing factor in the prediction of the available friction. Another GLM was tried without *WorkType*, following Eq. 9.3. It was obtained a R^2 = 0,489 and an adjusted R^2 = 0,446 (Table 9.66).

Table 9.66. Test of Between-Subjects effects for model of Eq. 9.3 for all the stretches with known pavement structure with age not lower than 2 years.

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	3265,985	8	408,248	11,14	<0,001	0,489	89,173	1,000
Intercept	5593,539	1	5593,539	152,72	<0,001	0,622	152,723	1,000
LnAADT	249,217	1	249,217	6,80	0,011	0,068	6,804	0,733
LnH.AADT	15,467	1	15,467	0,42	0,517	0,005	0,422	0,099
PSVreq	189,044	1	189,044	5,16	0,025	0,053	5,162	0,614
SurfType	1147,412	2	573,706	15,66	<0,001	0,252	31,328	0,999
PaveType	13,539	1	13,539	0,37	0,545	0,004	0,37	0,092
SurfType * PaveType	75,207	2	37,604	1,03	0,362	0,022	2,053	0,224
Error	3406,154	93	36,625					
Total	295919,58	102						
Corrected total	6672,14	101						

PaveType and Ln*H.AADT* still show low significance. *PaveType* has been discarded since it is not influencing the skid resistance. A new GLM without *PaveType* shows that Ln*H.AADT* has still no significance. Therefore, it is discarded from the model. A model as displayed in Eq. 9.7 is developed (Table 9.67 and Table 9.68), obtaining a R^2 = 0,466 and adjusted R^2 = 0,444. All the variables show significance over 90%

$$MSSC = Intercept + LnAADT + PSV_{req} + SurfType \qquad [9.14]$$

Table 9.67144. Test of Between-Subjects effects for model of Eq. 9.7 for all the stretches with known pavement structure with age not lower than 2 years.

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	3112,197	4	778,049	21,2	<0,001	0,466	84,8	1
Intercept	5560,814	1	5560,814	151,519	<0,001	0,61	151,519	1
LnAADT	750,149	1	750,149	20,44	<0,001	0,174	20,44	0,994
PSVreq	116,165	1	116,165	3,165	0,078	0,032	3,165	0,421
SurfType	1109,204	2	554,602	15,112	<0,001	0,238	30,223	0,999
Error	3559,942	97	36,7					
Total	295919,58	102						
Corrected total	6672,14	101						

An alternative GLM was also tried including the Ln*H.AADT*, PSV_{req} and *SurfType*, following the ideas of Szatkowski and Hosking (1972). All these variables were significant (Table 9.69) but a lower R^2 values were obtained (R^2 = 0,387; adjusted R^2 = 0,362).

Table 9.68145. Parameter estimates for model of Eq. 9.7 for all the stretches with known pavement structure with age not lower than 2 years

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	U`pper			
Intercept	119,797	9,647	12,418	<0,001	100,65	138,943	0,614	12,418	1
LnAADT	-4,661	1,031	-4,521	<0,001	-6,707	-2,615	0,174	4,521	0,994
PSVreq	-0,464	0,261	-1,779	0,078	-0,982	0,054	0,032	1,779	0,421
[SurfType=1]	-8,643	1,692	-5,109	<0,001	-12	-5,285	0,212	5,109	0,999
[SurfType=2]	-5,737	1,602	-3,581	0,001	-8,917	-2,558	0,117	3,581	0,944
[SurfType=3]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

Table 9.69. Test of Between-Subjects effects for model including LnH.AADT, PSV_{req} and SurfType for all the stretches with known pavement structure with age not lower than 2 years

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	2581,54	4	645,385	15,304	<0,001	0,387	61,216	1
Intercept	5144,756	1	5144,756	121,997	<0,001	0,557	121,997	1
LnH.AADT	219,491	1	219,491	5,205	0,025	0,051	5,205	0,617
PSVreq	571,996	1	571,996	13,564	<0,001	0,123	13,564	0,954
SurfType	996,078	2	498,039	11,81	<0,001	0,196	23,62	0,993
Error	4090,6	97	42,171					
Total	295919,58	102						
Corrected total	6672,14	101						

9.3.1.5. Conclusions from analyzing completely known sections

After the complete analysis for skid resistance with known pavement sections it can be concluded that it is not necessary to know the pavement structure. It has been demonstrated that *PaveType*, the variable that distinguishes between flexible and semi-rigid pavements do not influence the friction prediction. Additionally, the variable *TotBit*, which indicates the total thickness in cm of the bituminous layers has shown very low correlation with the dependent variable (*MSSC*), and has not included in any model with a 95% of significance. Moreover, the analysis according to groups of *WorkType* (New Outline or Rehabilitation and Maintenance) has shown that it is not an influencing factor, and, when analyzed all the data together, it is a qualitative variable with low signification.

Similar conclusions were also obtained in previous work for friction prediction (Pérez-Acebo *et al.* 2017a). Nonetheless, it was aimed to be demonstrated that for skid resistance performance models it is not necessary to know the complete pavement structure. This idea is well known in the literature, where many times has been probed that structural characteristics of the pavements do not influence nor has not relationship with surface characteristics (Pérez-Acebo *et al.*, 2018b; Flora, 2009; Prang *et al.*, 2012).

The remaining variables (*AADT*, *H.AADT*, PSV_{req}, *TotVeh*, *TotH.Veh*, *Age*, *R.Age* and *SurfType*) can be obtained if the data about the surface layer is known. Therefore, the Surface Layer file of the PMS of the

Regional Government of Biscay can be used as the characteristics of the surface are the only required. Traffic data are also available. Consequently, the complete length of the roads included in the road network managed by the RGB can be examined to predict the skid resistance. This analysis is conducted in section 9.4.

9.4. Skid resistance prediction models for known surface layers

As concluded in previous section, for the skid resistance modelling, surface characteristics and traffic data are the only required data. Hence, apart from the data of completely known pavement sections, the resting length of each road can be also introduced in the model development. With that aim, the Surface Layer files of all the roads are obtained and the total length is divided in stretches according to surface layer and when it was carried out, i.e. according to the last project in that stretch which modified the surface layer. Hence, division is mainly carried according to the projects that have been conducted in it. Even when similar surface type is placed in two contiguous areas, if they were carried out in different moments, they are indicated as different surfaces. This exhaustive analysis of the division of each road is exposed in the Road Files of the Annex II. In the last part of these files, under title "*Skid resistance of surface layers in 2016*" the complete division of the road is displayed. It is indicated the initial and final Kilometre Points, the project which is responsible of that actuation, the data of the work and the surface layer material and its thickness, as shown in Table 9.70.

Table 9.70. Example of data included in the section "Skid resistance of surface layers in 2016" in the Road Files of the Annex II of road BI-631 (Area 1)

Skid resistance of surface layers in 2016
19+1040 – 19+1480: PROJ-1334 (01/07/2011) LB2. 20+0000 – 21+0600: PROJ-1335 (01/07/2015) LB2. 21+0600 – 22+0190: PROJ-1337 (01/07/2011) LB2. 23+0000 – 23+0320: PROJ-280 (01/06/2012) AC 16 surf S 6 cm. 23+0680 – 23+0800: PROJ-280 (01/06/2012) AC 16 surf S 6 cm.

Once the road is divided according to the surface characteristics, data traffic make another division, which may not coincide with the previous one. These two divisions create stretches with similar characteristics, about both surface and traffic, with variable length (and number of observations) (Fig. 9.4). Once again, it is not possible to include all the observation (each 20 m) because there is a great variability within the observations with the same data in the independent variables. Therefore, as conducted in IRI analysis and in previous skid resistance analysis, the mean value of the stretch and the standard deviation are calculated. The data for pavement texture are also calculated following the exposed criteria.

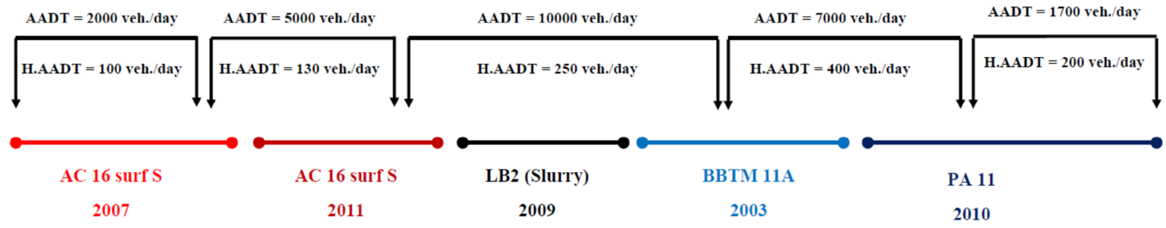


Fig. 9.4. Examples of road length division by surface characteristics and traffic data for analysis for skid resistance modelling.

The data incorporated to each observation is described below (Table 9.71):

- *Identification of the stretch*: the same columns as for IRI analysis (*columns A* to G) (section 8.4.1) and the ones for skid resistance analysis of completely known pavement sections are employed.
- *Skid resistance data*: *Skid resistance data*: **SCRIM Coefficient** (*column S1*) for the considered stretch of 20 m, as explained in 6.6.2, expressed from 0 to 100; and the **Texture** (*column S2*), in mm, representing the Mean Profile Depth.
- *Traffic data*: the same information as for IRI data and for skid resistance analysis. *Columns T1* to *T5* are deployed. Data of *columns T1*, *T2* and *T3* are referred to year 2016, the year of the data collection, and *columns T4* and *T5* indicate the accumulated total vehicles and accumulated total heavy vehicles since opening or M&R work until 2016, respectively.
- *Rainfall data*: rainfall data during the previous 15 days to the friction data collection in the area of the road from the nearest meteorological station, **Rainfall data**, in mm, are incorporated in *column S3*.
- *Polished Stone Value*: similar information as described in section 9.3.1 is included: H.AADT of the year when the new outline or M&R activity was finished (*column S4*), the corresponding heavy traffic category according to Ministerio de Fomento (2003b) (*column S5*), and finally the required minimum PSV (*column S6*) (Table 9.8).
- *Structural data*: only some information about the pavement section can be introduced from the Surface file, which is not as complete as what can be obtained from a Pavement File. Included data are: **date of last work**, which can be referred to the date that the stretch was opened to traffic or the date of the last M&R work (*column I*); **the year of the last work** (*column J*); **the year of the last work, in numer** (*column K*); **age** (*column L*), obtained subtracting column J to column F (*column L = column F – column J*); **real age** (*column M*), obtained subtracting column K to column G (*column M = column G – column K*); and **surface layer material** (*column N*), indicating the material extended in the surface layer.

Table 9.71. Data for each stretch in the analysis of known surface layers for skid resistance modelling, with example values from road BI-631.

A	B	C	D	E	F	G	S1	S2
Road	**Axle**	**Initial PK**	**Final PK**	**Date of data collection**	**Year of data collection**	**Year of data collection, in number**	**MSSC**	**Texture**
BI-631	Bi-631 unique	19+0990	19+1009	30/06/2016	2016	2016,5	49	0,62
BI-631	Bi-631 unique	19+1010	19+1029	30/06/2016	2016	2016,5	54	0,88

T1	T2	T3	T4	T5	S3	S4	S5	S6
AADT in 2016	**H.ADDT in 2016**	**Traffic category in 2016**	**Total Vehicles**	**Total Heavy vehicles**	**Rain15**	**H.AADT of year of last work**	**Traffic category of year of last work**	**PSV required**
11.840	189	T31	20.220.635	432.605	34,3	337	T2	50
11.840	189	T31	20.220.635	432.605	34,3	337	T2	50

I	J	K	L	M	N
Date of last work	**Year of last work**	**Year of last work in number**	**Age**	**Real Age**	**Surface layer material**
01/07/2011	2011	2011,5	5	5	LB2
01/07/2011	2011	2011,5	5	5	LB2

Following this methodology, and calculating the mean of the stretches with the same values in the variables (and the typical deviation, Eq. 9.2), more observations are available from each road. Table 9.72 shows the stretches considered in each road, distinguishing if they were collected in a two-lane road or in a double carriageway road.

9.4.1. Skid resistance prediction models for known surface layers for two-lane roads in Areas 1, 2 and 3

Initially, only the two-lane roads are analyzed. That is the reason for only considering Areas 1, 2 and 3, as Area 4 includes the freeways of the metropolitan area of Bilbao (despite some stretches of two-lane roads).

There are 112 and 321 observation from surface layer analysis in two-lane roads analysis in Area 1, 2 and 3, giving a total of 433 observation. Following exposed criteria, the stretches with a pavement that is not 2 years old (*R.Age* < 2 years) (44 stretches) are discarded since it is said not to have entered the equilibrium phase. After removing these data, a total of 389 are analyzed for modelling.

Table 9.73 shows the explanatory analysis of the data according to the variables. No one of the variables follow a normal distribution (Table 9.74). Correlations between quantitative independent variables and the dependent variable are shown in Table 9.75 and the best curve that fits the data between each independent variable and the dependent in Table 9.76.

For the qualitative independent variable, *SurfType*, an analysis similar to previous sections was conducted (Table 9.77). It is observed that there are not homogeneous variances (Table 9.78) and it can be stated that there is not mean equality (Table 9.79 and Table 9.80) with a 95% of confidence. Post-hoc comparisons show that there is no mean equality between LB and BBTM&PA and between Slurry and AC, but there is mean equality between AC and BBTM&PA (Table 9.81).

Table 9.14672. Stretches considered in each road of Areas 1, 2, 3 according to their surface layer project and traffic volume for skid resistance modelling (stretches from two-lane roads and double carriageway roads)

Area 1			Area 2			Area 3		
Road	**2L 2016**	**2C 2016**	**Road**	**2L 2016**	**2C 2016**	**Road**	**2L 2016**	**2C 2016**
BI-631	9		BI-623	9	2	BI-624	2	
BI-634	7		BI-633	15		BI-625	20	5
BI-635	15		BI-638	2		BI-630	12	
BI-735	5		BI-732	4		BI-712	4	
BI-737	7		BI-2224	5		BI-745	3	
BI-2101	2		BI-2301	2		BI-2521	4	
BI-2120	11		BI-2405	4		BI-2522	6	
BI-2121	26		BI-2543	6		BI-2617	0	
BI-2122	7		BI-2632	1		BI-2625	2	
Bi-2153	5		BI-2636	2		BI-2701	27	
Bi-2235	9		N-240	38		BI-2757	0	
Bi-2237	6		N-636	0		BI-2794	1	
Bi-2238	13					N-639	5	
BI-2704	11							
BI-2713	10							
BI-2731	4							
Total	147		**Total**	88	2	**Total**	86	5
Total conventional roads (Areas 1, 2, 3)					321			
Total double carriageway roads (Areas 1, 2, 3)					7			
TOTAL					328			

Table 9.73. Exploratory analysis for the quantitative variables in all the stretches with known surface layer in Areas 1, 2 and 3 in two-lane roads with real age not lower than 2 years

		MSSC	**AADT**	**H.AADT**	**Age**	**R.Age**	**TotVeh**	**TotH.Veh**	**PSV**	**Rain15**
Mean		53,25	7261,70	318,29	7,50	7,47	19482,19	1058,89	47,46	27,08
Standard error		0,46	297,30	15,95	0,25	0,25	1147,80	81,02	0,18	0,73
95% CI for mean	**Lower**	52,35	6677,19	286,92	7,00	6,98	17225,51	899,60	47,11	25,64
	Upper	54,15	7846,22	349,65	8,00	7,97	21738,87	1218,18	47,80	28,52
Variance		82,334	34381643	99004	24,843	24,948	512483095	2553390	11,88	208,27
Std. deviation		9,074	5864	315	4,984	4,995	22638	1598	3,45	14,43
Minimum		33	654	10	2	2	936,04	13,96	40	2,1
Maximum		85,875	29916	1361	29	28,6	165782,51	8022,56	50	45,8
Range		52,875	29262	1351	27	26,6	164846,47	8008,6	10	43,7
Interquartile range		13,351	7267	327	6	5,7	19439,81	747,16	5	25,4
Skewness		0,312	1,42	1,53	1,566	1,585	2,85	2,457	-0,956	-0,75
Kurtosis		-0,038	2,408	1,542	3,004	3,01	10,765	5,863	-0,414	-1,093

Table 9.74. Normality tests for dependent and independent quantitative variables all the stretches with known surface layer in Areas 1. 2 and 3 in two-lane roads with real age not lower than 2 years

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value)	Statistical	Degree of freedom	Significance (p-value)
MSSC	0,048	389	0,034	0,991	389	0,019
AADT	0,145	389	<0,001	0,868	389	<0,001
H.AADT	0,211	389	<0,001	0,792	389	<0,001
Age	0,135	389	<0,001	0,855	389	<0,001
R.Age	0,137	389	<0,001	0,853	389	<0,001
TotVeh	0,206	389	<0,001	0,702	389	<0,001
TotH.Veh	0,293	389	<0,001	0,629	389	<0,001
PSV	0,379	389	<0,001	0,708	389	<0,001
Rain 15	0,293	389	<0,001	0,8	389	<0,001

[a] Lilliefors Significance correction

Table 9.75. Correlation between the dependent variable and independent variables (coefficient of Pearson) for all the stretches with known surface layer in Areas 1, 2 and 3 in two-lane roads and with real age not lower than 2 years

Independent variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,541	<0,001
H.AADT	-0,459	<0,001
Age	-0,093	0,067
R.Age	-0,086	0,089
TotVeh	-0,407	<0,001
TotH.Veh	-0,34	<0,001
PSV	-0,305	<0,001
Rain 15	-0,078	0,123

Table 9.76. Equations that best correlate each independent variable individually with the dependent variable for all the stretches with known surface layer in Areas 1, 2 and 3 in two-lane roads with an age not lower than 2 years.

