Unpaved Low-Volume Roads Conceptual Framework Functional Evaluations and comparative Design Approaches

Mohamed Amir Hajri

Thesis submitted to the School of Technology and Management of the Polytechnic Institute of Bragança (Portugal) in collaboration with the Free University of Tunis (Tunisia) to satisfy the requirements of the double degree programme for the award of the title of

Master's degree in construction engineering

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Abstract

The road network plays a crucial role in the economic, social, and cultural development of a country. It connects rural and urban areas and facilitates the transportation of people and goods across the territory. Unpaved roads, often originating from dirt paths or trails, are characterized by relatively low construction costs and a high tolerance for deformation. Due to their importance, it is essential to design and study them properly to meet users' needs while ensuring safe and comfortable travel conditions.

This work presents a comparative analysis of three design methods used to improve the conditions of low-traffic unpaved roads. The first method comes from the ERA manual (2011), which uses the CBR and traffic variables to define the design. The second method is from Austroads (2009), which also considers these two variables. Finally, the FCE manual (Brito, 2011) relies solely on soil classification and the CBR value to determine the thickness of the road's base layer.

The study focuses on a road located in the Montesinho Natural Park, which provides access between two dams supplying the Bragança district. Thanks to laboratory and field tests conducted by Cabette (2018) and Freitas (2019), several physical and mechanical characteristics of the road structure were assessed, allowing the necessary thickness of the base layer to be determined based on the applied conditions.

The base layer thicknesses determined by the three methods are similar, ranging from 10 to 15 cm. Through a sensitivity analysis of the input variables for the different methodologies, it was found that for the ERA manual (2011), the most sensitive variable is traffic, while for the Austroads (2009) method, the most decisive factor is the CBR value, and for the FCE manual (Brito, 2011), the most sensitive variables are the foundation material's strength and the CBR value.

Keywords

Unpaved roads; design; method; manual.

Resumo

A rede rodoviária desempenha um papel crucial no desenvolvimento económico, social e cultural de um país. Liga as zonas rurais e urbanas e facilita o transporte de pessoas e mercadorias através do território. As estradas não pavimentadas, muitas vezes com origem em caminhos de terra ou trilhos, caracterizam-se por custos de construção relativamente baixos e uma elevada tolerância à deformação. Devido à sua importância, é essencial concebê-las e estudá-las adequadamente para satisfazer as necessidades dos utilizadores, assegurando simultaneamente condições de viagem seguras e confortáveis.

Este trabalho apresenta uma análise comparativa de três métodos de dimensionamento utilizados para melhorar as condições de estradas não pavimentadas de baixo tráfego. O primeiro método é o do manual ERA (2011), que utiliza as variáveis CBR e tráfego para definir o projeto. O segundo método é o da Austroads (2009), que também considera estas duas variáveis. Por fim, o manual da FCE (Brito, 2011) baseia-se apenas na classificação do solo e no valor do CBR para determinar a espessura da camada de base da estrada.

O estudo incide sobre uma estrada localizada no Parque Natural de Montesinho, que permite o acesso entre duas barragens que abastecem o distrito de Bragança. Graças aos ensaios laboratoriais e de campo realizados por Cabette (2018) e Freitas (2019), foram avaliadas várias caraterísticas físicas e mecânicas da estrutura da estrada, permitindo determinar a espessura necessária da camada de base com base nas condições aplicadas.

As espessuras da camada de base determinadas pelos três métodos são semelhantes, variando entre 10 e 15 cm. Através de uma análise de sensibilidade das variáveis de entrada para as diferentes metodologias, verificou-se que para o manual ERA (2011), a variável mais sensível é o tráfego, enquanto que para o método Austroads (2009), o fator mais determinante é o valor do CBR, e para o manual FCE (Brito, 2011), as variáveis mais sensíveis são a resistência do material de fundação e o valor do CBR.

palavras-chave

Estradas não pavimentadas; conceção; método; manual.

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List of abbreviations

AASHTO: American Association of State Highway and Transportation Officials

CBR: California Bearing Ratio

CP: Plate Load Test (Carga em Placa)

DER: Road Department (Departamento de Estradas de Rodagem)

DVI: Detailed Visual Inspection

EBVT: Low-Volume Traffic Road (Estrada de Baixo Volume de Tráfego)

ERA: Ethiopian Roads Authority

ERCI: Earth Road Condition Index

ESA: Equivalent Standard Axle

FCE: Forestry Civil Engineering

FWD: Falling Weight Deflectometer

GPM: Gravel PASER Manual

GRMS: Gravel Roads Management: Implementation Guide

LFWD: Light Falling Weight Deflectometer

MTPW: Ministry of Transport and Public Works

PASER: Pavement Surface Evaluation and Rating

PDL: Lightweight Dynamic Penetrometer (Penetrômetro Dinâmico Ligeiro)

RCS: Road Condition Survey

RSMS: Road Surface Management System

SATCC: Southern Africa Transport and Communications Commission

TMDA: Annual Average Daily Traffic (Tráfego Médio Diário Anual)

URCI: Unsurfaced Road Condition Index

Chapter I: Introduction

1. Introduction

1.1. Framework

The road network is a fundamental element in the economic, social, and cultural development of a country. It facilitates the transport of people and goods across the entire territory while creating employment opportunities, both directly and indirectly, in the transportation sector.

Dirt paths and trails have gradually evolved into unpaved roads. These roads emerged to address the need to connect rural areas to urban centers, thereby enabling rural populations to access essential services such as healthcare, education, and leisure provided by cities, while also supporting the transportation of agricultural products. This type of road is often chosen due to its relatively low construction cost and its ability to withstand significant deformations (Rodrigues, 2015).

However, frequent maintenance and conservation interventions are required for these roads, mainly due to the absence of inadequacy of proper drainage systems and the low resistance of the soil composing these roads. This issue arises because such infrastructures are typically not designed to accommodate heavy traffic, including trucks and agricultural machinery. These limitations, combined with climatic conditions such as heavy rainfall, exacerbate the damage, leading to significant deformations, localized failures, and frequent erosion. These problems make access difficult and increase discomfort for users (Guedes, 2018).

Table 1 provides a summarized overview of the proportion of unpaved roads relative to the total road network (in kilometers) in several countries, highlighting their importance in road transportation.

Table 1. Percentage of unpaved roads relative to the total road network (km) in some countries. (Adapted from CIA, 2020).

Continent	Country	Roads (km)	Unpaved Roads (%)	year
Africa	South Africa	750.000	79,8 %	2016
	Egypt	65.050	26,2 %	2017
	Angola	6.586.610	47,4 %	2018
America	Canada	1.042.300	60,1 %	2011
	Brasil	2.000.000	87,7 %	2018
	Argentine	281.290	58,2 %	2017
	Cuba	60.000	66,7 %	2015
Europa	Portugal	82.900	14,0 %	2008
	Hungary	303.601	62,1 %	2014
	Sweden	573.134	75,6 %	2016
Asia	Japon	1.218.772	18,5 %	2015
	Chine	4.960.600	12,5 %	2017
	Russia	1.283.387	27,7 %	2012

Based on the data in Table 1, it is clear that the proportion of unpaved roads is high worldwide. This phenomenon cannot be directly linked to a country's level of wealth, as even developed nations such as Japan, the United States, and Canada have significant percentages of unpaved roads, with 18.5%, 34.6%, and 60.1%, respectively.

Countries like Brazil, South Africa, and Sweden are among those with the highest rates, with more than 75% of their road networks consisting of unpaved roads. In comparison, Portugal has one of the lowest percentages, with only 14% of its national network being unpaved, although this figure remains significant relative to its total infrastructure.