Independent Variable	Equation type	Resume of the model					Parameter est.			
		R^2	F	Degrees of freedom 1	Degree of freedom 2	Sig.	Intercept	b1	b2	b3
AADT	Logarithm	0,342	200,865	1	387	<0,001	103,837	-5,922	-	-
H.AADT	Logarithm	0,274	145,709	1	387	<0,001	78,736	-4,804	-	-
Age	Linear	0,009	3,372	1	387	0,067	54,519	-0,169	-	-
R.Age	Linear	0,007	2,91	1	387	0,089	54,423	-0,157	-	-
TotVeh	Logarithm	0,303	168,18	1	387	<0,001	97,735	-4,755	-	-
TotH.Veh	Logarithm	0,252	130,093	1	387	<0,001	75,083	-3,562	-	-
PSV	Linear	0,093	39,677	1	387	<0,001	91,342	-0,803	-	-
Rain 15	Exponential	0,009	3,465	1	387	0,063	54,096	-0,001	-	-

Table 9.77 Exploratory analysis for the SurfType levels for the dependent variables (MSSC) for all the stretches with known surface layer in Areas 1. 2 and 3 in two-lane roads with an age not lower than 2 years.

		SurfType		
		AC (1)	**BBTM& PA (2)**	**Slurry (3)**
N		143	59	187
Mean		50,348	48,407	56,997
Standard error		0,749	0,753	0,621
95% CI for mean	**Lower**	48,868	46,900	55,772
	Upper	51,828	49,914	58,222
Variance		80,166	33,431	72,097
Standard deviation		8,954	5,782	8,491
Minimum		33	37,744	38
Maximum		73,318	63,495	85,875
Range		40,318	25,751	47,875
Interquartile range		12,635	8,131	12,006
Skewness		0,192	0,775	0,347
Kurtosis		-0,446	0,197	0,142

Table 9.78. Test of homogeneity of variances for levels in SurfType for the dependent variable (MSSC) for all the stretches with known surface layer in Areas 1. 2 and 3 in two-lane roads with an age not lower than 2 years

Levene Statistic	Degrees of Freedom 1	Degrees of Freedom 2	Significance
6,156	2	386	0,002

Table 9.79. Robust test of equality of means for levels in SurfType for the dependent variable (MSSC) for all the stretches with known surface layer in Areas 1. 2 and 3 in two-lane roads with an age not lower than 2 years

Test	Statistic	Degrees of freedom 1	Degrees of freedom 2	Significance
Welch	44,948	2	189,160	< 0,001
Brown-Forsythe	44,749	2	343,551	< 0,001

Table 9.80. ANOVA for levels in SurfType for the dependent variable (MSSC) for all the stretches with known surface layer in Areas 1. 2 and 3 in two-lane roads with an age not lower than 2 years.

Variable (SurfType)	Sum of Squares	Degrees of freedom	Mean Square	F	Significance
Between groups	5213,067	2	2606,534	37,636	< 0,001
Within groups	26732,712	386	69,256		
Total	31945,780	388			

Some multiple linear regression models were tried. The one with best coefficient of determination includes Ln*AADT* and Ln*TotVeh* as the unique variables (Table 9.82). However, problems of autocorrelation can be observed (DW statistic = 1,137 and Condition Index > 30 (Table 9.83).

Table 9.81. Multiple comparison for SurfType levels for the dependent variable (MSSC) for all the stretches with known surface layer in Areas 1. 2 and 3 in two-lane roads with an age not lower than 2 years.

Statistic	SurfType (I)	SurfType (J)	Mean Differences (I-J)	Std Error	Sig	95% confidence interval	
						Lower	Upper
Tamhane's T2	AC	BBTM&PA	1,941	1,062	0,194	-0,620	4,503
		Slurry	-6,649	0,973	<0,001	-8,984	-4,313
	BBTM&PA	AC	-1,941	1,062	0,194	-4,503	0,620
		Slurry	-8,590	0,976	<0,001	-10,947	-6,232
	Slurry	AC	6,649	0,973	<0,001	4,313	8,984
		BBTM&PA	8,590	0,976	<0,001	6,232	10,947
Dunnett's T3	AC	BBTM&PA	1,941	1,062	0,193	-0,619	4,502
		Slurry	-6,649	0,973	<0,001	-8,984	-4,313
	BBTM&PA	AC	-1,941	1,062	0,193	-4,502	0,619
		Slurry	-8,590	0,976	<0,001	-10,946	-6,233
	Slurry	AC	6,649	0,973	<0,001	4,313	8,984
		BBTM&PA	8,590	0,976	<0,001	6,233	10,946

Table 9.82. Analysis of variance of the multiple linear regression model proposed for all the stretches with known surface layer in Areas 1, 2 and 3 with an age not lower than 2 years

Analysis of variance						
Source	**Sum of squares**	**Degrees of freedom**	**Mean squares**	**F value**	**p-value**	**Durbin-Watson**
Model	11619,972	2	5809,986	110,335	<0,001	1,137
Error	20325,808	386	52,658			
Corrected total	31945,780	388				
Root mean square error	**Coefficient of variation**	**R**	**R^2**	**Adj. R^2**	**Colinearity statistics**	
					Tolerance	**VIF**
	7,257	0,603	0,364	0,360	0,386	2,593
Parameter estimates						
Variable	**Parameter estimate**	**Standard error**	**t value**	**p-value**	**95% confidence limits**	
Intercept	106,944	3,633	29,433	<0,001	99,800	114,088
LnAADT	-4,023	0,662	-6,074	<0,001	-5,325	-2,721
LNTotVeh	-2,066	0,565	-3,658	<0,001	-3,177	-0,956

Table 9.83. Colinearity diagnostics

Model	Dimension	Eigenvalue	Condition Index
1	1	2,991	1,000
	2	0,007	20,703
	3	0,002	34,664

Alternatively, a linear model including Ln*AADT* and PSV_{req} show a similar R^2 (0,354) and a Durbin-Watson statistic of 1,136 and similar condition index values.

Different General Linear Models were tried. With the previous experiences in modelling and the correlation between the dependent variable and the independent variables, different combinations of possible independent variables (introduced as covariables or factors) have been analyzed. Table 9.84, shows the equations that have been tried, the coefficient of determination, R^2, and the adjusted R^2 are exposed and the main problems that have detected from the data. This preliminary analysis comes principally from the Table of Test of Between-Subjects effects for each model, which indicates the significance of each variable in the F test. Additionally, the with the Parameter estimates for each model indicates the individual significance of each coefficient, by means of the t-test. These are some of the first data that must be checked, as explained in Chapter 7. Once these items have been checked, further data are analyzed. In the Table 9.104, the main observations and comments for each model are exposed in the last column. All the models include the intercept (Int) and variables that are considered as factors are indicated as (f). Apart from *SurfType*, which groups the surface layers in Asphalt Concrete (AC), Discontinuous and Porous Asphalt (BBTM&PA) and Slurry (LB2), it has also been used the variable *SurfDen*, as exposed in Table 9.11, which distinguishes between these groups, corresponding to each surface type: AC 16 surf S (1), AC 22 surf S (2), BBTM 11A (3), PA 11 (4) and LB2 (5). These surface types are the only ones that can be found in two-lane roads in Areas 1, 2 and 3 in Biscay.

Table 9.84147. Proposed General Linear Models for all the stretches with known surface layer in Areas 1, 2 and 3 in two-lane roads with real age not lower than 2 years.

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + LnAADT + LnH.AADT + PSV + SurfType(f)	0,491	0,484	Low significance for LnH.AADT (p=0,798) and PSV (p=0,955)
MSSC = Int + LnAADT + LnH.AADT + PSV(f) + SurfType(f) + PSV*SurfType	0,497	0,482	Low significance for LnH.AADT (p=0,592), PSV (p=0,748) and PSV*SurfType (p=0,454)
MSSC = Int + LnAADT + PSV + SurfType(f)	0,490	0,485	Low significance for PSV (p=0,987)
MSSC = Int + LnAADT + PSV(f) + SurfType(f)+ PSV*SurfType	0,496	0,483	Low significance for PSV (p=0,768) and PSV*SurfType (p=0,480)
MSSC = Int + LnAADT + PSV + SurfType(f)	0,490	0,485	Low significance for PSV (p=0.,987)
MSSC = Int + LnAADT + SurfType(f)	0,490	0,486	All the variables have a p-value<0,001
MSSC = Int + LnH.AADT + PSV + SurfType(f)	0,407	0,401	All the variables have a p-value<0,02
MSSC = Int + LnAADT + LnH.AADT + SurfType(f)	0,491	0,485	Low significance for LnH.AADT (p=0,803)
MSSC = Int + LnAADT + LnH.AADT + PSV + SurfDen(f)	0,491	0,482	Low significance for LnH.AADT (p=0,735) and PSV (p=0,994)
MSSC = Int + LnAADT + LnH.AADT + SurfDen(f)	0,491	0,483	Low significance for LnH.AADT (p=0,731)
MSSC = Int + LnAADT + SurfDen(f)	0,491	0,484	All the variables have a p-value<0,001

As observed, the best models with all the variables being significant (p-value < 0,05) is obtained including only Ln*AADT* as covariable and *SurfType* or *SurfDen* as fixed factors. Since the Ln*AADT* was the only quantitative variable in the models and *SurfType* and *SurfDen* the only fixed factors, it was tried to produce independent models for each surface type. However, obtained coefficient of determination were lower, and the differentiation between the subgroups in *SurfType* did not produce improvements (for AC: R^2 = 0,392; for BBTM&PA, R^2 =0,283 and for Slurry, R^2 =0,44).

As a next step in the research, it was decided to include the available stretches from double carriageway roads in the skid resistance modelling. In the IRI analysis, the presence of 2 or more lanes in a carriageway would affect the lane with the higher traffic of heavy vehicles absorbing some of the deformations (which could be detected by the IRI) differently from a two-lane road, where both lanes are supposed to have similar heavy traffic. Unlike in the IRI analysis, it is thought that the presence of more lanes will not affect the polishing effect on the lane with the highest heavy traffic volume. Consequently, the stretches from the double carriageway stretches from Areas 1, 2 and 3 are incorporated to the analysis. If an improvement in the correlation and in the models is observed, stretches from Area 4, which principally include freeways with double carriageways, will be included.

9.4.2. Skid resistance prediction models for known surface layers for all roads in Areas 1, 2 and 3

As indicated in Table 9.9 and Table 9.72 there are 451 stretches (123+328) coming from different projects with known surface layers and different traffic volumes. Once again, and following previous criteria, stretches with a real age lower than 2 years are discarded, as suggested by Kokkalis (1998). Moreover, in some stretches experimental micro-milling were conducted in 2015 and 2016 to improve the friction, and therefore, these sections must be discarded. From the 451 total observations, 46 are not 2 years old and hence are eliminated. The total amount of observations for the analysis of roads in Areas 1, 2 and 3 are 405.

Table 9.85 shows the correlation between the dependent variable and each of the independent quantitative variables. Table 9.86 shows the equation of the best curves that fits the data between the dependent variable (*MSSC*) and each of the independent variable. If these tables are compared with Table 9.75 and Table 9.76 of the previous analysis without double carriageway stretches, a low improvement is observed for the most correlated variables.

Table 9.85. Correlation between the dependent variable and the independent variables (coefficient of Pearson) in the all the stretches of Areas 1, 2 and 3 with a real age not lower than 2 years

Independent Variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,545	<0,001
H.AADT	-0,459	<0,001
Age	-0,104	0,037
R.Age	-0,097	0,05
TotVeh	-0,4	<0,001
TotH.Veh	-0,355	<0,001
PSV	-0,31	<0,001
Rain 15	-0,084	0,091

Some General Linear Models are tried, following the criteria of subsection 9.4.2. Table 9.87 shows the analyzed models and the comments for them, becoming a summary of conducted analysis.

Table 9.86. Equations that best correlate each independent variable individually with the dependent variable in the all the stretches of Areas 1, 2 and 3 with a real age not lower than 2 years.

Independent Variable	Equation type	Resume of the model					Parameter estimates			
		R^2	F	Degrees of freedom 1	Degree of freedom 2	Sig.	Intercept	b1	b2	b3
AADT	Logarithm	0,344	211,074	1	403	< 0,001	103,73	-5,907	-	-
H.AADT	Logarithm	0,276	153,706	1	403	< 0,001	78,95	-4,862	-	-
Age	Linear	,011	4,372	1	403	,037	54,488	-,189	-	-
R.Age	Linear	,009	3,865	1	403	,050	54,397	-,177	-	-
TotVeh	Logarithm	0,310	181,189	1	403	< 0,001	129,95	-4,717	-	-
TotH.Veh	Logarithm	0,261	142,582	1	403	< 0,001	100,05	-3,595	-	-
PSV	Linear	0,096	42,937	1	403	< 0,001	92,18	-0,822	-	-
Rain15	Linear	,007	2,877	1	403	,091	54,532	-,054	-	-

Table 9.87. Proposed General Linear Models for all the stretches with known surface layer in all the stretches of Areas 1, 2 and 3 with real age not lower than 2 years.

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + LnAADT + PSV(f) + SurfType(f)+ + PSV*SurfType	0,499	0,486	Low significance for PSV (p=0,767) and PSV*SurfType (p=0,451)
MSSC = Int + LnH.AADT + PSV(f) + SurfType(f)+ + PSV*SurfType	0,413	0,398	Low significance for PSV (p=0,575) and PSV*SurfType (p=0,884)
MSSC = Int + LnAADT + SurfType(f)	0,493	0,489	All the variables have a p-value<0,001
MSSC = Int + LnH.AADT + PSV + SurfType(f)	0,410	0,404	All the variables have a p-value<0,02

As observed, a low improvement is observed again when compared with models that do not include double carriageway stretches (Table 9.84). The low improvement can be attributed to the low quantity of double carriageway stretches incorporated to the analysis (from 389 observations to 405, after including 16 double carriageway stretches, 10 from completely known pavement sections and 6 from known surface layers). Consequently, stretches from Area 4, principally including double carriageway stretches are decided to be incorporated to the skid resistance analysis.