Given the widespread presence of unpaved roads and their critical role in regional and national development, it is essential to conduct research aimed at improving design, construction, and maintenance methods. Such advancements would help extend the lifespan of these roads, ensure greater comfort and safety for users, and minimize the associated environmental impacts.

1.2. Objectives

The main objective of this work is to conduct a comparative analysis of the design methods for low-traffic unpaved roads used in countries such as the United States, Australia, and the United Kingdom.

The secondary objectives are as follows:

- Present different design methods for low-traffic unpaved roads.
- Apply the parameters obtained from previous studies to the designs conducted in this research.
- Compare the results of previous tests.
- Perform a sensitivity analysis of the methods used.
- Assess the feasibility of applying the described methods.

1.3. Organization of the dissertation

The structure of this work is composed of five chapters.

In Chapter 1, an introduction to the central theme of the dissertation is presented. Subsequently, the objectives and the organization of the work are outlined.

Chapter 2 provides relevant information for the study, divided into two parts: the first addresses the transportation system in general and specifies the concepts used to define LBVT roads and unpaved roads, which are the two classifications to which the studied road belongs; the second conceptualizes mechanical and functional evaluations, presenting the various tests and methodologies used for this purpose.

In Chapter 3, the design of a studied road-located in the district of Bragança, Portugal-is carried out using the results obtained from previous studies and the methods presented in Chapter 2.

Chapter 4 aims to compare the results of the tests conducted previously, the studied design methods, and to present a sensitivity analysis of the various design methodologies.

Finally, Chapter 5 contains the most important conclusions of the study and recommendations for future research.

Chapter II: Bibliographical review

2.1. Types of Roads

Road design is a crucial phase prior to the construction or rehabilitation of road infrastructures. This stage involves defining the essential characteristics of the road, such as its type and the materials to be used for its various layers. The selection of the pavement type is influenced by a number of determining factors, including the anticipated traffic load, subgrade properties, local climatic conditions, and the availability of construction materials. To guide this design, preliminary studies provide essential site and region data, enabling technical choices to be adjusted in line with specific project needs and environmental constraints (CEPSA, 2014). The main types of roads commonly found in road networks are:

- Paved roads, in which roads are included in flexible, rigid and semi-rigid pavements.
- Unpaved roads, which include road on soil (earth) and roads on artificial aggregates.

2.1.1 Paved roads

Paved, or coated roads are roads that have traffic lanes whose surface is covered with a durable material, usually integrating mixes with asphalt bitumen, and/or cement concrete. This surfacing improves the quality of the road by making it smoother, more weather-resistant and more durable. The main purpose of surfacing is to provide a stable, uniform surface for vehicles, improve driving comfort and reduce wear and tear on vehicles and road infrastructure, extending their useful life and reducing the number of conservation/rehabilitation interventions. In addition, these roads facilitate the management of rainwater run-off and contribute to improved road safety (Montel 2016).

2.1.1.1. Flexible pavements

A flexible pavement is a road structure made up of several layers of materials whose rigidity decreases from the surface to the base. It is able to deform under the effect of traffic loads without cracking, thanks to its composition of upper layers of bituminous mixes (wearing course and binder course) and lower layers of unbound granular materials (granular base and sub-base), resting on the road base (Monte 2012). This type of pavement is characterized by its considerable deformability under loads, as a result of being made up of layers with a bituminous asphalt mixture.

These superimposed layers define the structure of the flexible pavement, a cross-section of which is illustrated in Figure 1.

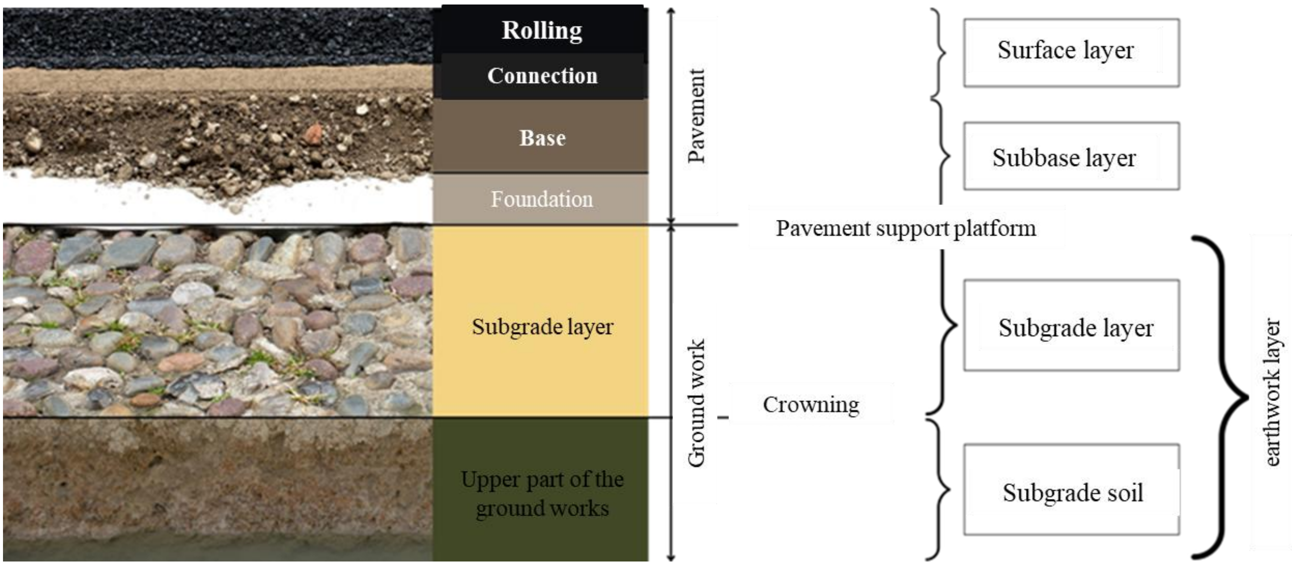


Figure 1. Cross-section of a flexible pavement

Each layer is described below, starting at the surface and working downwards:

- **Surface course**

Also often called as "wearing course", this is the top layer in direct contact with vehicle tires. It generally consists of a 4 to 6 cm thick asphalt bituminous mix. Its role is to ensure the comfort and safety of users, as well as the waterproofing of the structure.

- **Binder course**

Located just below of the surface course, it is also made of asphalt and is 4-6 to 12 cm thick. It provides the link between the wearing course and the base course.

- **Base layer**

This is the main layer that transmits the traffic loads to the pavement foundation. Its thickness varies from 10 to 20 cm. It is normally made up of simple aggregate materials and in some cases, can be treated with hydraulic binders or asphalt bitumen.

- **Foundation layer**

It distributes the loads on the subgrade. It is 20 cm to 1.0 m thick. It generally consists of untreated or treated gravel.

– **Shape layer**

This is the last layer before the natural subsoil. It homogenizes the bearing capacity of the soil and protects the upper layers. Its thickness varies from 30 to 50 cm. This multi-layer structure ensures good stress distribution and elastic deformation under vehicle traffic.

2.1.1.2. Rigid pavements

A rigid pavement refers to a type of pavement where the surface layer is made of concrete (usually Portland cement concrete). These pavements are much more rigid than the previous one (10 times more) and their rely stiffness results from of the concrete slab with

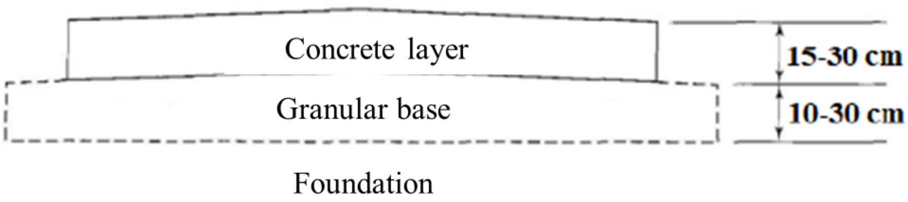


Figure 2. Cross-section of a rigid pavement. Source: Adapted from Pavement Analysis and Design (Huang 2004).

allows to distribute loads over a wider area (CEPSA ,2014), as shown in Figure 2.

The characteristics and properties of the layers are described below:

- The surface layer, known as the concrete slab, plays a crucial role in this type of pavement. It serves as the driving surface while directly supporting the loads imposed by traffic. The concrete used for this layer provides high compressive strength, making it very durable. It can support heavy loads while remaining less prone to deformation, even after years of use. The thickness of this slab generally varies between 15 and 30 cm, depending on the expected traffic intensity. (CEPSA, 2014).
- Base layer, composed of granular materials or cement-treated materials, this layer supports the slab by providing a stable and even surface for load distribution. To ensure the stability of the structure, this layer must be well-compacted to avoid differential settlements that could compromise the integrity of the slab. (CEPSA, 2014).

Thus, the layers of the rigid pavement work in synergy to ensure a durable structure that can withstand significant loads, with effective force distribution across the entire surface.

2.1.1.3. Semi-rigid pavements

Semi-rigid pavements have a layer structure very similar to that of flexible pavements, but integrate a rigid cemented base layer (DNIT, 2006). This configuration, which combines a top layer of bituminous mix providing a safe and comfortable surface with a concrete layer offering a solid and resistant base, resulting in an ideal floor with characteristics of both rigid and flexible pavement. Due to its composition, this type of pavement is also the least economical among those mentioned. It is rarely used for new road constructions but is generally employed to rehabilitate existing rigid pavements (Huang 2004). The cross-section of this type of pavement is illustrated in Figure 3.

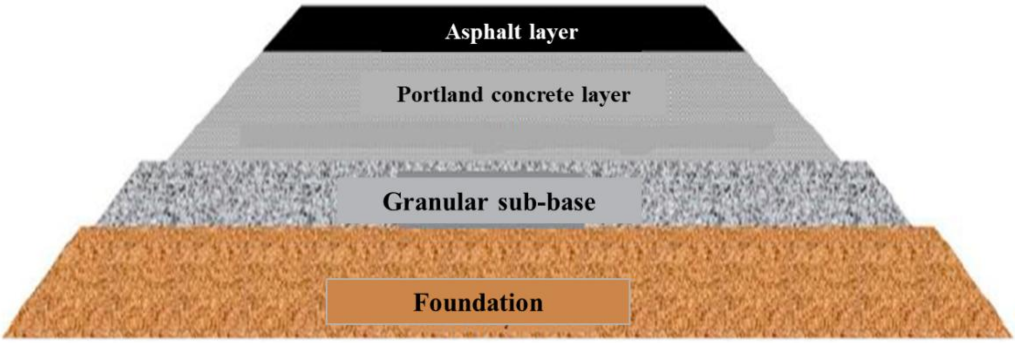


Figure 3. Cross-section of a semi-rigid pavements. (Plati 2019)

2.1.2. Unpaved roads

Unpaved roads, often called gravel or earth roads, are traffic lanes that do not have an asphalt or concrete surface. They are generally used in rural areas where traffic volumes are low. These roads play a crucial role in linking remote areas, facilitating the transport of agricultural and forestry products, and providing access to recreational areas. Because of their low construction and maintenance costs, they are preferred in many areas where paving is not economically viable. However, these roads require careful management, particularly to maintain their shape and ensure good drainage to minimize deterioration (Skorseth et Selim 2000).

A large proportion of these roads will remain unpaved due to very low traffic volumes and/or lack of funds to improve the sub-base and base before applying the pavement layer(s) (Skorseth et Selim 2000). It is also possible that these are the only roads that may exist in protected ecological or royal parks, where paved roads are not permitted as they are detrimental in some way to the local environment.

The figure below, taken from the International Road Federation database IRF (2024), shows the distribution of the total road network in the United States, distinguishing between paved and unpaved roads, over the period 2017 to 2020. The total road network remains stable at around 6.6 million kilometers each year. Paved roads make up the majority of the network, with around 4.4 million kilometers, while unpaved roads account for around 2.2 million kilometers, or about a third of the total. These proportions remain constant from year to year, suggesting that there has been no major change in the conversion from unpaved to paved roads. This stability in the data highlights the importance of unpaved roads in the overall network, particularly in rural and low-traffic areas. Paved roads (in light blue) make up the majority of the network, with around 18,000 kilometers, while unpaved roads (in black) account for a smaller share, of around 2,000 kilometers. There has been no significant change in the distribution of paved and unpaved roads over these five years, indicating stability in the country's road infrastructure.

This suggests that, as in the United States, unpaved roads remain a stable component of the network, probably in rural or low-traffic areas. This distribution highlights the need for specific maintenance strategies for these unpaved roads.

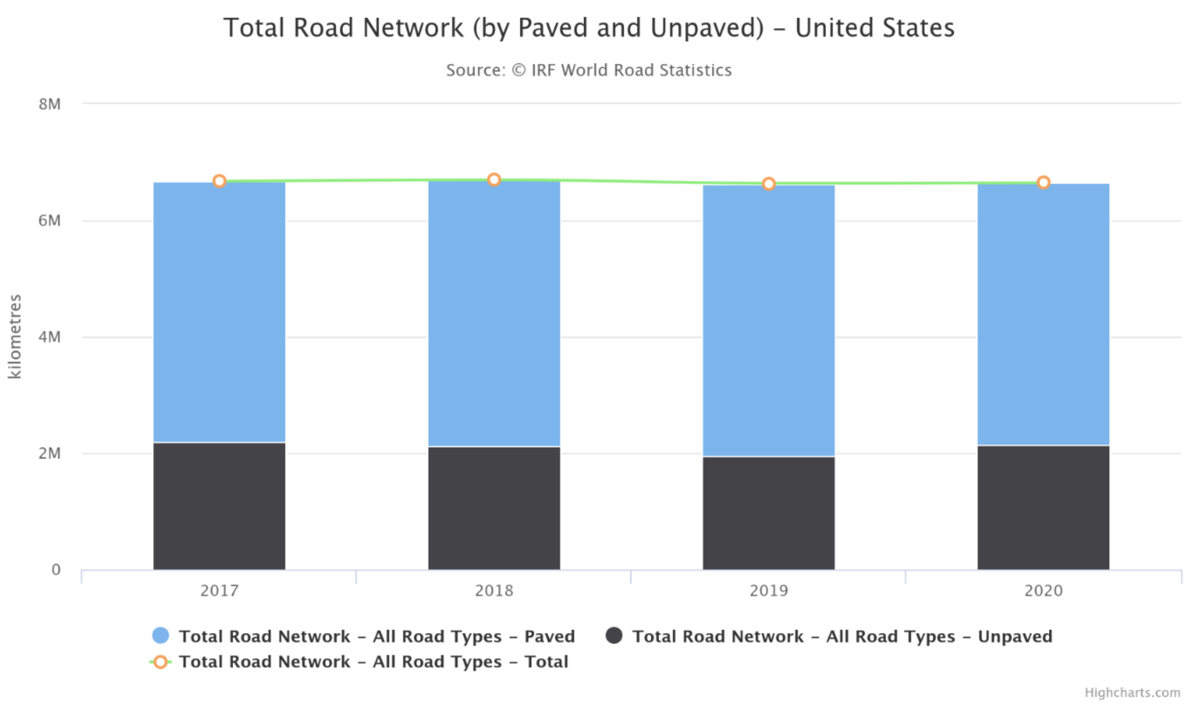


Figure 4: Temporal graph of the length of the road network in the United States
Source: Adapted from the IRF database website (IRF, 2024)

In developing countries, the percentage of kilometers of unpaved roads can reach lower values. One example is Tunisia which, according to official data from the International Road Federation database IRF (2022) the breakdown of the total road network in Tunisia between paved and unpaved roads, for the period 2017 to 2021. The total network is stable over the years, with around 20,000 kilometers of roads. Tunisia's road network expanded from 19,500 to 19,963 kilometers, with the proportion of paved roads increasing from 80% to 84.1%. This reflects efforts to modernize infrastructure and improve connectivity, particularly in rural areas.