9.4.3. Skid resistance prediction models for known surface layers for any type of road in all Areas (1, 2, 3 and 4)

As concluded in previous subsection (9.4.2) the introduction of double carriageway stretches from Areas 1, 2 and 3 in the skid resistance modelling not only did not introduce error in the model but did improve the results when comparing with values obtained when not including them. Consequently, it was decided to develop a friction model that incorporates any type of road (two-lane or double carriageway roads). Consequently, stretches from roads of Area 4, which includes the main roads in the Metropolitan Area of Bilbao, i.e. freeways (double carriageway) around the metropolitan ring of Bilbao are introduced in the modelling. A similar analysis of the Surface Layer files of roads of Area 4 is performed, observing the projects that has modified the surface layer and the complete division of the road, separated by carriageway (ascending, descending, or the unique one in case of two-lane parts) are exposed for every road of Area 4 in each Road File of the Annex II under section “Skid resistance of surface layers in 2016)” Table 9.88 shows the stretches considered in each road (divided by single or double carriageway) according to the criteria commented in Fig. 9.4.

Table 9.88. Stretches considered in each road of Area 4 according to their surface layer project and traffic volume for skid resistance modelling (stretches from two-lane roads and double carriageway roads)

	Area 4	
Road	**Two-lane stretches**	**Double carriageway stretches**
A-8	-	117
BI-604	-	24
BI-628	6	16
BI-631	-	37
BI-636	17	32
BI-637	1	36
BI-644	-	10
BI-647	4	6
BI-738	-	-
N-633	-	13
N-634	64	32
N-637	-	58
N-644	-	4
Total	**92**	**385**
TOTAL	**477**	

The 477 new observations from Area 4 are added to the 451 from Areas 1, 2 and 3. From a total of 928 observations, there are 114 with a real age lower than 2 years. After discarding them, following previously commented criteria, a total 814 observations are analyzed. These data represent all the stretches of the road network of the Regional Government of Biscay with a surface layer coming from a different project (which implies variable age) and a different traffic volume (both AADT and H.AADT). The explanatory analysis of the variables is shown in Table 9.89. Normality tests show that any of the quantitative variables follow a normal distribution (Table 9.90).

Table 9.89. Explanatory analysis for the quantitative variables in all the stretches with known surface layer and a real age not lower than 2 years

		MSSC	AADT	H.AADT	Age	R.Age	TotVeh	TotH.Veh	PSV	Rain15
Mean		49,51	27109,59	809,94	8,53	8,54	88494	3252,04	49,09	28,32
Standard error		0,28	1147,03	33,28	0,21	0,21	5018	188,7	0,13	0,50
95% CI for mean	**Lower**	48,95	24858,10	744,62	8,12	78655,5	2882,04		48,84	27,34
	Upper	50,06	29361,08	875,26	8,93	98343,2	3622,82		49,34	29,29
Variance		65,99	1070960304	901324	34,924	34,92	20491922500	28983978	12,93	201,4
Std. deviation		8,12	32725,5	949,4	5,91	5,91	143153	5386,67	3,60	14,2
Minimum		31	654	10	2	2	93604	13,96	40	2,1
Maximum		85,88	139210	4872	36	36,5	1143417	48379	56	46,9
Range		54,88	138556	4862	34	34,5	1142481	48365	16	44,8
Interquartile range		9,81	28196	770	7	7	101331	3393,6	0	25,4
Skewness		0,802	1,728	1,871	1,461	1,474	3,143	3,514	-0,527	-0,732
Kurtosis		0,842	2,118	3,163	2,565	2,614	11,680	15,84	0,866	-0,95

Table 9.90. Normality tests for dependent and independent quantitative variables in all the stretches with known surface layer and a real age not lower than 2 years

Variables	Kolmogorov-Smirnov[a]			Shapiro-Wilk		
	Statistical	Degree of freedom	Significance (p-value	Statistical	Degree of freedom	Significance (p-value
MSSC	0,080	814	< 0,001	0,962	814	< 0,001
AADT	0,268	814	< 0,001	0,835	814	< 0,001
H.AADT	0,239	814	< 0,001	0,743	814	< 0,001
Age	0,216	814	< 0,001	0,750	814	< 0,001
R.Age	0,364	814	< 0,001	0,776	814	< 0,001
TotVeh	0,135	814	< 0,001	0,871	814	< 0,001
TotH.Veh	0,134	814	< 0,001	0,871	814	< 0,001
PSV	0,270	814	< 0,001	0,602	814	< 0,001
Rain 15	0,274	814	< 0,001	0,588	814	< 0,001
TotBit	0,239	814	< 0,001	0,742	814	< 0,001

[a] Lilliefors Significance correction

Correlation between the dependent variable and the independent quantitative variables is exposed in Table 9.91. As previous analysis, the best correlations can be found with *AADT* and *H.AADT* and PSV_{req}, although with *TotVeh* and *TotHVeh* also good correlations can be found. *Rain15*, *Age* and *R.Age* show very little correlations. As in other analysis, transformations of the variables have been studied. Firstly, the equations of the curve that best correlate data for every independent variable and the dependent variables (MSSC) was found (Table 9.92). Then, observing that *AADT* and *H.AADT* obtained the best correlation by means of the natural logarithm, these transformed variables were also studied to get the best curve to fit the data. The correlation of some transformed independent variables with MSSC was also investigated (Table 9.93).

Table 9.91. Correlation between the dependent variable and independent variables (coefficient of Pearson) in all the stretches with known surface layer and a real age not lower than 2 years

Independent Variables	Correlation with MSSC	Significance of the correlation (bilateral)
AADT	-0,343	< 0,001
H.AADT	-0,307	< 0,001
Age	-0,118	0,001
R.Age	-0,118	0,001
TotVeh	-0,263	< 0,001
TotH.Veh	-0,23	< 0,001
PSV	-0,35	< 0,001
Rain 15	-0,086	0,014

The qualitative independent variable (*SurfType*), which distinguishes between Asphalt Concrete (AC), Discontinuous and Porous Asphalt (BBTM&PA) and Slurries, was analyzed (Table 9.94). A new qualitative variable was created, named *RoadType*, which distinguishes between two-lane roads (2L) and double carriageway roads (2C), to check if this distinction has influence in the skid resistance modelling.

Table 9.92. Equations that best correlate each independent variable individually with the dependent variable in all the stretches with known surface layer and a real age not lower than 2 years

Independent Variable	Equation type	Resume of the model					Parameter estimates			
		R^2	F	Degrees of freedom 1	Degree of freedom 2	Sig.	Intercept	b1	b2	b3
AADT	Logarithm	0,330	400,031	1	812	< 0,001	84,332	-3,668	-	-
H.AADT	Logarithm	0,266	294,514	1	812	< 0,001	70,766	-3,511	-	-
Age	Linear	0,014	11,505	1	812	0,001	50,891	-0,162	-	-
R.Age	Linear	0,014	11,560	1	812	0,001	50,896	-0,163	-	-
TotVeh	Logarithm	0,310	364,041	1	812	< 0,001	102,727	-3,073	-	-
TotH.Veh	Logarithm	0,266	294,656	1	812	< 0,001	88,178	-2,766	-	-
PSV	Linear	0,122	113,339	1	812	< 0,001	88,324	-0,791	-	-
LnAADT	Inverse	0,367	470,866	1	812	< 0,001	15,003	321,457		
LnH.AADT	Inverse	0,300	348,229	1	812	< 0,001	30,009	113,003		

Table 9.93. Correlation between the dependent variable and some transformed independent variables (coefficient of Pearson)in all the stretches with known surface layer and a real age not lower than 2 years

Independent Variables	Correlation with MSSC	Significance of the correlation (bilateral)
LnAADT	-0,574	< 0,001
LnH.AADT	-0,516	< 0,001
1/LnAADT	0,606	< 0,001
1/LnH.AADT	0,548	< 0,001
(AADT)^1/2	-0,454	< 0,001
(H.AADT)^1/2	-0,407	< 0,001
LnAADT	-0,556	< 0,001
LnH.AADT	-0,516	< 0,001

Table 9.94. Exploratory analysis for the qualitative variables, grouped by levels, for the dependent variables (MSSC) in all the stretches with known surface layer and a real age not lower than 2 years

		RoadType		SurfType		
		2L	2C	AC (1)	BBTM&PA (2)	Slurry (3)
N		475	339	196	401	217
Mean		52,21	45,71	49,348	45,899	56,313
Standard error		0,40	0,27	0,595	0,249	0,564
95% CI for mean	**Lower**	51,42	45,18	48,174	45,410	55,201
	Upper	53,01	46,25	50,522	46,387	57,425
Variance		77,66	25,09	69,428	24,793	69,053
Std. deviation		8,81	5,01	8,332	4,979	8,310
Minimum		33	31	33,000	31,000	38,000
Maximum		85,88	64,04	73,318	63,495	85,875
Range		52,88	33,04	40,318	32,495	47,875
Interquartile range		12,46	6,51	10,718	6,356	10,703
Skewness		0,453	0,214	0,383	0,236	0,440
Kurtosis		0,116	0,786	-0,183	0,734	0,237

An Analysis of Variance was carried out for levels in *SurfType*. The Levene statistic (Table 9.95) indicates that there are not homogeneous variances in the levels (p < 0,001). The robust tests of equality of means and the ANOVA show that it is accepted that the means are different with more than a 95% of confidence (p < 0,05) (Table 9.96 and Table 9.97). Additionally, the Tamhane's T2 and Dunnett's T3 statistics show that there is not equality means between any two levels in SurfType (p < 0,05) (Table 9.98).

Table 9.95148. Test of homogeneity of variances for levels in SurfType in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC)

Levene Statistic	Degrees of Freedom 1	Degrees of Freedom 2	Significance
46,471	2	811	< 0,001

Table 9.96. Robust test of equality of means for levels in SurfType in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC).

Test	Statistic	Degrees of freedom 1	Degrees of freedom 2	Significance
Welch	146,218	2	355,846	< 0,001
Brown-Forsythe	131,787	2	506,852	< 0,001

Table 9.97. ANOVA for levels in SurfType levels in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC).

Variable (SurfType)	Sum of Squares	Degrees of freedom	Mean Square	F	Significance
Between groups	15278,619	2	7639,309	161,462	< 0,001
Within groups	38371,227	811	47,313		
Total	53649,846	813			

Table 9.98. Multiple comparison for SurfType levels in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC)

Statistic	SurfType (I)	SurfType (J)	Mean Differences (I-J)	Std Error	Sig	95% confidence interval Lower	95% confidence interval Upper
Tamhane's T2	AC	BBTM&PA	3,449	0,645	< 0,001	1,899	4,999
		Slurry	-6,965	0,820	< 0,001	-8,931	-4,999
	BBTM&PA	AC	-3,449	0,645	< 0,001	-4,999	-1,899
		Slurry	-10,415	0,616	< 0,001	-11,895	-8,934
	Slurry	AC	6,965	0,820	< 0,001	4,999	8,931
		BBTM&PA	10,415	0,616	< 0,001	8,934	11,895
Dunnett's T3	AC	BBTM&PA	3,449	0,645	< 0,001	1,900	4,999
		Slurry	-6,965	0,820	< 0,001	-8,931	-4,999
	BBTM&PA	AC	-3,449	0,645	< 0,001	-4,999	-1,900
		Slurry	-10,415	0,616	< 0,001	-11,895	-8,935
	Slurry	AC	6,965	0,820	< 0,001	4,999	8,931
		BBTM&PA	10,415	0,616	< 0,001	8,935	11,895

As there are more surface layer materials within each level of SurfType, the levels of SurfDen (exposed in Table 9.11), SurfDen was established as a new qualitative independent variable and was analyzed (Table

9.99). The Levene statistic for SurfDen levels shows that levels have not homogeneous variances. However, the test for the mean differences cannot be carried out because some levels have only one value. Consequently, it seems necessary to group some materials in “almost similar” denominations. Taking Table 9.11 as base, a new qualitative independent variable was created, SurfDen2, which gather some surface layer materials to have a representative quantity of data in each level of Asphalt Concrete (Table 9.100). It was decided to divide Asphalt Concrete mixes according to the maximum diameter of aggregates, 16 and 22 mm, and not taking into account the granulometry of the mixing (dense or semi-dense).

Table 9.99. Exploratory analysis for the qualitative variable SurfDen, grouped by levels, for the dependent variables (MSSC) in all the stretches with known surface layer and a real age not lower than 2 years

		SurfDen							
		AC 16 surf S	**AC 16 surf D**	**AC 22 surf D**	**AC 22 surf S**	**BBTM 11A**	**BBTM 11B**	**PA 11**	**LB2**
N		174	3	1	18	312	10	79	217
Mean		49,173	48,667		50,645	46,38	47,80	43,77	56,31
Standard error		0,627	2,228		2,262	0,28	1,30	0,50	0,56
95% CI for mean	**Lower**	47,936	39,080		45,873	45,82	44,86	42,78	55,20
	Upper	50,410	58,253		55,418	46,93	50,73	44,77	57,43
Variance		68,336	14,893		92,098	24,96	16,79	19,73	69,05
Std. deviation		8,267	3,859		9,597	5,00	4,10	4,44	8,31
Minimum		33,000	45,600		33,944	31,00	41,19	33,33	38,00
Maximum		73,318	53,000		64,625	63,49	54,67	53,75	85,88
Range		40,318	7,400		30,681	32,49	13,48	20,42	47,88
Interquartile range		10,486	.		13,621	6,45	6,63	5,87	10,70
Skewness		0,478	1,318		-0,307	0,26	-0,03	-0,01	0,44
Kurtosis		-0,043	.		-0,750	0,87	-0,41	-0,02	0,24

Table 9.100149. Qualitative independent variables for surface layer materials

Surface denomination (SurfDen)	Surface type (SurfType)	Surface denomination 2 (SurfDen2)
AC 16 surf S (1)	Asphalt concrete (AC) (1)	AC 16 (1)
AC 22 surf S (2)	Asphalt concrete (AC) (1)	AC 22 (2)
AC 16 surf D (3)	Asphalt concrete (AC) (1)	AC 16 (1)
AC 22 surf D (4)	Asphalt concrete (AC) (1)	AC 22 (2)
BBTM 11A (5)	Discontinuous and porous asphalt (BBTM&PA) (2)	BBTM 11A (3)
BBTM 11B (6)	Discontinuous and porous asphalt (BBTM&PA) (2)	BBTM 11B (4)
PA 11 (7)	Discontinuous and porous asphalt (BBTM&PA) (2)	PA (5)
LB2 (8)	Slurry (3)	LB2 (6)

The explanatory analysis of SurfDen2 levels is presented in Table 9.101. The Levene statistic (Table 9.102) shows that levels have not homogeneous variances ($p < 0{,}01$). Both the robust test of equality of means and the ANOVA indicate that there is not mean equality ($p < 0{,}001$) (Table 9.103 and 9.104). The Tamhane’s T2 statistic presents between which levels there is a mean difference (Table 9.105). Dunnett’s T3 was not included as provided similar values.

The Levene statistic for *RoadType* indicates that levels for this factor (two-lane and double carriageway) have not homogeneous variances and the t-test shows that there is mean difference (p < 0,001) (Table 9.106).

Table 9.101. Explanatory analysis for SurfDen2 in all the stretches with known surface layer and a real age not lower than 2 years, grouped by levels, for the dependent variables (MSSC).

		SurfDen2					
		AC 15	**AC 22**	**BBTM 11A**	**BBTM 11B**	**PA 11**	**LB2**
N		177	19	312	10	79	217
Mean		49,164	51,059	46,377	47,796	43,771	56,313
Standard error		0,617	2,179	0,283	1,296	0,500	0,564
95% CI for mean	Lower	47,947	45,820	45,820	44,864	42,776	55,201
	Upper	50,382	46,930	46,933	50,727	44,766	57,425
Variance		67,345	90,228	24,958	16,791	19,732	69,053
Std. deviation		8,206	9,499	4,996	4,098	4,442	8,310
Minimum		33,000	33,944	31,000	41,191	33,333	38,000
Maximum		73,318	64,625	63,495	54,667	53,750	85,875
Range		40,318	30,681	32,495	13,476	20,417	47,875
Interquartile range		10,401	13,196	6,449	6,633	5,872	10,703
Skewness		0,483	-0,408	0,264	-0,031	-0,013	0,440
Kurtosis		-0,004	-0,703	0,866	-0,410	-0,018	0,237

Table 9.102. Test of homogeneity of variances for levels in SurfDen2 in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC)

Levene Statistic	Degrees of Freedom 1	Degrees of Freedom 2	Significance
20,551	5	808	< 0,001

Table 9.103. Robust test of equality of means for levels in SurfDen2 in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC

Test	Statistic	Degrees of freedom 1	Degrees of freedom 2	Significance
Welch	62,438	5	65,784	< 0,001
Brown-Forsythe	65,511	5	117,717	< 0,001

Table 9.104150. ANOVA for levels in SurfDen2 levels in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC).

Variable (SurfType)	Sum of Squares	Degrees of freedom	Mean Square	F	Significance
Between groups	15805,269	5	3161,054	67,490	< 0,001
Within groups	37844,576	808	46,837		
Total	53649,846	813			

Table 9.105. Multiple comparison for levels SurfDen2 levels in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC)

Statistic	SurfType (I)	SurfType (J)	Mean Differences (I-J)	Std Error	Sig	95% confidence interval Lower	Upper
Tamhane's T2	AC 16	AC 22	-1,895	2,265	1,000	-9,368	5,579
		BBTM 11A	2,788	0,679	0,001	0,782	4,793
		BBTM 11B	1,369	1,435	0,999	-3,716	6,453
		PA 11	5,394	0,794	0,000	3,046	7,741
		LB2	-7,149	0,836	0,000	-9,612	-4,686
	AC 22	AC 16	1,895	2,265	1,000	-5,579	9,368
		BBTM 11A	4,682	2,197	0,512	-2,687	12,051
		BBTM 11B	3,263	2,535	0,970	-4,893	11,420
		PA 11	7,288	2,236	0,057	-0,138	14,715
		LB2	-5,254	2,251	0,365	-12,705	2,196
	BBTM 11A	AC 16	-2,788	0,679	0,001	-4,793	-0,782
		AC 22	-4,682	2,197	0,512	-12,051	2,687
		BBTM 11B	-1,419	1,326	0,996	-6,494	3,656
		PA 11	2,606	0,574	0,000	0,894	4,318
		LB2	-9,937	0,631	0,000	-11,798	-8,075
	BBTM 11B	AC 16	-1,369	1,435	0,999	-6,453	3,716
		AC 22	-3,263	2,535	0,970	-11,420	4,893
		BBTM 11A	1,419	1,326	0,996	-3,656	6,494
		PA 11	4,025	1,389	0,185	-1,040	9,090
		LB2	-8,518	1,413	0,001	-13,589	-3,446
	PA 11	AC 16	-5,394	0,794	0,000	-7,741	-3,046
		AC 22	-7,288	2,236	0,057	-14,715	0,138
		BBTM 11A	-2,606	0,574	0,000	-4,318	-0,894
		BBTM 11B	-4,025	1,389	0,185	-9,090	1,040
		LB2	-12,543	0,754	0,000	-14,770	-10,315
	LB2	AC 16	7,149	0,836	0,000	4,686	9,612
		AC 22	5,254	2,251	0,365	-2,196	12,705
		BBTM 11A	9,937	0,631	0,000	8,075	11,798
		BBTM 11B	8,518	1,413	0,001	3,446	13,589
		PA 11	12,543	0,754	0,000	10,315	14,770

Table 9.106. Levene statistic for homogeneous variances and t-test for equality of means for levels of RoadType for MSSC in all the stretches with known surface layer and a real age not lower than 2 years

Levene statistic		t-test for equality of means							
F	Sig.	Mean difference	t	Degrees of freedom	Significance (bilateral)	Mean difference	Standard error	Interval of 95% of confidence for the difference Lower	Upper
100,53	< 0,001	Equal variances assumed	12,243	812,000	< 0,001	6,501	0,531	5,459	7,543
		Equal variances not assumed	13,340	776,982	< 0,001	6,501	0,487	5,544	7,458

With previous experiences in friction modelling, some General Linear Models were tried, with different combinations of covariables (quantitative variables) and factors (qualitative variables: *SurfType*, *SurfDen2* and *RoadType*). Table 9.107 shows the obtained R^2 and adjusted R^2 for each model and some comments about it. This table summaries a complete analysis that was carried out.

As observed, the best correlations with all the variables statistically significant (95% of confidence level; p <

0,05), include 1/Ln*AADT* and PSV_{req} and *SurfType* (Eq. 9.8), or *SurfDen2* (Eq 9.9), with a R^2 of 0,488 and 0,498, respectively. When introducing *SurfDen* as a factor with *PSV*(f) and 1/Ln*AADT*, although improving the R^2, PSV_{req}(f) has low significance.

$$MSSC = Intercept + 1/LnAADT + PSV_{req}(f) + SurfType(f) + PSV * SurfType \quad [9.8]$$

$$MSSC = Intercept + 1/LnAADT + PSV_{req}(f) + SurfDen2(f) + PSV * SurfDen2 \quad [9.9]$$

Table 9.107. Proposed General Linear Models in all the stretches with known surface layers.

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + LnAADT + SurfType(f)	0,423	0,421	All variables have a p-value < 0,001
MSSC = Int + LnAADT + SurfType(f) + RoadType(f) + SurfType * RoadType	0,439	0,435	Only the product SurfType*RoadType has low significance (p = 0,387)
MSSC = Int + 1/LnAADT +1/LnH.AADT + SurfType(f)	0,462	0,460	1/LnH.AADT has low significance (p = 0,81)
MSSC = Int + 1/LnAADT +1/LnH.AADT + SurfType(f) + RoadType(f) + SurfType * RoadType	0,479	0,474	Medium significance of 1/LnH.AADT (p = 0,191) and SurfType * RoadType (p = 0,181
MSSC = Int + 1/LnAADT + SurfType(f)	0,462	0,460	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfType(f) + RoadType(f) + SurfType * RoadType	0,478	0,474	Medium significance of SurfType*RoadType (p=0,128)
MSSC = Int + 1/LnAADT + PSV(f) + SurfType(f) + PSV*SurfType	0,488	0,481	All variables have a p-value < 0,02
MSSC = Int + 1/LnAADT + PSV(f) + SurfType(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,506	0,495	Only the products of factors including PSV show medium significance (p≈0,12).
MSSC = Int + 1/LnAADT + 1/LnH.AADT + PSV(f) + SurfType(f) + PSV*SurfType	0,489	0,480	1/LnH.AADT show low significance (p=0,735)
MSSC = Int + 1/LnH.AADT + PSV(f) + SurfType(f) + PSV*SurfType	0,442	0,434	All variables have a p-value < 0,001
MSSC = Int + 1/LnH.AADT + PSV(f) + SurfType(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,454	0,442	Low significance of RoadType (P=0,447) and products including PSV (P >0,138)
MSSC = Int + 1/LnAADT + 1/LnH.AADT + PSV(f) + SurfDen(f) + PSV*SurfDen	0,501	0,486	Low significance of 1/LnH.AADT (p=0,880) and PSV (0,282)
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen(f) + PSV*SurfDen	0,502	0,488	Low significance of PSV(f) (0,281)
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,515	0,495	Low significance of PSV (p=0,089) and all the products of factors (p>0,156).
MSSC = Int + 1/LnAADT + 1/LnH.AADT + PSV + SurfDen2(f)	0,469	0,463	Low significance of 1/LnH.AADT (p=0,826) and PSV (p=0,828)
MSSC = Int + 1/LnAADT + 1/LnH.AADT + SurfDen2(f)	0,468	0,464	Low significance of 1/LnH.AADT (p=0,851)
MSSC = Int + 1/LnAADT + SurfDen2(f)	0,468	0,465	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfDen2(f) + RoadType(f) + SurfDen2*RoadType	0,488	0,480	SurfDen2*RoadType have a p-value = 0,158
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen2(f) + PSV*SurfDen2	0,498	0,486	All variables have a p-value < 0,03
MSSC = Int + 1/LnAADT + PSV(f) + SurfDen(f) + RoadType(f) + PSV*SurfType + PSV*RoadType + SurfType*RoadType + PSV*SurfType*RoadType	0,513	0,495	Products of factor have low significance (p>0,10)
MSSC = Int + LnAADT + PSV(f) + SurfDen2(f) + PSV*SurfDen2	0,476	0,463	All variables have a p-value < 0,01

When *RoadType* is incorporated as a factor to Eq. 9.8 and Eq. 9.9, the R^2 and the adjusted R^2 improve considerably (from 0,488 to 0,506 in Eq. 9.8 and from 0,498 to 0,513 in Eq. 9.9), but the products of the factors have low significance.

Observing that the combinations of the factors have normally low significance, it is decided to check models that do not include all the combination of factors and even, the combination of factors with covariables is tried. The software of SPSS v.24 allows choosing all the introduced variables in the General Linear Model, combining the factors as wanted and combining factors and covariables.

Additionally, it can be observed that the qualitative variable *RoadTyp* really influences the predicted values and it is included in models with highest R^2. As explained in 8.4.1, the variable *H.AADT* reflects the variable quantity of lanes can exist in a double carriageway. However, since *AADT* has been predominantly more correlated to *MSSC* than *H.AADT*, it is not the same to have in a freeway an AADT of 50.000 vehicle/day in a two-lane carriageway or in three-lane carriageway. The quantity of vehicles in each lane is completely different. Therefore, it seems necessary to include a new variable to reflect the quantity of lanes in each carriageway of a freeway. It is called *Lanes*, and its levels are shown in Table 9.108.

The levels of the new qualitative variables Lanes were analysed. The Levene statistic (Table 9.109) shows that there are not homogeneous variances in the levels ($p < 0{,}001$). There is not mean equality as the robust test and the ANOVA indicate with more than a 95% of confidence (Table 9.110 and 9.111). Moreover, the Thamhane's T2 and Dunnett's T3 statistics indicate that level "two-lane road" has not mean equality with any of the other two groups but there is mean equality between levels of 2 and 3 lanes in a double carriageway road (Table 9.112).

Table 9.108. Types of roads and its classification according to the qualitative variables RoadTypes and Lanes

Type of road	Levels of variable *RoadType*	Levels of variable *Lanes*
Unique carriageway, two-lane road	1	1
Double carriageway, two lanes per carriageway in each direction	2	2
Double carriageway, three or four lanes per carriageway in each direction*	2	3

* In the traffic data of the RGB, sections with 3 or 4 lanes per carriageway are indicated similarly and hence, they cannot be distinguished

Table 9.109. Test of homogeneity of variances for levels in Lanes in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC)

Levene Statistic	Degrees of Freedom 1	Degrees of Freedom 2	Significance
58,709	2	793	< 0,001

Table 9.151110. Robust test of equality of means for levels in Lanes in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC).

Test	Statistic	Degrees of freedom 1	Degrees of freedom 2	Significance
Welch	96,963	2	235,290	< 0,001
Brown-Forsythe	140,256	2	621,954	< 0,001

Table 9.111. ANOVA for levels in Lanes levels in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC).

Variable (Lanes)	Sum of Squares	Degrees of freedom	Mean Square	F	Significance
Between groups	8438,469	2	4219,234	79,425	< 0,001
Within groups	42126,131	793	53,122		
Total	50564,600	795			

With the new possible qualitative variable, *Lanes*, and the multiple possible combinations of factors and factors with covariables, a complete analysis was conducted. Models with better coefficient of determination, R^2, and with most of the variables statistically significant are shown in Tables 9.113 to 9.115. They only represent a summary of a more complete analysis. Table 9.113 shows the models that included *SurfType* as a factor for the surface material, Table 9.114 included *SurfDen* and models in Table 9.115 included *SurfDen2.*

Table 9.112.. Multiple comparison for Lanes levels in all the stretches with known surface layer and a real age not lower than 2 years, for the dependent variable (MSSC)

Statistic	SurfType (I)	SurfType (J)	Mean Differences (I-J)	Std Error	Sig	95% confidence interval	
						Lower	Upper
Tamhane's T2	Two-lane road	Double with 2 lanes	6,742	0,498	< 0,001	5,549	7,935
		Double with 3 lanes	6,133	0,618	< 0,001	4,644	7,621
	Double with 2 lanes	Two-lane road	-6,742	0,498	< 0,001	-7,935	-5,549
		Double with 3 lanes	-0,610	0,557	0,620	-1,957	0,738
	Double with 3 lanes	Two-lane road	-6,133	0,618	< 0,001	-7,621	-4,644
		Double with 2 lanes	0,610	0,557	0,620	-0,738	1,957
Dunnett's T3	Two-lane road	Double with 2 lanes	6,742	0,498	< 0,001	5,549	7,935
		Double with 3 lanes	6,133	0,618	< 0,001	4,644	7,621
	Double with 2 lanes	Two-lane road	-6,742	0,498	< 0,001	-7,935	-5,549
		Double with 3 lanes	-0,610	0,557	0,618	-1,957	0,737
	Double with 3 lanes	Two-lane road	-6,133	0,618	< 0,001	-7,621	-4,644
		Double with 2 lanes	0,610	0,557	0,618	-0,737	1,957

Observing the plots for the independent variables which best correlate the dependent variable, AADT and H.AADT vs. the dependent variables (MSSC) and calculating the distances of Malahanobis and Cook, possible outliers were identified. They were analyzed to check if they could represent an outlier far from the curve that best fit, producing leverage. After analyzing them, some cases were removed. They represent cases very distant from the expected value and the reason in most of the cases was that the observed mean value of the SCRIM Coefficient came from a short stretch with only 2 or 3 values of SC. These cases have considered as non representative and have been removed. The cases that have been removed are listed in Table 9.116, indicating the reason for being removed.