The highway network also grew, from 562.4 kilometers in 2017 to 743.9 kilometers in 2021, strengthening transport infrastructure and facilitating travel.

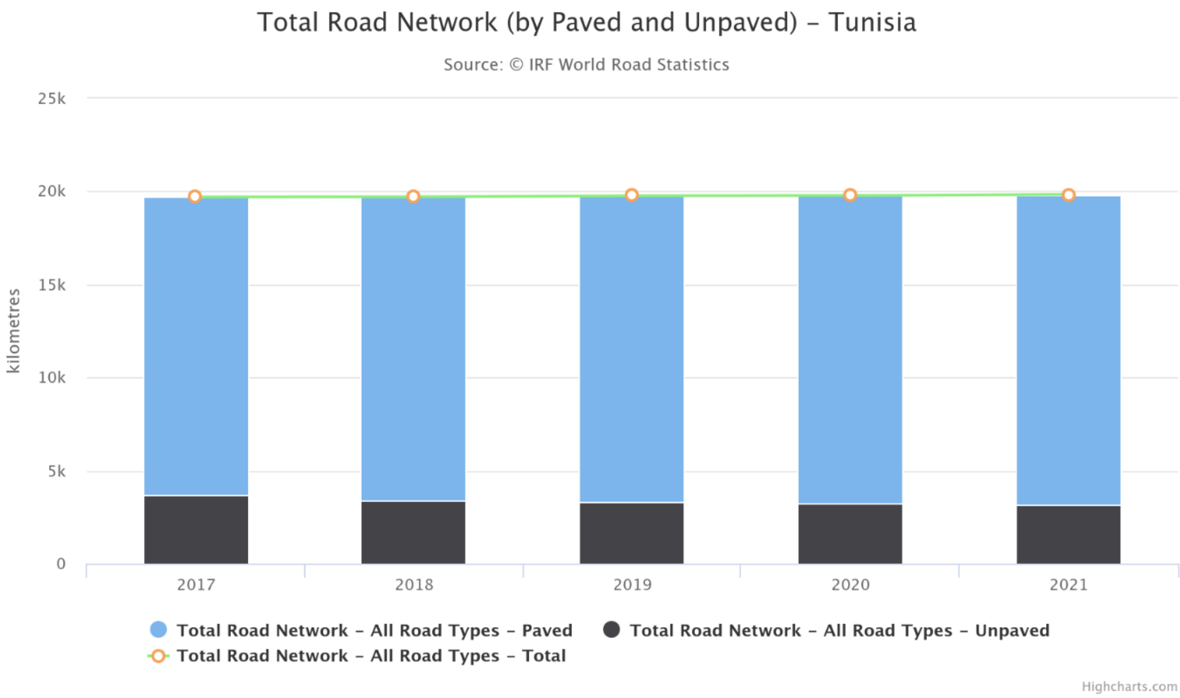


Figure 5: Temporal graph of the length of the road network in Tunisia
Source: Adapted from the IRF database website (IRF, 2024)

This information underlines the importance of unpaved roads to a nation, highlighting the need to preserve them, monitor them regularly through periodic assessments and rehabilitate them where necessary.

In terms of the popularity of this type of road, there is no significant difference between hot and cold regions. For example, in Tunisia, a country with a warm climate, a large part of the road network is made up of unpaved roads, often covered with gravel. Similarly, in cold regions, unpaved roads also predominate (Zubeck et Doré 2009). Typical cross-sections of these unpaved roads can be seen in Figure 6.

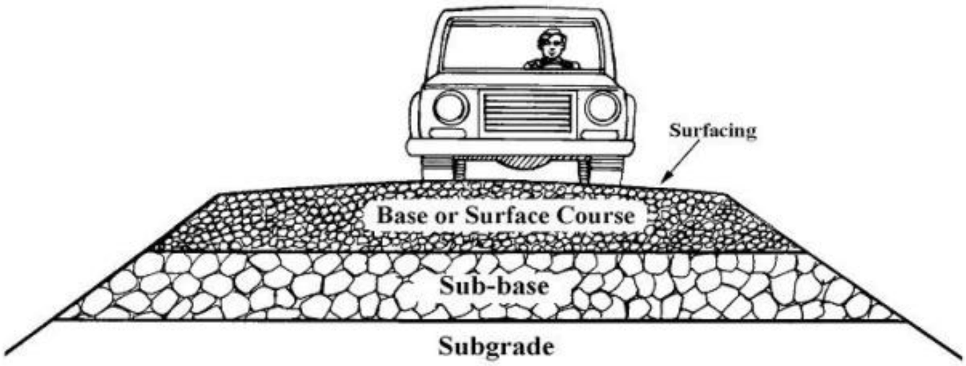


Figure 6. Road Structural Section (Keller et Sherar 2003).

- **Surface course (surfacing)**

This is the top layer of the road, also known as the wearing course. Various materials such as rocks, paving stones, crushed aggregates, as well as coatings such as bituminous surface treatments and asphalt concrete, can be used for this layer. These materials are intended to improve circulation comfort and safety, provide structural support and protect the road surface from bad weather, particularly during periods of rain (Keller et Sherar 2003).

- **Base, or surface course (Base)**

This is the main load-distributing layer to the pavement. It is generally made up of materials such as crushed stone or gravel, gravelly soils, decomposed rock, as well as sand and sandy clays which may, or not, be stabilized with cement, lime or bitumen (Keller et Sherar 2003).

- **Sub-base**

This is the secondary, load-distributing layer beneath the base. It is generally made of materials that are less resistant and durable than those used for the base, such as untreated natural gravel, a mixture of gravel and sand, or a mixture of gravel, sand and clay (Keller et Sherar 2003).

- **Subgrade**

This is the surface of the roadbed consisting of the foundation soil, on which the sub-base, base course or wearing course is constructed. For roads without a base course or wearing course, this section of the roadbed becomes the final wearing surface. It is generally located at the level of the material in place. (Keller and Sherar 2003)

2.2. Low volume roads (EBVT)

Low volume roads (LVVHs), also known as low traffic volume roads (LTVRs), play a crucial role in the road network, particularly in rural and remote areas, where they link smaller towns and serve agricultural or ecological areas. These roads, although exposed to potentially high loads, carry limited traffic, generally less than 300 daily vehicles average, or an equivalent standard number of axles of less than one million over a service life of around 15 years. The classification of these roads varies from region to region, with some manuals, such as Malawi's Ministry of Transport and Public Works, setting the limit at 300 vehicles per day, while others, such as the Southern African Transport and Communications Commission (SATCC), set it at 200. In developed countries, where a large proportion of roads are paved, manuals often provide a general approach to these roads, while in developing countries, where unpaved roads are more common, specific guidelines are provided to optimize their construction and maintenance. These unpaved roads are sensitive to climatic conditions and soils types, making their functional and structural assessment crucial. Evaluation methods, such as the Global Quality Index (GQI), for example, are used to analyse their surface condition, while geotechnical tests, such as the dynamic CBR and and tests with plates (dynamic or static), assess their load-bearing capacity, thus ensuring sustainable and optimum management of these essential infrastructures.