Table 9.113. Proposed General Linear Models in all the stretches with known surface layer and a real age not lower than 2 years, with only some combination of factors and combining factors and covariables, with SurfType as the factor for representing the surface layer material

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + 1/LnAADT + SurfType(f) + RoadType(f)	0,475	0,420	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfType(f) + RoadType(f) + RoadType*1/LnAADT	0,505	0,502	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f)	0,493	0,490	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + SurfType*Lanes	0,496	0,491	SurfType*Lanes is not significant (p=0,249)
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + 1/LnAADT*Lanes	0,510	0,506	All variables have a p-value < 0,01
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + RoadType(f)+SurfType*RoadType	0,493	0,488	All variables have a p-value < 0,02
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + RoadType(f)+SurfType*RoadType*1/LnAADT	0,516	0,508	PSV has low significance (p=0,629)
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + RoadType(f) + Lanes(f) + Lanes*1/LnAADT	0,511	0,504	PSV has low significance (p=0,827)
MSSC = Int + 1/LnAADT + SurfType(f) + PSV(f) + Lanes(f) + Lanes*1/LnAADT + SurfType*PSV	0,519	0,509	PSV has low significance (p=0,102)
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + Lanes*1/LnAADT + SurfType*PSV	0,519	0,509	PSV*SurfType has medium significance (p=0,104)
MSSC = Int + LnAADT + SurfType(f) + Lanes(f) + Lanes*LnAADT + SurfType*PSV	0,520	0,510	PSV*SurfType has medium significance (p=0,121)

Table 9.114152. Proposed General Linear Models in all the stretches with known surface layer and a real age not lower than 2 years, with only some combination of factors and combining factors and covariables, with SurfDen as the factor for representing the surface layer material

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + 1/LnAADT + SurfDen(f) + SurfDen*PSV	0,502	0,488	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + SurfDen*PSV	0,509	0,494	All variables have a p-value < 0,01
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + SurfDen*PSV + RoadTyoe*1/LnAADT	0,529	0,514	PSV*SurfDen has low significance (p=0,215)
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + RoadTyoe*1/LnAADT	0,518	0,512	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f)	0,509	0,500	PSV has medium significance (p=0,107)
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f) + SurfDen*PSV	0,520	0,504	PSV has low significance (p=0,538)
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f) + SurfDen*PSV + Lanes*1/LnAADT	0,533	0,517	PSV has low significance (p=0,339) and PSV*Surf*Den medium significance (p=0,110)
MSSC = Int + 1/LnAADT + SurfDen(f) + Lanes(f) + Lanes*1/LnAADT	0,423	0,516	All variables have a p-value < 0,005
MSSC = Int + LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT	0,525	0,518	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT	0,520	0,518	1/LnAADT has low significance (p=0,214)
MSSC = Int + LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT + PSV(f) + PSV*SurfDen	0,534	0,518	PSV has low significance (p=0,433) and PSV*Surf*Den medium significance (p=0,170)

Table 9.115. Proposed General Linear Models in all the stretches with known surface layer and a real age not lower than 2 years, with only some combination of factors and combining factors and covariables, with SurfDen2 as the factor for representing the surface layer material

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f)	0,494	0,487	All variables have a p-value < 0,03
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f) + PSV*SurfDen2	0,507	0,493	All variables have a p-value < 0,06
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f) + RoadType*1/LnAADT	0,515	0,507	PSV has low significance (p=0,884)
MSSC = Int + 1/LnAADT + SurfDen2(f) + PSV(f) + RoadType(f) + PSV*SurfDen2 + RoadType*1/LnAADT	0,526	0,513	PSV has low significance (p=0,264)
MSSC = Int + 1/LnAADT + SurfDen2(f) + RoadType(f) + PSV*SurfDen2 + RoadType*1/LnAADT	0,526	0,513	PSV*SurfDen2 has lowmedium significance (p=0,134)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV(f)	0,506	0,480	PSV has medium significance (p=0,099)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV(f) + PSV*SurfDen2	0,517	0,503	PSV has low significance (p=0,277)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2	0,517	0,503	All variables have a p-value < 0,03
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV(f) + Lanes*1/LnAADT	0,520	0,511	PSV has low significance (p=0,937)
MSSC = Int + 1/LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2 + Lanes*1/LnAADT	0,530	0,516	PSV*SurfDen2 has low significance (p=0,209)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2 + Lanes*LnAADT	0,531	0,517	PSV*SurfDen2 has low significance (p=0,262)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2*PSV*LnAADT	0,556	0,531	All variables have a p-value < 0,01
MSSC = Int + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2 + PSV*SurfDen2*PSV*LnAADT	0,572	0,541	Intersection has low significance (p=0,301)

After removing the observations that are considered as spurious (18), 796 observations remain for the analysis. With them, the equations from Table 9.113 to 9.115 with the best R^2 have been recalculated in Table 9.117.

As observed in Table 9.117, the models show an improvement in the coefficient of determination, R^2, although in some cases, the significance of some variables (or product of factors) gets worse. Once the outliers have been removed, it has continued the analysis of possible models for explaining the available friction. Table 9.118 shows a summary of the complete examination of models that was conducted. It has been preferred models with all the variables statistical significant (p-value < 0,05) than models with high R^2 or adjusted R^2.

Table 9.116. List of observation that were removed as considered as outliers

Road (direction)	Initial KP - Final KP	MSSC	AADT	H.AADT	Data of construction	Surface	Explanation
A-8 (desc)	131+0599 - 130+0969	54,3	70821	2498	30/05/2001	LB2	Many year with a high traffic. It must be rehabilitated. Unique stretch with slurry in the most important motorway
BI-604 (asc)	6+0796 - 6+0956	34,5	14853	584	26/02/1995	PA 11	Low value for not so high AADT.
N-634 (asc)	97+0679 - 97+0738	33,3	20610	773	31/08/1994	PA 11	Few observation for the main
BI-631 (asc)	6+0740 - 7+0009	34,3	29350	455	01/06/1997	PA 11	19 years old, it must be have changed
BI-631 (desc)	1+0260 - 0+0941	35,59	42408	552	01/03/2007	PA 11	Very low value
N-637 desc	28+0140 - 28+0061	53,75	30777	1600	01/06/2007	PA 11	Only 4 stretches of 20m for the mean
BI-637 (asc)	17+0210 - 18+0369	37,86	35342	389	21/08/2006	PA 11	Few observation for the mean
BI-637	18+0370 - 18+0669	38,06	35342	389	21/08/2006	PA 11	Distant from the mean
N-637 (desc)	10+0660 - 9+0671	53,1	137109	4079	01/06/1997	BBTM 11A	High value for the AADT and H.AADT
BI-637 (desc)	9+0157 - 8+0978	50,75	120758	2174	19/09/2013	BBTM 11A	High value for the AADT and H.AADT
BI-628 asc	16+0400 - 16+0479	32,50	1300	335	01/11/2005	BBTM 11A	Few observation for the mean
BI-631 (asc)	1+0260 - 1+0439	31,00	42408	552	01/03/2007	BBTM 11A	Very low mean
A-8 (asc)	108+0203 - 108+0722	61,15	56127	2385	01/06/2014	BBTM 11A	Perhaps it is not 2 years old
A-8 (asc)	1030+0890 - 131+0000	58,73	76821	2498	20/11/1987	AC 22 surf S	It must have been maintained after 29 years
BI-638	8+0880 - 8+0899	33	6532	245	01/06/2013	AC 16 surf S	It comes from a single 20 m-stretch
N-639	21+0300 - 21+0359	36	4392	209	01/06/2012	AC 16 surf S	Only 3 stretches of 20m for the mean
BI-625	377+0370 - 378+0029	33,95	12510	500	01/06/2013	AC 22 surf S	Very low value
N-240	55+0188 - 55+0207	33,18	29910	1361	01/06/2009	AC 16 surf S	Only 2 stretches of 20m for the mean

Table 9.117. Recalculation of the best General Linear Models of Table 9.113 to 9.115 without spurious data indicated in Table 9.116.

Proposed model	R^2	Adj R^2	Comments and observations
MSSC = Int + 1/LnAADT + SurfType(f) + Lanes(f) + 1/LnAADT*Lanes	0,523	0,518	All variables have a p-value < 0,004
MSSC = Int + LnAADT + SurfType(f) + Lanes(f) + Lanes*LnAADT + SurfType*PSV	0,528	0,519	PSV*SurfType has low significance (p=0,735)
MSSC = Int + 1/LnAADT + SurfDen(f) + RoadType(f) + SurfDen*PSV + RoadTyoe*1/LnAADT	0,529	0,523	All variables have a p-value < 0,001
MSSC = Int + 1/LnAADT + SurfDen(f) + PSV(f) + Lanes(f) + SurfDen*PSV + Lanes*1/LnAADT	0,539	0,522	PSV has low significance (p=0,472) and PSV*Surf*Den medium significance (p=0,536)
MSSC = Int + LnAADT + SurfDen(f) + Lanes(f) + Lanes*LnAADT + PSV(f) + PSV*SurfDen	0,540	0,524	PSV has low significance (p=0,589) and PSV*Surf*Den medium significance (p=0,529)
MSSC = Int + 1/LnAADT + SurfDen2(f) + RoadType(f) + PSV*SurfDen2 + RoadType*1/LnAADT	0,533	0,520	PSV*SurfDen2 has low significance (p=0,602)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + PSV*SurfDen2 + Lanes*LnAADT	0,537	0,522	PSV*SurfDen2 has low significance (p=0,729)
MSSC = Int + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2 + PSV*SurfDen2*PSV*LnAADT	0,577	0,546	Intersection has low significance (p=0,310)

Table 9.153118. Proposed General Linear Models in all the stretches with known surface layer and a real age not lower than 2 years, with only some combination of factors and combining factors and covariables, after removing the spurious data.

Proposed model	**R^2**	**Adj R^2**	**Comments and observations**
MSSC = Int + LnAADT + SurfType(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2*Lanes*LnAADT	0,555	0,532	All variables have a p-value < 0,03
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2	0,537	0,522	PSV*SurfDen2 has low significance (p=0,729)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes*LnAADT + PSV*SurfDen2*LnAADT	0,542	0,528	PSV*SurfDen2*LnAADT has medium significance (p=0,117)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfDen2*LnAADT	0,546	0,528	PSV*SurfDen2*LnAADT has medium significance (p=0,139)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfType*LnAADT	0,542	0,430	All variables have a p-value < 0,05
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + PSV*SurfType*LnAADT*Lanes	0,554	0,537	All variables have a p-value < 0,01
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + SurfType*LnAADT*Lanes	0,536	0,530	All variables have a p-value < 0,01
MSSC = Int + LnAADT + SurfDen2(f) + PSV(f) + Lanes*LnAADT	0,503	0,495	All variables have a p-value < 0,01
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + PSV(f) + Lanes*LnAADT	0,531	0,522	PSV has low significance (p=0,881)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT	0,530	0,524	All variables have a p-value < 0,01
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + SurfDen2*PSV*LnAADT	0,540	0,528	SurfDen2*PSV*LnAADT has medium significance (p=0,139)
MSSC = Int + LnAADT + SurfDen2(f) + Lanes(f) + Lanes*LnAADT + SurfDen2*PSV*LnAADT*Lanes	0,560	0,534	All variables have a p-value < 0,04

Observing the models of Table 9.118 and Tables 9.113 **-** 9.115, some conclusions can be extracted:

- The quantitative variables that produce the best models are Ln*AADT* and 1/Ln*AAD*T and from the qualitative variables *PSV*, *RoadType*, *Lanes*, *SurfType* and *SurfDen2*. The qualitative variable *SufDen*, which could be expected to be the most influencing factor, since it determines all the possible surface layer material, is not the one that best correlate, because there are some cases with few observations, such as AC 22 surf D (1 observation) and AC 16 surf D (3). Consequently, it seems reasonable to group surface materials with similar properties and characteristics, as in the levels of *SurfType* and *SurfDen2*.
- *H.AADT* produces worse models than *AADT* (or their transformations). If both quantitative variables are introduced, *H.AADT* usually becomes not statistically significant.
- The inclusion of combinations of factors generally let to better models than if they were not combined.
- *PSV* as a single factor is usually not statistically significant and if removed, the value of R^2 remains and the adjusted R^2 improves as it has a fewer variables in the model.
- The variable *Lanes* as a factor produces better models than *RoadType* as a factor.
- The introduction of 1/Ln*AADT***Lanes* or Ln*AADT***Lanes* always improve the result and gets a better value than introducing 1/Ln*AADT***RoadType* or Ln*AADT***RoadType*. Therefore, it is remarked the importance of the variable *Lanes*, which better describes the presence of 3 (or 4) lanes in the double carriageway. It is a better factor than *RoadType*.

- Although Ln*AADT* or 1/Ln*AADT* is introduced as covariable and 1/Ln*AADT***Lanes* or Ln*AADT***Lanes* as a combined variable, the additional inclusion of *Lanes* as a factor improves the model and is always significant.
- If 1/Ln*AADT* as covariable and the combination of *Lanes**Ln*AADT* are introduced (or alternatively Ln*AADT* and Lanes*1/Ln*AADT*); the covariable becomes insignificant. Ln*AADT* or 1/Ln*AADT* must be employed in both variables not to become insignificant.
- The combination of *PSV* as a factor with other factors such as *SurfType* of *SurfDen2* makes the combination not significant. If *Lanes* is also combined the significance is improved but usually not better than a *p-value* < 0,05. If 3 factors are combined with Ln*AADT* the models gets better.

With these conclusions and with the R^2 of the long list of examined models, it was decided to propose 2 models as the ones that best represent the friction available in the road network of Biscay:

- A short model, with a covariable and individual factors and only a combination of two variables not to have a long list of coefficients to be applied, in the form of Eq. 9.10, obtaining a R^2 = 0,530, an adjusted R^2 = 0,524 and all the variables significant (p-value < 0,001)

$$MSSC = Intercept + LnAADT + SurfDen2(f) + Lanes(f) + Lanes(f) * LnAADT \qquad [9.10]$$

- A long model, with variables included in Eq. 9.10 and the combination of 3 factor and a covariable to get a better result, R^2 = 0,560, adjusted R^2 = 0,534 and all the variables significant with a 95% of confidence (p-value < 0,04) (Eq. 9.11).

$$\begin{aligned} MSSC = {} & Intercept + LnAADT + SurfDen2(f) + Lanes(f) + Lanes(f) * LnAADT \\ & + SurfDen2(f) * PSV(f) + Lanes(f) * LnAADT \end{aligned} \qquad [9.11]$$

Although a better short model and a better long model could be obtained individually, it was preferred to use the same variables in both equations. The long model was wanted to have only additional variables to the short model. The only difference between both proposed equations is the last component of Eq. 9.11, which is a combination of 3 factors and the covariable.

Table 9.119 shows the test of Between-Subjects effects for the model of Eq. 9.10, where it can be observed that all the variables are significant (p < 0,001). Table 9.120 presents the estimations of the parameters (coefficients) of the model of Eq. 9.10. The matrixes L of constrast coefficient that allow obtaining the coefficients associated to each effect (coefficients that define the hypothesis presents in the model) are shown in Fig 9.5 and Tables 9.121 and 9.122. Tables 9.123 and 9.124 show the marginal mean of the dependent variable for each level of factor

Table 9.119. Test of Between-Subjects effects for model of Eq. 9.10, proposed short model.

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	26815,861	10	2681,586	88,638	< 0,001	0,53	886,382	1,000
Intercept	3333,343	1	3333,343	110,182	< 0,001	0,123	110,182	1,000
LnAADT	631,574	1	631,574	20,876	< 0,001	0,026	20,876	0,995
Lanes	1950,421	2	975,211	32,235	< 0,001	0,076	64,47	1,000
Lanes*LnAADT	2315,593	2	1157,797	38,27	< 0,001	0,089	76,541	1,000
SurfDen2	5284,665	5	1056,933	34,936	< 0,001	0,182	174,681	1,000
Error	23748,739	785	30,253					
Total	2015919,12	796						
Corrected total	50564,6	795						

Table 9.154120. Parameter estimates for model of Eq. 9.10, short model

Parameters	B	Std. Error	t	Sig.	95% CI		Partial eta-squared	Non centrality parameter	Observed Power
					Lower	U`pper			
Intercept	78,497	21,451	3,659	< 0,001	36,389	120,606	0,017	3,659	0,955
LnAADT	-2,321	1,894	-1,226	0,221	-6,038	1,396	0,002	1,226	0,232
[Lanes=1]	26,693	21,607	1,235	0,217	-15,722	69,107	0,002	1,235	0,235
[Lanes=2]	-16,663	22,030	-,756	0,450	-59,907	26,581	0,001	0,756	0,118
[Lanes=3]	0[a]	.	.	.	.	.	.	.	.
[Lanes=1] * LnAADT	-3,352	1,915	-1,750	0,081	-7,112	,408	0,004	1,750	0,416
[Lanes=2] * LnAADT	1,328	1,955	,679	0,497	-2,510	5,166	0,001	0,679	0,104
[Lanes=3] * LnAADT	0[a]	.	.	.	.	.	.	.	.
[SurfDen2=1]	-6,989	0,562	-12,433	< 0,001	-8,093	-5,886	0,165	12,433	1,000
[SurfDen2=2]	-5,047	1,391	-3,628	< 0,001	-7,778	-2,316	0,016	3,628	0,952
[SurfDen2=3]	-5,468	0,680	-8,046	< 0,001	-6,802	-4,134	0,076	8,046	1,000
[SurfDen2=4]	-4,213	1,857	-2,269	0,024	-7,859	-,568	0,007	2,269	0,620
[SurfDen2=5]	-7,467	0,959	-7,789	< 0,001	-9,349	-5,586	0,072	7,789	1,000
[SurfDen2=6]	0[a]	.	.	.	.	.	.	.	.