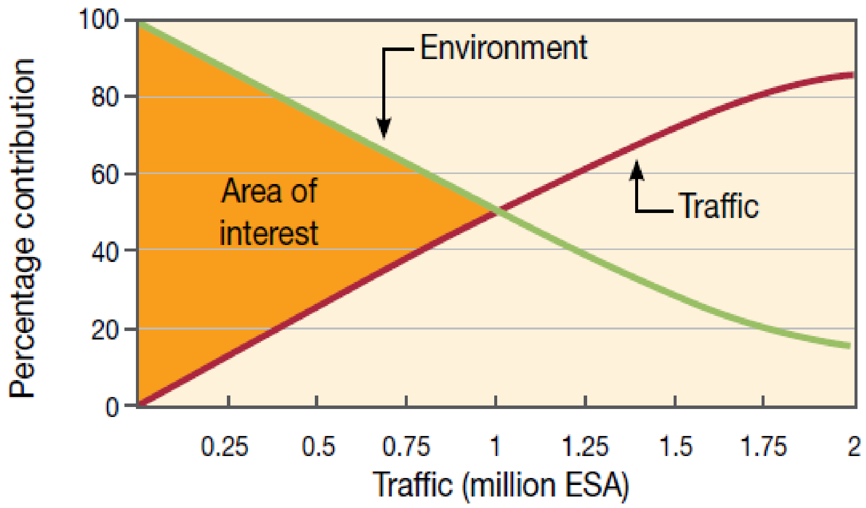


Figure 7. Influence of the environment and traffic on the number of standard axles
Source: Adapted from Design Manual for Low Volume Roads - Part A (ERA, 2016)

The Ethiopian Roads Authority (ERA, 2016), in its design manual for low-traffic roads in Ethiopia, defines these roads as simply being designed to carry low volumes of traffic. The manual states that the traffic threshold used for this definition is relatively arbitrary, as the characteristics that define a low-traffic road evolve progressively with increasing design traffic. However, information is provided about the traffic values that set the parameters for this type of road. The manual states that, to classify a road as low traffic, the cumulative volume of equivalent standard axles (of 80 kN) must be less than 1 million over a service life of 15 years. In addition, an annual daily average of 300 vehicles (equipped with four or more wheels) is estimated for half of this lifetime (ERA, 2016).

In the ERA manual2 (2016), a distinction is also made between factors influencing EBVT outcomes and performance, categorizing them into controllable and uncontrollable factors.

Controllable factors include elements such as traffic, construction management and road maintenance management. The latter is supported by manuals, such as those mentioned above, which provide good practice and a variety of options adapted to specific situations. This gives those responsible for the design, construction and monitoring of the road greater flexibility and decision-making capacity.

On the other hand, uncontrollable factors include elements such as climatic conditions, the hydrology of the surface and foundation soil, the type of soil used as a base, the morphology of the terrain and even the materials available nearby for road construction.

Given that the environment is the most decisive condition for the useful life of the road, from start of use to reaching one million standard axle passes, as shown in Figure 7, the choice of materials and the initial composition of the road are of paramount importance. This helps to minimize the impact of uncontrollable factors on road deterioration. In addition, it is essential that post-construction monitoring procedures and future rehabilitation interventions are adapted to the regional specificities associated with these factors, to ensure that interventions are both cost-effective and appropriate to the conditions of the road concerned.

2.3. Monitoring unpaved roads

Once a road has been built, whether paved or unpaved, it is essential to monitor it regularly. Paved roads have more costly and complex structures, requiring careful monitoring to intervene quickly in the event of pathologies or problems on the surface or in the structure. The first signs of deterioration on this type of road can sometimes be treated with simple interventions. However, if these problems are detected too late, the deterioration can reach a critical stage, requiring partial or total reconstruction of the pavement's layers. Such a situation not only entails higher rehabilitation costs, but also longer repair times, which can cause significant disruption to traffic.

In the case of unpaved roads, regular checks are also essential. In addition to the impact of traffic, climatic conditions, such as seasonal variations, temperature fluctuations and periods of rain of varying intensity, play a major role in their deterioration (ERA, 2016). Surface layers, whether gravel or local soils, are particularly vulnerable to these conditions, which can make it difficult for vehicles to travel and compromise the functionality of the road.

Roads, whether paved or unpaved, are assessed using two types of analysis: structural assessment and functional assessment. These analyses provide crucial information that can be used to determine whether conservation/rehabilitation needs to be done, and to what extent. Thanks to these assessments, it is possible to tailor maintenance strategies to the specific needs of each road, ensuring more efficient and cost-effective management.

2.3.1. Structural assessment

Structural assessment, also known as mechanical assessment, involves a number of geotechnical tests, particularly bearing capacity compaction tests, carried out either in the laboratory or directly on the study site (field tests).

Laboratory tests are useful for obtaining detailed information about soil properties. To do this, soil samples are taken on site, following strict sampling procedures to ensure that they are as representative as possible. However, despite all the care taken in this collection, the samples may undergo modifications, and thus not perfectly reflect the actual state of the soil on site, introducing a margin of inaccuracy in the results obtained (Marques, 2015). Nevertheless, these tests provide essential data and key parameters that, when correlated, allow a better understanding of the characteristics of the soil studied (Caputo 2022).

Field tests, on the other hand, offer the advantage of being carried out directly on site, under actual soil conditions. These tests are therefore faster and more accurate, because the soil is not altered by the sampling process. They are also characterized by their objectivity: the tests are carried out using specialized equipment and standardized procedures, which reduces the variability of the results. This standardization guarantees greater reliability, with minimal or non-existent differences between the results obtained by different assessors carrying out the same types of tests.

It is important to note that the data from this assessment should not only be collected during the road design phase. For example, values relating to the bearing capacity of the subgrade can also be very valuable after construction. In fact, in the post-construction phase, these assessments make it possible to measure the mechanical state of the road after a period of exposure to traffic and climatic conditions. In this way, they provide an indication of the degree of deterioration of the soil, which is essential for planning any maintenance or rehabilitation work that may be required. This approach ensures that the road remains in good condition throughout its lifetime, by adapting maintenance work to the actual conditions encountered in the field.

2.3.1.1. Proctor (laboratory test)

Soil compaction is mainly assessed by determining the maximum dry density (MDD) and optimum moisture content (OMC), parameters that are crucial for the construction of structures such as motorways, bridges and buildings. The compaction test most commonly used to measure these parameters is the Proctor test, available in standard and modified versions. This test determines the maximum dry density (DMS) of the soil as well as the optimum water content (OWC) for it to reach its most compacted state under a given numbers of blows. These soil compaction characteristics are essential for soil improvement work, such as the construction of motorway embankments, railways or earth structures. In the laboratory, determination of the DSM makes it possible to compact soil in the field to the required density while maintaining a near-optimum water content (Li et al. 2024).

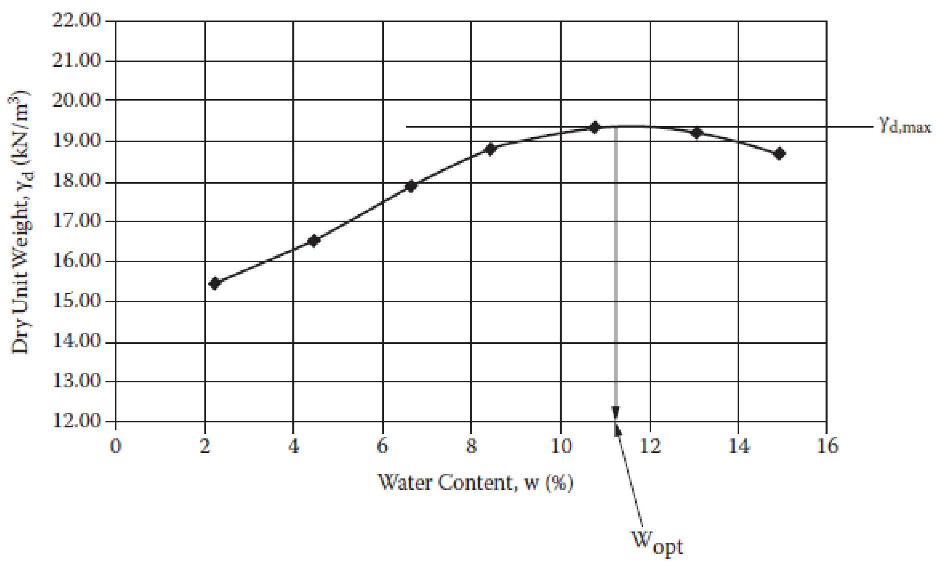


Figure 8. Example compaction curve (Ishibashi et Hazarika 2015).

2.3.1.2. California Bearing Ratio - CBR

The CBR test is a penetration test used to assess the mechanical strength of road sub-base and base courses. Developed by the **California Department of Transportation**, the test measures the pressure required to penetrate a soil sample using a standard surface plunger. The values obtained are compared with those of a reference material, crushed limestone, to determine the percentage of CBR. (CBR Pavement Design).

The California Bearing Ratio (CBR) test was carried out in accordance with the **ASTM D-1883-07 standard [36]**, Figure shows some of the test phases.

a. Soil with optimum water content

b. Compaction

c. Immersion of the compacted soil specimens in water.

d. CBR test equipment

Figure 9. Laboratory procedures for the CBR test.

In summary, the test begins by determining the optimum moisture content and specific dry weight of the material, using the Proctor method. Next, the expansive properties of the material and the CBR (California Bearing Ratio) are assessed. For the latter, the relationship between the stress measured during the test and the reference stress for standard crushed stone is established using equation 1.

CBR = (Measured force / Standard force) * 100 **[1]**

CBR in situ

The in situ CBR test was carried out in accordance with BS 1377:9 [44], and some of the test steps are shown in Figure.

a. Attaching the equipment to the reaction vehicule

b. Equipment ready for the test.

c. Axial force and piston penetration measuring device

d. A Presentation of the soil after testing.

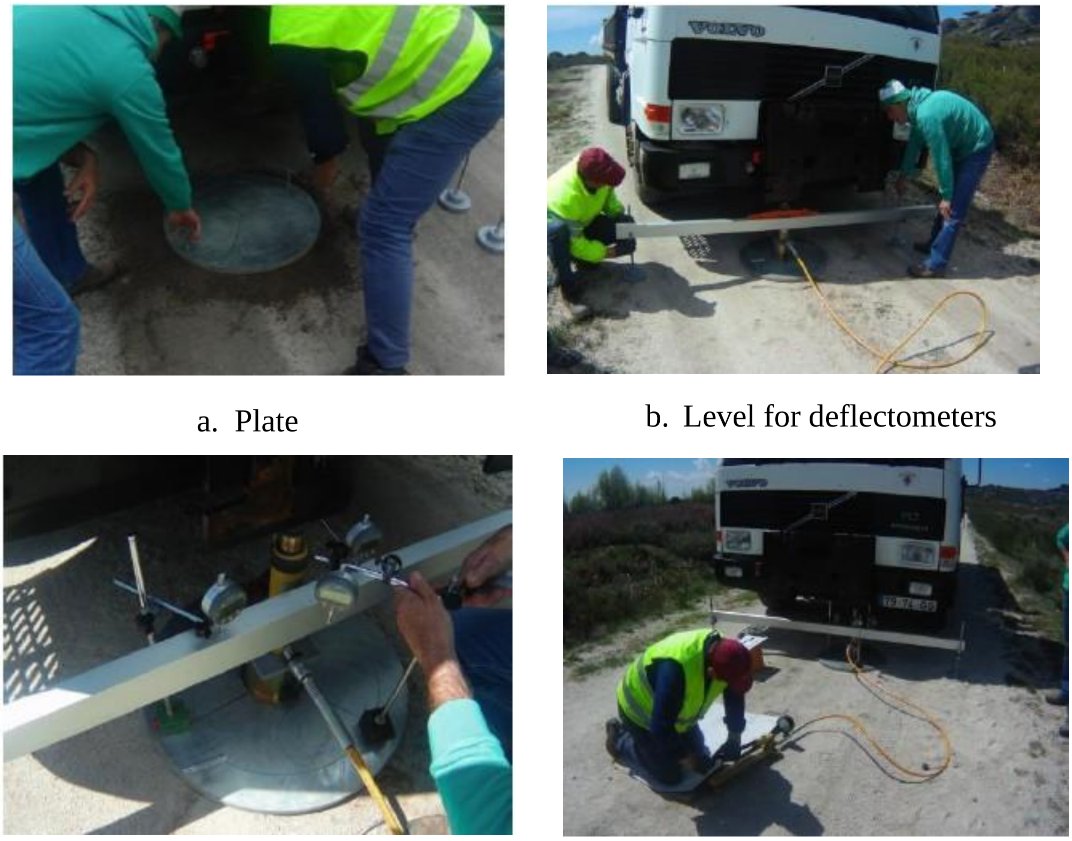
Figure 10. In situ CBR test. (Cabette 2019).

2.3.1.3. Plate load test - ECP (field test)

The Plate Load Test (PLT), initially designed to assess soil behavior under heavy loads representing the actual conditions encountered in building foundations, has gradually been adopted in various civil engineering disciplines. Today, it is widely used to characterize not only road foundation soils, but also other granular layers. According to (Kraemer, Pardillo, et Rocci 2004) this test has become an essential tool for the evaluation of unpaved roads with low traffic volumes, in particular for measuring the mechanical properties of soils subjected to traffic loads.

The plate load test was carried out based on **the NF P94-117-1 standard [41]**. The test procedure involves applying a vertical load to a 0.6 metre plate and measuring the penetration to determine the modulus of deformability.

As shown in Figure, a hydraulic jack and a reaction truck are needed to apply the vertical load and the resulting deflection of the surface results from the reading of three points, distributed over 120 degrees of the deflectometers. With the average deflection, the modulus of deformability is calculated.



a. Plate

b. Level for deflectometers

c. Digital deflectometers

d. Hydraulic Jack

Figure 11. Plat load test equipment(Cabette 2019).

The test protocol includes two cycles of loading and unloading, during which the deformation of the soil is recorded. When a load is applied, the soil undergoes a deformation that stabilizes rapidly. However, during unloading, some of the deformation is recovered, illustrating the elastic component of the soil's behavior. This process of double loading and unloading provides a better understanding of the soil's capacity to recover under stress and enables its rigidity to be measured more accurately (Kraemer, Pardillo, et Rocci 2004).