[a] Set to zero because this parameter is redundant.

Intercept		LnAADT		Lanes			Lanes*LnAADT			SurfDen2					
Parameter	L1	Parameter	L2	Parameter	L3	L4	Parameter	L6	L7	Parameter	L9	L10	L11	L12	L13
Intercept	1	Intercept	0	Intercept	0	0	Intercept	0	0	Intercept	0	0	0	0	0
LnAADT	0	LnAADT	1	LnAADT	0	0	LnAADT	0	0	LnAADT	0	0	0	0	0
[Lanes=1]	0,333	[Lanes=1]	0	[Lanes=1]	1	0	[Lanes=1]	0	0	[Lanes=1]	0	0	0	0	0
[Lanes=2]	0,333	[Lanes=2]	0	[Lanes=2]	0	1	[Lanes=2]	0	0	[Lanes=2]	0	0	0	0	0
[Lanes=3]	0,333	[Lanes=3]	0	[Lanes=3]	-1	-1	[Lanes=3]	0	0	[Lanes=3]	0	0	0	0	0
[Lanes=1]*LnAADT	0	[Lanes=1]*LnAADT	0,333	[Lanes=1]*LnAADT	0	0	[Lanes=1]*LnAADT	1	0	[Lanes=1]*LnAADT	0	0	0	0	0
[Lanes=2]*LnAADT	0	[Lanes=2]*LnAADT	0,333	[Lanes=2]*LnAADT	0	0	[Lanes=2]*LnAADT	0	1	[Lanes=2]*LnAADT	0	0	0	0	0
[Lanes=3]*LnAADT	0	[Lanes=3]*LnAADT	0,333	[Lanes=3]*LnAADT	0	0	[Lanes=3]*LnAADT	-1	-1	[Lanes=3]*LnAADT	0	0	0	0	0
[SurfDen2=1]	0,167	[SurfDen2=1]	0	[SurfDen2=1]	0	0	[SurfDen2=1]	0	0	[SurfDen2=1]	1	0	0	0	0
[SurfDen2=2]	0,167	[SurfDen2=2]	0	[SurfDen2=2]	0	0	[SurfDen2=2]	0	0	[SurfDen2=2]	0	1	0	0	0
[SurfDen2=3]	0,167	[SurfDen2=3]	0	[SurfDen2=3]	0	0	[SurfDen2=3]	0	0	[SurfDen2=3]	0	0	1	0	0
[SurfDen2=4]	0,167	[SurfDen2=4]	0	[SurfDen2=4]	0	0	[SurfDen2=4]	0	0	[SurfDen2=4]	0	0	0	1	0
[SurfDen2=5]	0,167	[SurfDen2=5]	0	[SurfDen2=5]	0	0	[SurfDen2=5]	0	0	[SurfDen2=5]	0	0	0	0	1
[SurfDen2=6]	0,167	[SurfDen2=6]	0	[SurfDen2=6]	0	0	[SurfDen2=6]	0	0	[SurfDen2=6]	-1	-1	-1	-1	-1

Fig. 9.5. L matrix of the model of Eq. 9.10, for obtaining the coefficients associated to each effect.

Table 9.121. Matrix L' of contrast coefficient that allow obtaining the associated coefficients for each effect.

Parameter	**Lanes**		
	Two lanes	**Double carriageway with 2 lanes**	**Double carriageway with 3 lanes**
Intercept	1	1	1
LnAADT	9,477	9,477	9,477
[Lanes=1]	1	0	0
[Lanes=2]	0	1	0
[Lanes=3]	0	0	1
[Lanes=1] * LnAADT	9,477	0	0
[Lanes=2] * LnAADT	0	9,477	0
[Lanes=3] * LnAADT	0	0	9,477
[SurfDen2=1]	0,167	0,167	0,167
[SurfDen2=2]	0,167	0,167	0,167
[SurfDen2=3]	0,167	0,167	0,167
[SurfDen2=4]	0,167	0,167	0,167
[SurfDen2=5]	0,167	0,167	0,167
[SurfDen2=6]	0,167	0,167	0,167

Table 9.122. Matrix L' of contrast coefficients that allow obtaining the associated coefficients for each effect.

Parameter	SurfDen2					
	AC 16	AC 22	BBTM 11A	BBTM 11B	PA 11	LB2
Intercept	1	1	1	1	1	1
LnAADT	9,477	9,477	9,477	9,477	9,477	9,477
[Lanes=1]	0,333	0,333	0,333	0,333	0,333	0,333
[Lanes=2]	0,333	0,333	0,333	0,333	0,333	0,333
[Lanes=3]	0,333	0,333	0,333	0,333	0,333	0,333
[Lanes=1] * LnAADT	3,159	3,159	3,159	3,159	3,159	3,159
[Lanes=2] * LnAADT	3,159	3,159	3,159	3,159	3,159	3,159
[Lanes=3] * LnAADT	3,159	3,159	3,159	3,159	3,159	3,159
[SurfDen2=1]	1	0	0	0	0	0
[SurfDen2=2]	0	1	0	0	0	0
[SurfDen2=3]	0	0	1	0	0	0
[SurfDen2=4]	0	0	0	1	0	0
[SurfDen2=5]	0	0	0	0	1	0
[SurfDen2=6]	0	0	0	0	0	1

Table 9.123. Estimated marginal means of the dependent variable (MSSC) for the levels of the factor Lanes

Dependent variable: MSSC				
Lanes	Mean	Standard error	95% Confidence interval	
			Lower bound	Upper bound
Two lanes	46,565[a]	0,518	45,548	47,582
Double carriageway with 2 lanes	47,565[a]	0,660	46,269	48,861
Double carriageway with 3 lanes	51,639[a]	3,549	44,672	58,607

a. The covariables that appear in the model are evaluated in the following values: LnAADT = 9,4774.

Table 9.124. Estimated marginal means of the dependent variable (MSSC) for the levels of the factor SurfDen2

Dependent variable: MSSC				
SurfDen2	Mean	Standard error	95% Confidence interval	
			Lower bound	Upper bound
AC 16	46,465[a]	1,314	43,884	49,045
AC 22	48,407[a]	1,790	44,894	51,920
BBTM 11A	47,986[a]	1,238	45,555	50,416
BBTM 11B	49,241[a]	2,107	45,104	53,377
PA 11	45,987[a]	1,254	43,525	48,448
LB2	53,454[a]	1,311	50,881	56,027

a. The covariables that appear in the model are evaluated in the following values: LnAADT = 9,4774.

Fig. 9.6a and 9.6b present the diagrams of dispersion by level and provide graphic information about the variance homogeneity and allow detecting the possible existence of any type of relation between the size of the means and the size of the variances.As the variances are not equal, as tested by the Levene test, points in Figures 9.6a and 9.6b are not horizontally aligned.

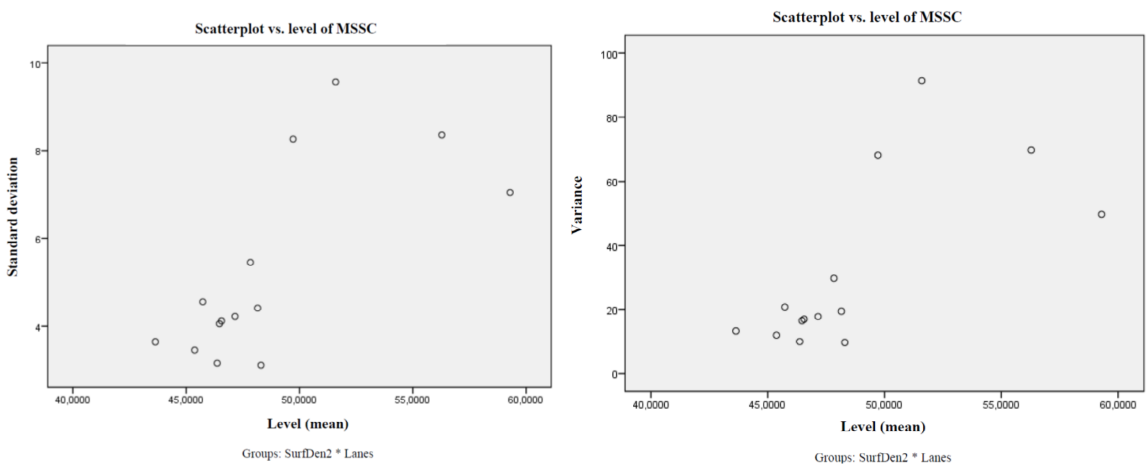


Fig. 9.6. Scatterplots by level for Eq. 9.10. a) Standard deviation, b) Variance.

The plot of residuals of Fig. 9.7 alows observing that they are random and independent between them. As the plot of predicted values vs standardized residuals is random (there are not any pattern), the residuals are independent. The residual variances are homogeneous since the dispersion of the standardized residuals is similar along all the values of predicted values. Predicted and observed values show a linear pattern, which is reflected by the coefficient of determination.

The plot of the profil of the effects (Fig. 9.8) shows that the lines are not crossed, indicating that the iteraction between them is significative. Each point represent the mean values of the dependent variable (MSSC) found in each subgroup after combining each level of Lanes with each level of SurfDen2. A more detailed correlation between observed values and predicted values can be observed in Fig. 9.9.

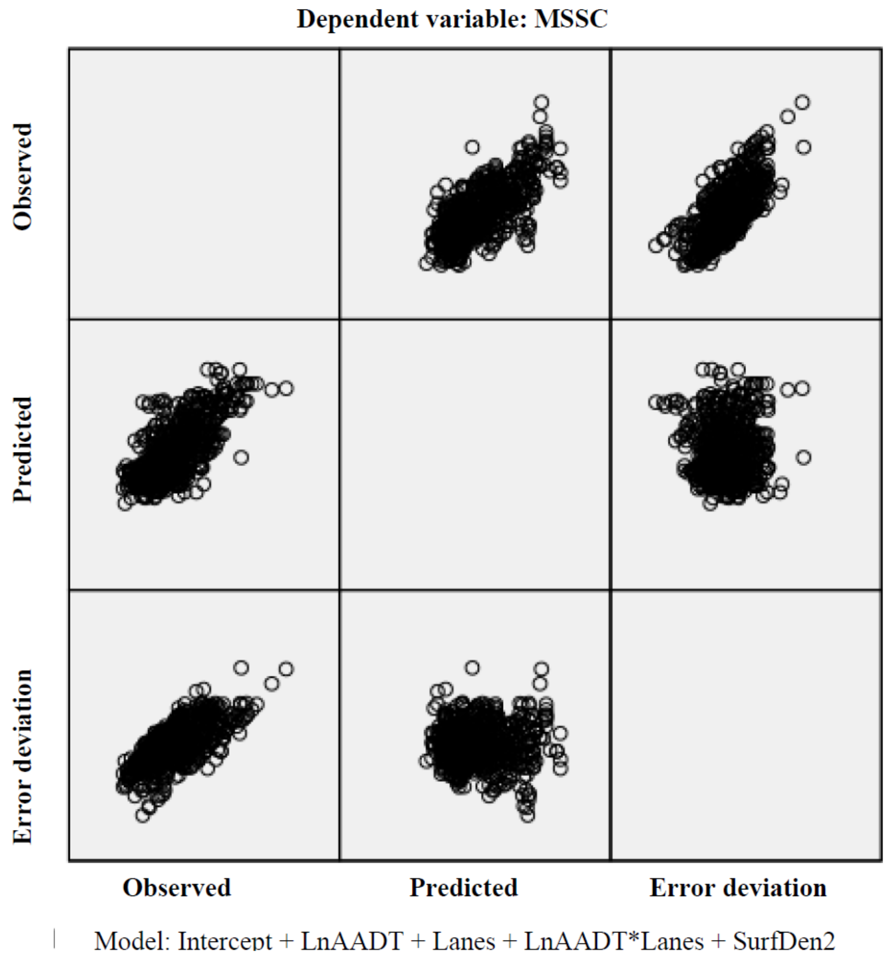


Fig. 9.7. Plot of residuals (standardized), observed and predicted values of model of Eq. 9.10.

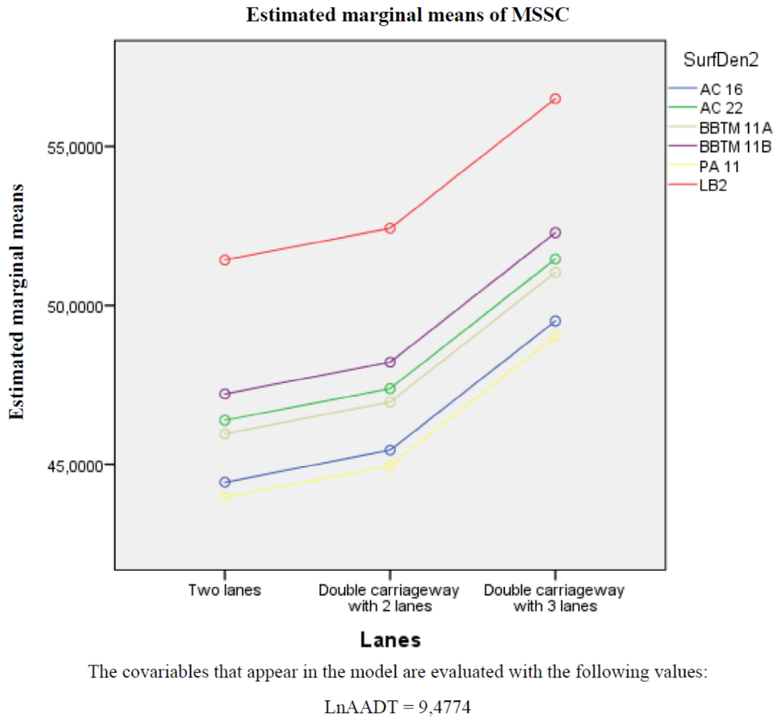


Fig. 9.8. Plot of the profils of the effects of the factors Lanes and SurfDen2 with model of Eq. 9.10