After these cycles, the recorded strains are analyzed using the Boussinesq formula (equation 2), which relates the applied vertical stress, the plate diameter and the average strain. This approach is used to calculate the modulus of elasticity of the soil. However, in this particular case, the modulus is referred to as the deformability modulus (Ev2), because the test does not only take into account the purely elastic behaviour of the soil, but also its response to permanent deformations (Kraemer, Pardillo, et Rocci 2004). This parameter is crucial for assessing the bearing capacity and durability of soils under repeated loads, such as those encountered in road infrastructures.

$$\boldsymbol{Ev2\ (MPa)} = 0{,}75 * d * q/s \qquad \textbf{[2]}$$

Where:

d - Diameter of the metal plate (m)

q - Applied vertical stress (MPa)

s - Average deformation (m)

The main limitation of the plate load test lies in the reaction system used. It often requires heavy vehicles with sufficient mass to apply the desired load to the ground, which can be an obstacle to carrying out the test. Finding a suitable vehicle and having the necessary space on the site to carry out the test are important constraints that can lead to the exploration of other test methods, thus sometimes ruling out the use of this technique (Kraemer, Pardillo, et Rocci 2004).

The modulus of deformability measured during this test is directly linked to the bearing capacity of the soil to a certain depth, encompassing the pressure bulb. This data is crucial for assessing the load-bearing capacity of road foundations, as it enables us to determine the soil's resistance to bearing the loads imposed by traffic.

2.3.1.4. LFWD Test

To assess the structure of the road, tests were carried out using Zorn Instruments' ZFG 3.0 Light Weight Deflectometer (LWD). The equipment has a mobile mass of 15 kg, which falls from a height of 67 cm, already discounting the height of the mass. This generates a maximum force of 10.6 kN, which is distributed on the ground via its 300 mm diameter load plate. From this, a maximum pressure of 0.15 MN/m² can be generated in the soil (Zorn, 2016).

The equipment provides us with data such as the pavement deformation (s, in mm), the dynamic modulus of elasticity (Evd, in MN/m²), the impact velocity (v, in mm/s) and the geographical coordinates of the location.

To carry out the test with the LFWD, the following steps were followed, according to **ASTM standard E2835-11**(2015):

- Clean the surface of larger granules or protruding materials.
- Position the loading plate on the surface.
- Rotate the plate 45 degrees to the right and left.
- Place the rest of the equipment on the plate.
- Connect the sensor cable to the reading device.
- Position the mobile mass on the release device.
- Check the verticality of the device using the water bubble.
- Release the mobile mass in free fall and hold it before it falls a second time.
- Repeat the last 3 steps 6 times.

Deformation and velocity data should only be read for the last three impacts, as the first three are used to settle the soil beneath the equipment and the average of these values is used to calculate the dynamic modulus of elasticity.

The tests should be repeated at another location, according to the ASTM (2015) standard, if: the plate has a slope of more than 4%; the deflections have a difference of more than +/- 10%; there is a faulty drop, or the equipment has moved.

Figure 12 illustrates the step-by-step procedure used.

Figure 12. LFWD test sequence

2.3.1.5. Correlation between tests

An essential aspect of the various tests presented is the need to establish correlations between them, whether through physical, mathematical or experimental relationships. These correlations make it possible to generalise soil parameters, thus offering researchers, operators, technicians or companies responsible for the study the possibility of converting data collected with one type of equipment into other useful units or parameters. This is particularly advantageous when a specific situation allows the use of only one of the available pieces of equipment. Thanks to these correlations, it becomes possible to apply road design methods that were previously unthinkable due to the different parameters required by the various methodologies.

These correlations are not only limited to design methods, but can also be applied to other parameters, methodologies or good practices that depend on the results of these tests. Numerous studies and articles have investigated the relationship between parameters commonly used in road design, such as CBR and the deformability modulus (Ev2), which are among the most popular and play a central role in the decision-making of those responsible for the works.

A well-known equation that seeks to establish a relationship between these two parameters for foundation soils can be found in the Interim Advice Note (IAN), a document regularly updated by UK organisations such as the Highways Agency (Highways Agency et al., 2009). Equation 3, shown below, has been validated for CBR values between 2% and 12%, providing a sound basis for interpreting test results interchangeably.

$$Ev2\ (MPa) = 17{,}6 * (CBR)^{0.64} \qquad \textbf{[3]}$$

A simpler relationship is proposed in the Shell method (Kraemer, Pardillo, and Rocci 2004), which is widely recognised and well established in the field of transport infrastructure for road design. This method expresses the modulus of soil deformability (Ev2) as a function of the CBR value, as shown in equation 4 below.

$$Ev2\ (MPa) = 10 * CBR \qquad \textbf{[4]}$$

Since Dynamic CBR (CBRd) and Dynamic Deformability Modulus (Evd) are tests based on more recent procedures and equipment, there is still little research and in-depth studies on their results. However, ZORN's (2016) LWD equipment manual proposes interesting correlations between the classical deformability modulus (Ev2) and these two new parameters, obtained using LWD equipment. These correlations are presented in the form of abacuses, as shown in Figure 13, and make it possible to establish practical links between these values. This represents an important advance, as these relationships make it easier to interpret the results of recent tests within the framework of more established methods, thus facilitating their integration into road design and infrastructure quality control projects (Kolodi 2023).

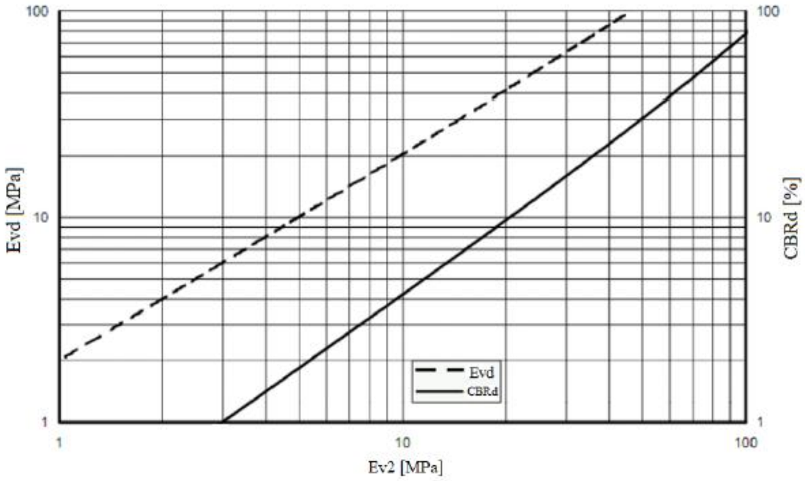


Figure 13. Abacus of correlations between dynamic parameters and modulus of deformability (Kolodi 2023).

This informations presented in the manual is subject to change, as the company points out, due to the diversity of granulometries and soil qualities. This variability can introduce further discrepancies in the results obtained by the equipment, resulting in undesirable fluctuations that can affect the accurate assessment of soil characteristics on site.

2.3.2. Degradation of unpaved roads

The second method for assessing the condition of roads, whether surfaced or not, is to carry out a functional assessment, i.e., to analyse the condition of the road surface and its drainage capacity, which has a direct influence on its performance. These aspects, combined with the load capacity and mechanical characteristics determined during the structural assessment, help to establish the level of deterioration of the pavement. The functional assessment mainly aims to examine the condition of the road surface, where most pathologies are concentrated, through visual observations. These observations, which are often translated into notes or concepts, make it possible to identify the condition of the road from the user's point of view as he drives along. The experience of driving on the road is directly correlated with the degree of deterioration and its severity (Bernucci et al. 2007).

The condition of the road surface has a direct impact not only on user comfort but also on vehicle maintenance. A degraded or uneven road places additional strain on vehicles, leading to faster wear of mechanical parts, such as suspension systems. This results in higher maintenance costs, increased fuel and tire expenses, and longer travel times due to a less smooth and uniform surface. All these factors contribute to a rise in vehicle operating costs, meaning the expenses borne by the user for travel. This highlights that road conditions affect not only travel comfort and speed but also the overall economy of the transportation system. Additionally, poorly maintained roads can indirectly impact individuals' purchasing power, as increased vehicle operating costs, particularly for vehicles used to transport goods, are reflected in the prices of those goods (Bernucci et al. 2007).