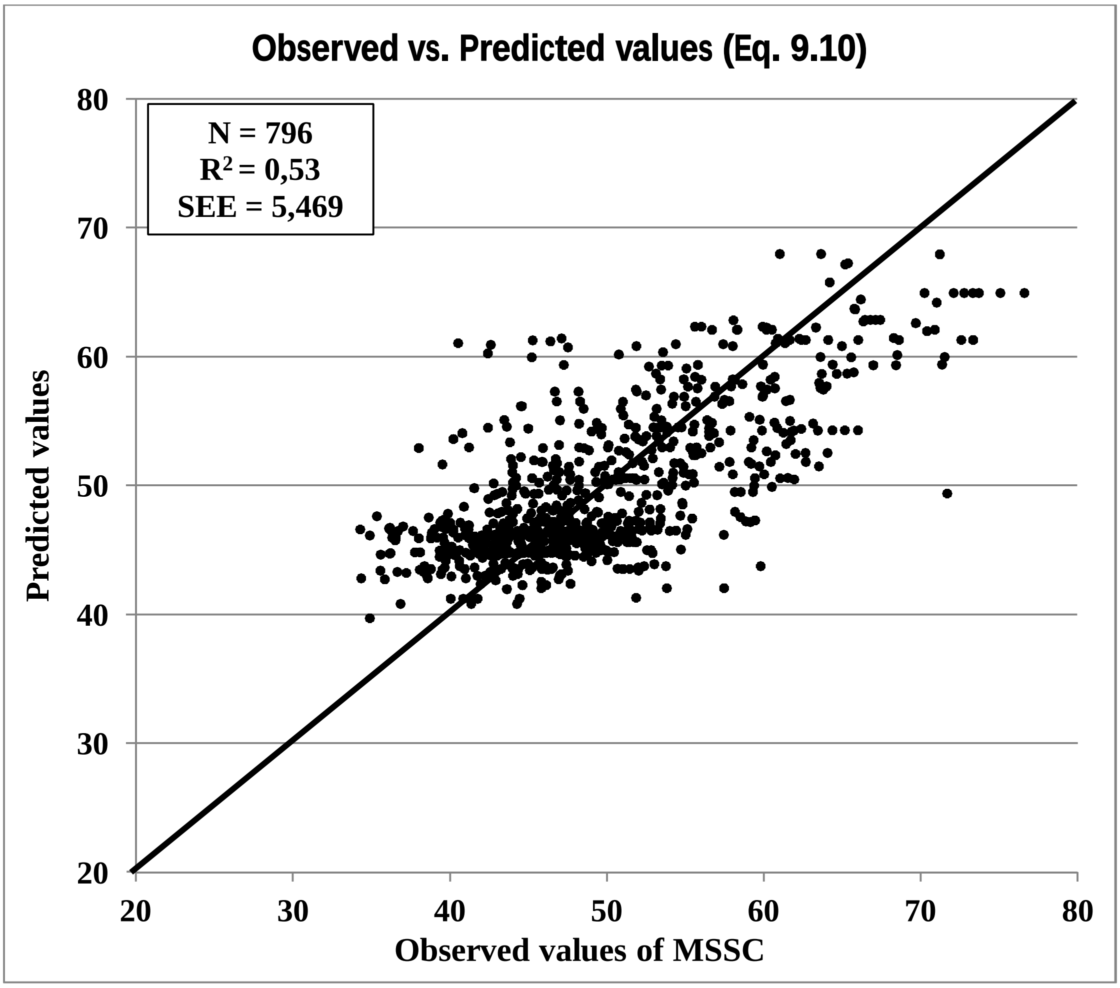


Fig. 9.9. Observed vs. Predicted values with Eq. 9.10.

With regard to the model proposed in Eq. 9.11 the test of Between-Subjects effects of the model is presented in Table 9.125. As seen all the factors and variables have a significance over 95 % (p-value < 0,05). The estimation of the parameters (coefficients) of the long model (Eq. 9.11) is displayed in Table 9.126.

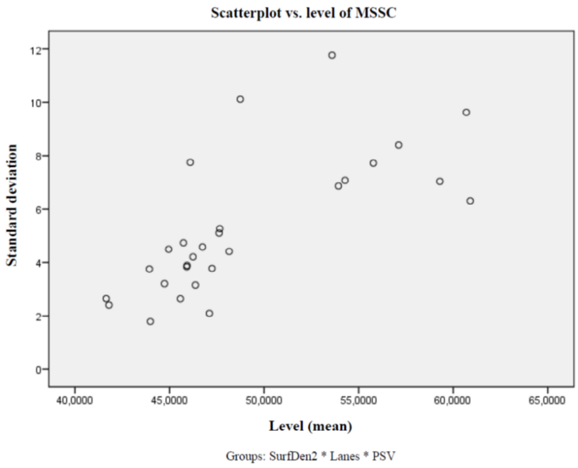
Table 9.125. Test of Between-Subjects effects for the model of Eq. 9.11, proposed short model

Origin	Type III Sum of Squares	Degree of freedom	Mean Square	F	Sig.	Partial eta-squared	Non centrality parameter	Observed Power
Corrected model	28298,071[a]	43	658,095	22,226	< 0,001	0,560	955,701	1,000
Intercept	1711,608	1	1711,608	57,806	< 0,001	0,071	57,806	1,000
LnAADT	652,245	1	652,245	22,028	< 0,001	0,028	22,028	0,997
Lanes	263,004	2	131,502	4,441	0,012	0,012	8,882	0,763
Lanes*LnAADT	870,993	2	435,496	14,708	< 0,001	0,038	29,416	0,999
SurfDen2	899,968	5	179,994	6,079	< 0,001	0,039	30,394	0,996
PSV * SurDen2 * Lanes * LnAADT	1482,21	33	44,915	1,517	0,033	0,062	50,058	0,992
Error	22266,529	752	29,610					
Total	2015919,12	796						
Corrected total	50564,6	795						

Table 9.126. Parameter estimates for the model of Eq. 9.11, long model

Parameters	B	Std. Error	t	Sig.	95% CI		Observed Power
					Lower	U`pper	
Intercept	114,946	26,275	4,375	0,000	63,364	166,527	0,992
LnAADT	-2,065	2,420	-0,853	0,394	-6,817	2,686	0,136
[Lanes=1]	9,473	25,685	0,369	0,712	-40,949	59,896	0,066
[Lanes=2]	-19,756	24,738	-0,799	0,425	-68,319	28,807	0,125
[Lanes=3]	0[a]	.	.	.	.	.	.
[Lanes=1] * LnAADT	-5,719	2,497	-2,291	0,022	-10,620	-0,818	0,629
[Lanes=3] * LnAADT	1,817	2,227	0,816	0,415	-2,554	6,189	0,129
[Lanes=3] * LnAADT	0[a]	.	.	.	.	.	.
[SurfDen2=1]	-34,317	7,609	-4,510	0,000	-49,255	-19,379	0,995
[SurfDen2=2]	63,566	37,759	1,683	0,093	-10,560	137,692	0,390
[SurfDen2=3]	-40,151	10,373	-3,871	0,000	-60,515	-19,787	0,972
[SurfDen2=4]	-0,722	50,872	-0,014	0,989	-100,590	99,147	0,050
[SurfDen2=5]	-47,639	18,074	-2,636	0,009	-83,121	-12,157	0,749
[SurfDen2=6]	0[a]	.	.	.	.	.	.
[PSV=40]*[SurfDen2=1]*[Lanes=1]* LnAADT	3,243	0,923	3,515	0,000	1,432	5,054	0,939
[PSV=40]*[SurfDen2=2]*[Lanes=1]*LnAADT	-7,108	3,993	-1,780	0,075	-14,946	0,731	0,428
[PSV=40]*[SurfDen2=2]*[Lanes=2]*LnAADT	-10,560	4,398	-2,401	0,017	-19,194	-1,926	0,669
[PSV=40]*[SurfDen2=6]*[Lanes=1]*LnAADT	-1,060	0,842	-1,258	0,209	-2,713	0,594	0,242
[PSV=44]*[SurfDen2=1]*[Lanes=1]*LnAADT	3,063	0,939	3,262	0,001	1,220	4,906	0,903
[PSV=44]*[SurfDen2=2]*[Lanes=1]*LnAADT	-10,389	5,242	-1,982	0,048	-20,679	-0,099	0,508
[PSV=44]*[SurfDen2=6]*[Lanes=1]*LnAADT	-0,374	0,238	-1,570	0,117	-,842	0,094	0,348
[PSV=45]*[SurfDen2=1]*[Lanes=1]*LnAADT	3,115	0,844	3,691	0,000	1,459	4,772	0,958
[PSV=45]*[SurfDen2=1]*[Lanes=2]*LnAADT	-1,595	1,684	-0,947	0,344	-4,900	1,710	0,157
[PSV=45]*[SurfDen2=1]*[Lanes=3]*LnAADT	-1,090	1,627	-0,670	0,503	-4,284	2,105	0,103
[PSV=45]*[SurfDen2=2]*[Lanes=1]*LnAADT	-7,302	4,254	-1,716	0,087	-15,654	1,050	0,403
[PSV=45]*[SurfDen2=2]*[Lanes=3]*LnAADT	-9,791	3,716	-2,635	0,009	-17,086	-2,496	0,749
[PSV=45]*[SurfDen2=3]*[Lanes=1]*LnAADT	4,993	1,401	3,565	0,000	2,243	7,742	0,945
[PSV=45]*[SurfDen2=5]*[Lanes=1]*LnAADT	5,007	2,237	2,239	0,025	,617	9,398	0,609
[PSV=45]*[SurfDen2=5]*[Lanes=2]*LnAADT	-0,328	0,423	-0,776	0,438	-1,159	0,503	0,121
[PSV=45]*[SurfDen2=6]*[Lanes=1]*LnAADT	-0,435	0,143	-3,038	0,002	-0,716	-0,154	0,859
[PSV=50]*[SurfDen2=1]*[Lanes=1]*LnAADT	2,903	0,842	3,446	0,001	1,249	4,557	0,931
[PSV=50]*[SurfDen2=1]*[Lanes=2]*LnAADT	-1,130	1,666	-0,678	0,498	-4,401	2,141	0,104
[PSV=50]*[SurfDen2=2]*[Lanes=1]*LnAADT	-8,084	4,293	-1,883	0,060	-16,511	0,343	0,468
[PSV=50]*[SurfDen2=2]*[Lanes=3]*LnAADT	-9,397	3,716	-2,529	0,012	-16,692	-2,102	0,714
[PSV=50]*[SurfDen2=3]*[Lanes=1]*LnAADT	3,834	1,125	3,406	0,001	1,624	6,043	0,925
[PSV=50]*[SurfDen2=3]*[Lanes=2]*LnAADT	-0,663	1,424	-0,465	0,642	-3,459	2,133	0,075
[PSV=50]*[SurfDen2=3]*[Lanes=3]*LnAADT	-0,392	1,384	-0,283	0,777	-3,109	2,326	0,059
[PSV=50]*[SurfDen2=4]*[Lanes=1]*LnAADT	-0,468	5,442	-0,086	0,931	-11,151	10,215	0,051
[PSV=50]*[SurfDen2=4]*[Lanes=2]*LnAADT	-4,427	5,199	-0,851	0,395	-14,633	5,780	0,136
[PSV=50]*[SurfDen2=5]*[Lanes=1]*LnAADT	4,482	1,939	2,311	0,021	0,675	8,289	0,636
[PSV=50]*[SurfDen2=5]*[Lanes=2]*LnAADT	-0,091	0,357	-0,254	0,799	-0,792	0,610	0,057
[PSV=50]*[SurfDen2=5]*[Lanes=3]*LnAADT	0,112	0,264	0,423	0,672	-0,407	0,630	,071
[PSV=50]*[SurfDen2=6]*[Lanes=1]*LnAADT	0[a]	.	.	.	.	.	.
[PSV=50]*[SurfDen2=6]*[Lanes=2]* LnAADT	-3,604	1,801	-2,001	0,046	-7,140	-0,068	0,515
[PSV=55]*[SurfDen2=3]*[Lanes=2]*LnAADT	-0,626	1,408	-0,445	0,657	-3,390	2,138	0,073
[PSV=55]*[SurfDen2=3]*[Lanes=3]*LnAADT	-0,320	1,390	-0,230	0,818	-3,049	2,410	0,056
[PSV=56]*[SurfDen2=3]*[Lanes=2]*LnAADT	-0,570	1,408	-0,405	0,686	-3,334	2,194	0,069
[PSV=56]*[SurfDen2=3]*[Lanes=3]*LnAADT	-0,456	1,385	-0,329	0,742	-3,175	2,264	0,062
[PSV=56]*[SurfDen2=5]*[Lanes=2]*LnAADT	0[a]	.	.	.	.	.	.
[PSV=56]*[SurfDen2=5]*[Lanes=3]*LnAADT	0[a]	.	.	.	.	.	.

Fig. 9.10a. and 9.10b show the diagrams of dispersion by level. As seen, the points in both plots are not horizontally aligned, indicating that there are not homogeneous variances between the levels.



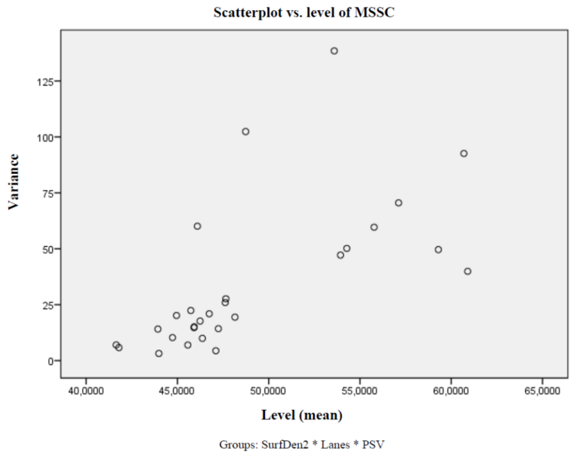


Fig. 9.10. Dispersion diagrams by level for Eq. 9.11, a) Standard deviation, b) Variance.

Fig. 9.11 shows the plot of residuals and it can be observed in the plot “Predicted values vs. Standardized residuals” that there is not pattern and hence, residuals are independent. This plot also shows that the dispersion is similar along all the predicted values, indicating that the residuals variances are homogeneous. The plot of observed and predicted show a linear pattern, contrasted by the coefficient of determination (R^2 = 0,560).

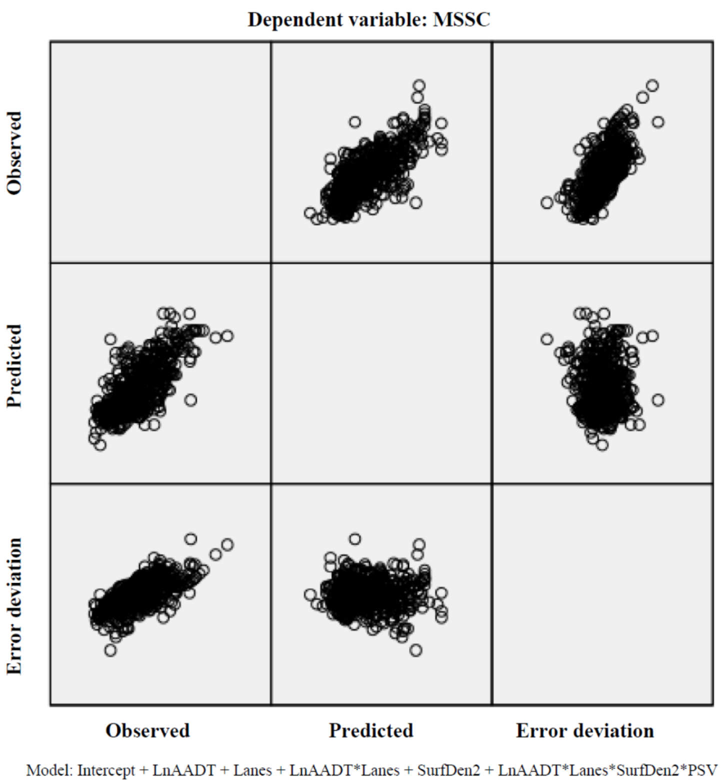


Fig. 9.11. Plot of residuals (standardized), observed and predicted values of the model of Eq. 9.11.

In Fig. 9.12 a more detailed plot of observed values vs. predicted values for Eq. 9.11 is displayed.

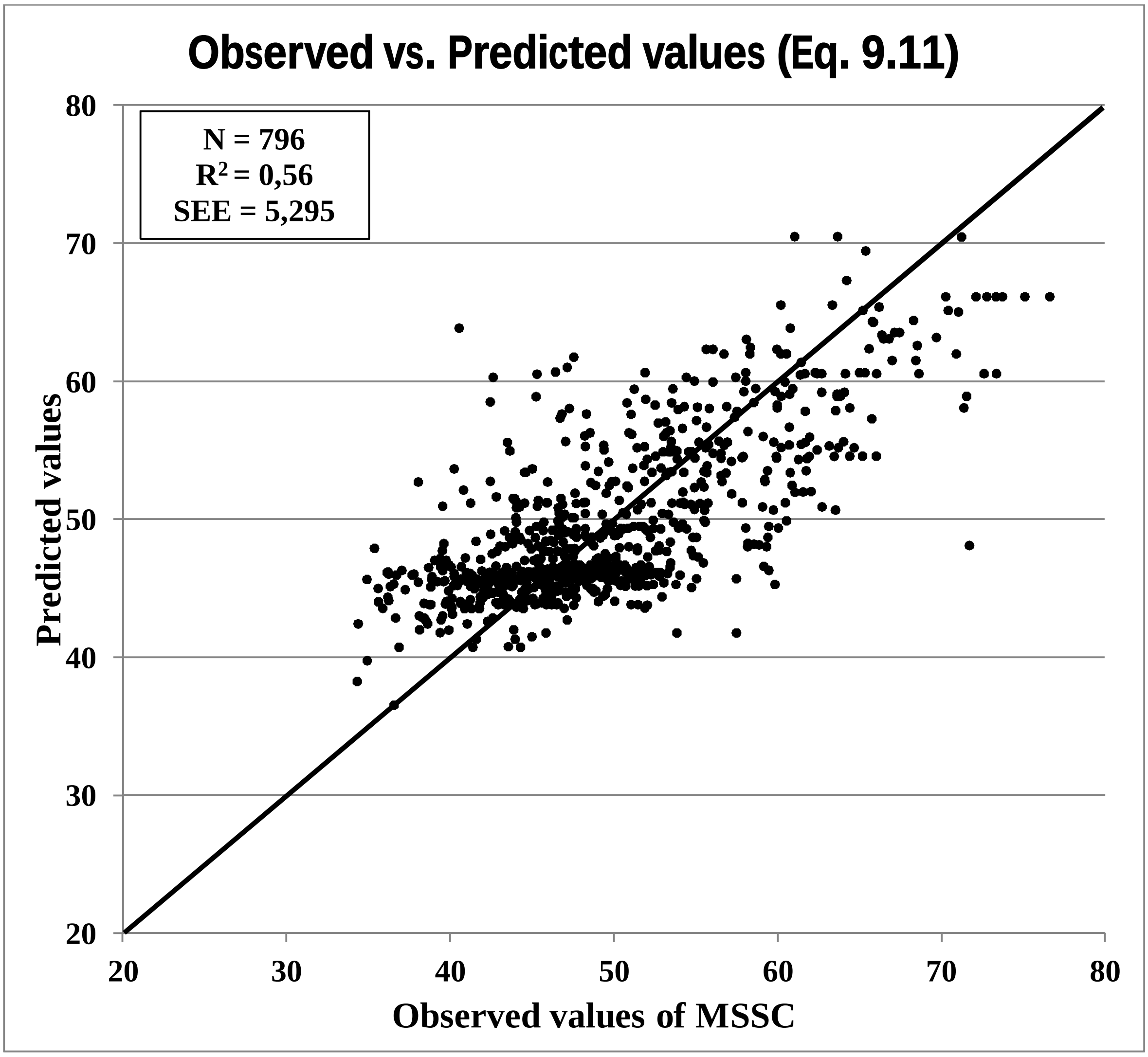


Fig. 9.12. Observed vs. Predicted values with Eq. 9.10.

Unlike to the IRI models where the data for heavy traffic (*TotH.Veh*, accumulated total number of heavy vehicles that circulated over the project lane) was preferred instead of data for total traffic (*TotVe*h, accumulated total number of heavy vehicles that circulated in both direction), for skid resistance prediction it was preferred to employ the *AADT* of total traffic instead of *H.AADT*. Employing the same structure as in the proposed equations for friction (Eq. 9.10 and Eq. 9.10) a greater difference in the coefficient of determination (R2) is observed if Ln*H.AADT* is employed instead of Ln*AADT* (Table 9.127).

Table 9.127. Coefficients of determination of Eq. 9.10 and 9.11 employing LnAADT and LnH.AADT

Variable related to traffic introduced	Model as proposed in Eq. 9.10	Model as proposed in Eq. 9.11
LnAADT	0,530	0,560
LnH.AADT	0,464	0,510

As seen, the difference is greater in friction prediction models that in IRI models. Moreover, the correlation between both variables related to traffic is not as high as for IRI models (Table 8.22), as shown in Table 9.128.

Table 9.128. Correlation between the variables related to Annual Average Daily traffic(coefficient of Pearson)

	AADT	H.AADT	LnAADT	LnH.AADT
AADT	1	0,864	0,843	0,769
H.AADT	0,864	1	0,759	0,846
LnAADT	0,843	0,759	1	0,911
LnH.AADT	0,769	0,846	0,911	1

This greater difference in the correlation between the variables related to traffic is logical as a greater quantity of roads are included in the analysis and the number of observation are much greater (105 for IRI modelling, 796 for MSSC modelling). Therefore, it is considered that it is not a biased analysis of a low quantity of data. In this analysis all the know road surface of the territory of Biscay are included, and results explain that better correlation is found with Annual Average Daily Traffic including all the vehicles. This fact is different if compared with the models proposed by Szatkwoski and Hosking (1972) and Transit NZ (2002a). Nevertheless, Szatkwoski and Hosking (1972) indicated that if introducing AADT of total traffic in the model (Eq. 5.69) a high correlation was also found (R = 0,84) (Eq. 5.70), but lower than with Qcv, number of Commercial Vehicles (CV) (> 1.500 kg) per lane per day (R = 0,91). Nonetheless, later, when verified the equation again with values recorded in practice, it was shown that the correlation was not so high, underestimating the MSSC achieved in practive at higher traffic levels and at lower traffic, it predicted higher values that those actually observed.

Moreover, it is difficult to know if the polishing action originated in laboratory tests is similar to the one produced by heavy vehicles or to the one produced by all the vehicles, combining heavy and passenger cars. The polishing action, as shown in Fig. 5.47 and Fig. 5.53 shows that the equilibrium phase is reached after some polishing cycles and from then, the values do not get worse. However, laboratory conditions are not able to reproduce the real condition, when variated vehicles polish the surface aggregates and the weather condition which allows the surface to be affected by seasonal variations.

Consequently, considering all the explanations above, for skid resistance prediction in Biscay, two equations including the Annual Averange Daily Traffic (and not the Annual Average Daily Traffic of heavy vehicles) are selected as performing models.

9.5. Summary of the proposed models

The following models are proposed for predicting the available Mean Summer SCRIM Coefficient in any road according to the type of surface layer material, the Average Annual Daily Traffic, the type of road and the required Polished Stone Value of the aggregates of the surface layer in the regulations.

The surface layer materials that can be used in the model are (SurfDen2):

e) Asphalt Concrete, with a maximum aggregate size of 16 mm (AC 16) or 22 mm (AC 22)
f) Discontinuous mixes, including BBTM 11A or BBTM 11B
g) Porous Asphalt, PA 11
h) Slurry, more specifically the category LB2

The road types that can be introduced in the model depends on the number of lanes in each direction (Lanes):

d) Two-lane roads, with one lane in each direction
e) Double carriageway motorways, with two lanes in each direction
f) Double carriageway motorways, with three or four lanes in each direction

The required Polished Stone Value is established according to the different regulations in Spain, which depends on the surface layer material, the Heavy Traffic Category of the lane when it is opened to the traffic and the year when the road was opened to traffic (to know the regulation that was in force in that moment).

The short model, which has verified a coefficient of determination of 0,530, is expressed by Eq. 9.12:

$$MSSC = 78{,}497 - 2{,}321 * LnAADT + A_{LANES} + B_{SURF} + C_{LANES} * LnAADT \quad [9.12]$$

Where

MSSC is the Mean Summer SCRIM Coefficient, expressed in a range of 0 to 100.

Ln*AADT* is the natural logarithm of the Average Annual Daily Traffic of the road, in both directions, expressed in vehicles/day.

A_{LANES} is the coefficient that considers the type of road and has the values of Table 9.129.

Table 9.129. Values of the coefficient A_{LANES} in Eq. 9.12

Type of road	A_{LANES}
Two-lane roads, with one lane in each direction	26,693
Double carriageway motorways, with two lanes in each direction	16,663
Double carriageway motorways, with three or four lanes in each direction	0

B_{SURF} is the coefficient that takes into account the surface layer material and has the values of the Table 9.130.

Table 9.130. Values of the coefficient B_{SURF} in Eq. 9.12.

Surface layer material	B_{SURF}
AC 16	-6,989
AC 22	-5,047
BBTM 11A	-5,468
BBTM 11B	-4,213
PA 11	-7,467
LB2	0

C_{LANES} is the coefficient that affect the value of the LnAADT, to reflect the more real distribution of the traffic in the right lane and has the values of the of Table 9.131.

Table 9.131. Values of the coefficient C_{LANES} in Eq. 9.12.

Type of road	C_{LANES}
Two-lane roads, with one lane in each direction	-3,352
Double carriageway motorways, with two lanes in each direction	1,328
Double carriageway motorways, with three or four lanes in each direction	0

For a more exact value, a long model is proposed, Eq. 9.13, which has a slightly better coefficient of determination (R^2 = 0,560) and, apart from the variables described in Eq. 9.12 (with different values), includes an additional term to reflect the combination of the required *PSV*, the surface layer material, the type of road and the AADT of the road (in both directions).

$$MSSC = 114{,}946 - 2{,}065 \cdot LnAADT + A_{LANES} + B_{SURF} + C_{LANES} \cdot LnAADT + D_{S-P-L} \cdot LnAADT \quad [9.13]$$

Where

MSSC is the Mean Summer SCRIM Coefficient, expressed in a range of 0 to 100.

Ln*AADT* is the natural logarithm of the Average Annual Daily Traffic of the road, in both directions, expressed in vehicles/day.

A_{LANES} is the coefficient that considers the type of road and has the values of Table 9.132.

Table 9.132. Values of the coefficient A_{LANES} in Eq. 9.13, proposed long model.

Type of road	A_{LANES}
Two-lane roads, with one lane in each direction	9,473
Double carriageway motorways, with two lanes in each direction	-19,756
Double carriageway motorways, with three or four lanes in each direction	0

B_{SURF} is the coefficient that takes into account the surface layer material and has the values of the Table 9.133.

Table 9.133. Values of the coefficient B_{SURF} in Eq. 9.13, proposed long model.

Surface layer material	B_{SURF}
AC 16	-34,317
AC 22	63,566
BBTM 11A	-40,151
BBTM 11B	-0,722
PA 11	-47,639
LB2	0

C_{LANES} is the coefficient that affect the value of the Ln*AADT*, to reflect the more real distribution of the traffic in the left lane and has the values of the of Table 9.134.

Table 9.134. Values of the coefficient C_{LANES} in Eq. 9.13, proposed long model.

Type of road	C_{LANES}
Two-lane roads, with one lane in each direction	-5,719
Double carriageway motorways, with two lanes in each direction	1,817
Double carriageway motorways, with three or four lanes in each direction	0

$D_{S\text{-}P\text{-}L}$ is the coefficient that considers the combination of the surface layer material, the required Polished Stone Value of the aggregates in the regulation and the type of road. As there is a multiple combination of the levels of these factors, a long list of values is presented for each combination. They are listed according to the road type, which only has 3 levels (Tables 9.133 to 9.135).

Table 9.135. Values of the coefficient $D_{S\text{-}P\text{-}L}$ for two-lane roads, with one lane in each direction in Eq. 9.13, proposed long model

Surface layer material	**PSV**					
	40	**44**	**45**	**50**	**55**	**56**
AC 16	3,243	3,063	3,115	2,903	-	-
AC 22	-7,108	-10,389	-7,302	-8,084	-	-
BBTM 11A	-	-	4,993	3,834	-	-
BBTM 11B	-	-	-	-0,468	-	-
PA 11	-	-	5,007	4,482	-	-
LB2	-1,060	-0,374	-0,435	0	-	-

Table 9.136. Values of the coefficient $D_{S\text{-}P\text{-}L}$ for double carriageway motorways, with two lanes in each direction in Eq. 9.13, proposed long model

Surface layer material	**PSV**					
	40	**44**	**45**	**50**	**55**	**56**
AC 16	-	-	-1,595	-1,130	-	-
AC 22	-10,560	-	-	-	-	-
BBTM 11A	-	-	-	-0,663	-0,626	-0,570
BBTM 11B	-	-	-	-4,427	-	-
PA 11	-	-	-0,328	-0,091	-	0
LB2	-	-	-	-3,604	-	-

Table 9.137. Values of the coefficient $D_{S\text{-}P\text{-}L}$ for double carriageway motorways, with three or four lanes in each direction in Eq. 9.13, proposed long model

Surface layer material	**PSV**					
	40	**44**	**45**	**50**	**55**	**56**
AC 16	-	-	-1,090	-	-	-
AC 22	-	-	-9,791	-9,397	-	-
BBTM 11A	-	-	-	-0,392	-0,320	-0,456
BBTM 11B	-	-	-	-	-	-
PA 11	-	-	-	0,112	-	0
LB2	-	-	-	-	-	-

As it can be observed, not all the possible combinations of the levels of the three factors (Lanes, PSV_{req} and

SurfDen2) are indicated. From the specifications themselves, some combinations are impossible, as it would not be feasible to have a T00 traffic category in a two-lane road, which would require a *PSV*=56. Similarly, a three-lane motorway will have a high traffic category that would require a high PSV. Moreover, for motorways with high volumes, in Biscay it is preferably recommended to employ discontinuous mixes and porous asphalts, since the rainfall data along the year is high due to the oceanic climate. In the case a combination is not displayed in Table 9.133 to 9.135, it is suggested to only employ the short model (Eq. 9.12).

9.6. Conclusions

In the chapter, a complete analysis of the factors that affect the available skid resistance was conducted. In a first step, assuming the seasonal variations documented in the literature, values from 2011, which were collected in winter were transformed in summer values, which are supposed to be at their minimum. The transformation was conducted using variation range observed in a research in the province next to Biscay, Gipuzkoa, which has similar climate. Influencing factors were Average Annual Daily Heavy Traffic (*H.AADT*) and the required *PSV*.

In 2016, skid resistance values were collected in the road network of Biscay in summer, so it can be assumed that they represent the minimum available friction values. Firstly, taking advantage of the analysis of the completely known sections that was carried out for the *IRI* analysis, only sections with known pavement section in two-lane roads were studied. Results showed that the pavement type (flexible or semi-rigid), the total thickness of bituminous layers and the type of work conducted (the section may come from a new outline or from a rehabilitation or maintenance activity) do not influence the skid resistance. Variables that really influenced the friction were the traffic volume (total or only heavy traffic), the surface layer material and required *PSV*. Taking this fact into account, it is possible to analyze all the sections if the surface layer material and traffic volumes are known. The analysis was extended to all the two-lane roads of Conservation Areas 1, 2 and 3 and results were satisfactory. Double carriageway stretches in these areas (1, 2 and 3) were included and better results were obtained. Consequently, roads from Conservation Area 4, which is composed of the motorways in the metropolitan area of Bilbao, were also introduced in the modeling.

A new variable was introduced, *Lanes*, to consider the quantity of lanes in the different road. Additional classifications for the surface materials were also included to gather the materials in homogeneous groups. After analysing several multiple linear regression models and General Linear Models, which can introduce qualitative variables, two models were proposed. A short model, which can referred as the basic one, that only considers on the quantity of lanes of the road, the surface layer material and the Average Annual Daily Traffic of the whole road in both directions and a long model, which can be regarded as a complementary one, which also includes the influence of the required Polished Stone Value according to the regulations.