Traffic actions, whether combined with climatic actions or not, can degrade the road surface if it has not been prepared with appropriate material and/or lacks adequate drainage conditions. Alzubaidi et Magnusson 2002 developed a schematic diagram of this issue, as shown in Figure 14.

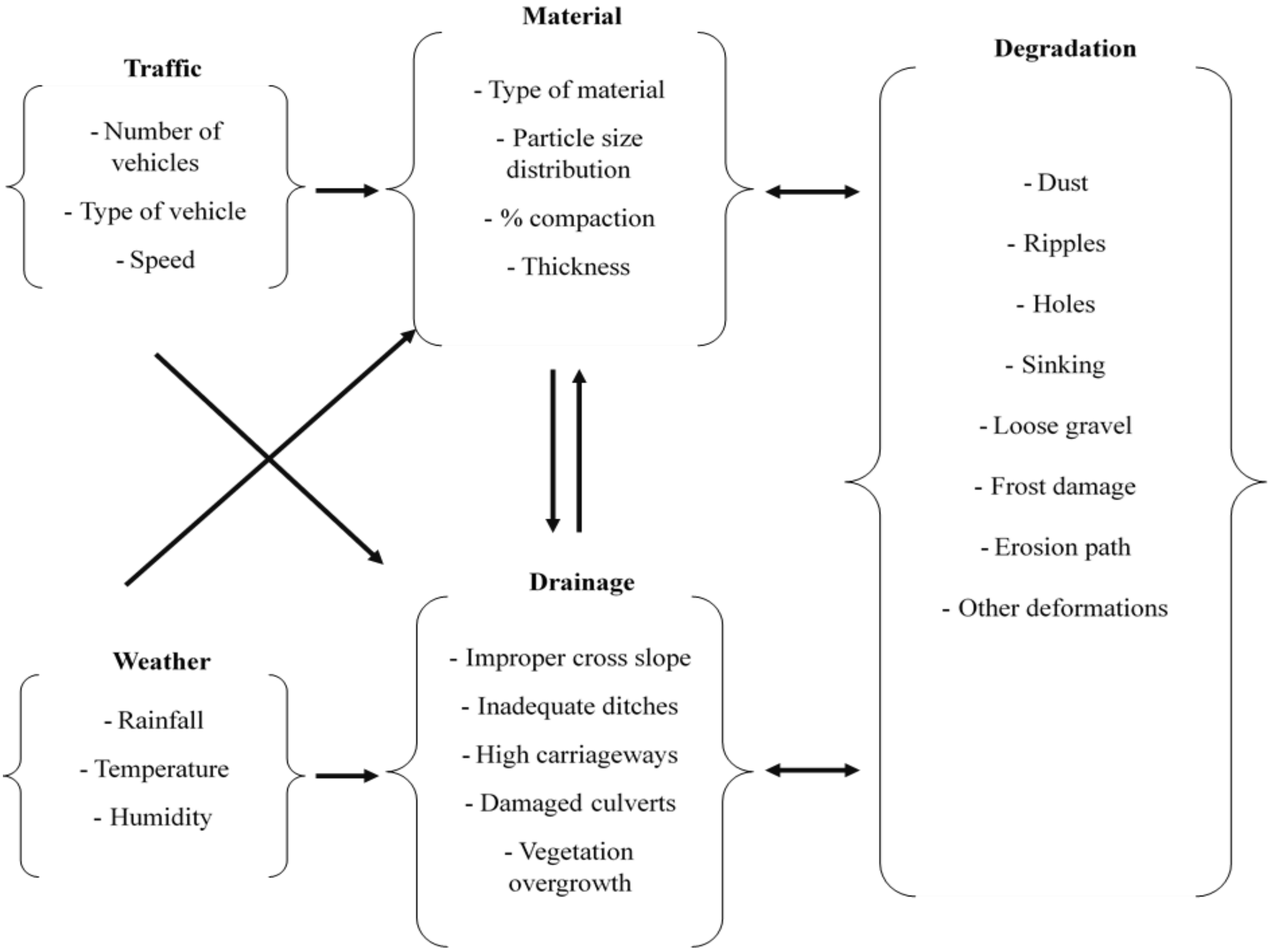


Figure 14. Schematic diagram of the degradation process of unpaved roads (Alzubaidi et Magnusson 2002).

2.3.2.1. Rutting

A longitudinal depression formed on the road surface, specifically in the areas where vehicle wheels pass. This type of deformation is caused by the permanent displacement of materials from one or more layers of the road, often due to repeated traffic loads. Rutting occurs primarily when the road's bearing capacity is insufficient, especially during rainy periods, which exacerbates soil deformation due to moisture and vehicle passage.

Figure 15. Example of subsidence on the unpaved EBVT at Montesinho (Kolodi 2023)

The appearance of ruts is worsened by ineffective drainage. Ruts create depressions where rainwater stagnates, preventing its natural flow toward the drainage system (ditches, culverts) and thereby accelerating road degradation. This phenomenon also promotes the formation of other issues, such as potholes, as accumulated water further deteriorates the surface and underlying layers.

The depth and severity of ruts vary according to weather conditions and traffic intensity. In dry periods, rut depth increases due to soil compaction by heavy vehicles, while in wet seasons, light vehicles generally avoid these tracks, limiting their immediate worsening, but trucks continue to compact them further. Ruts develop more quickly on roads composed of coarse materials like quartz or laterite, while those covered with clay are more resistant. The presence of slopes also helps reduce their severity by facilitating natural water drainage (Patologias das estradas…).

2.3.2.2. Corrugation

Corrugations are transverse deformations on a road surface caused by the repeated passage of vehicles. They appear as waves or grooves perpendicular to the direction of traffic, resulting from the displacement of loose surface aggregates. This phenomenon is triggered by traffic actions, especially in high-traffic areas like intersections, or in sections where vehicles frequently accelerate and decelerate. This repeated wear deteriorates the surface granular layer, contributing to the formation of corrugations.

These corrugations primarily form on roads where the surface consists of granular materials with low plasticity and small particle size (less than 5 mm), with few fine particles. The loss of these fines due to traffic further worsens the phenomenon. They typically occur under heavy traffic conditions, when the speed and weight of vehicles cause a displacement of loose surface material, creating waves with alternating ridges and troughs.

Figure 16. Photograph of an unpaved road with a corrugated surface (Andrews, Mathias, et Austroads 2009)

Corrugations are particularly problematic as they affect ride quality, reduce user comfort, and accelerate vehicle wear, especially on suspension systems.

2.3.2.3. Potholes

Potholes are localized deformations, either small or large in radius, that form when surface material is displaced due to traffic forces. Their development is accelerated by the infiltration and accumulation of water within the hole, which promotes the progressive disintegration of the road. Suspended materials are easily removed with each passing vehicle, further weakening the structure. This ongoing degradation results from both the loss of surface material and the formation of weak points in the underlying soil, worsening the road's instability. (Types of Defects in Unpaved Roads – Bibliography)

Figure 17. Photograph of an unpaved road with potholes on its surface (Australian Road Research Board. 2009)

2.3.2.4. Dust

Dust emitted by unpaved roads creates safety and environmental issues, particularly through dust clouds formed as vehicles pass. This dust results from the wear of the finest gravel particles caused by traffic, and the amount released depends on several factors, including vehicle speed and volume, road composition and compaction, material durability, and climate. When vehicles pass, two phenomena lead to the formation of these dust clouds: the first is the detachment of fine particles (diameters less than 0.425 millimeters) from the surface layer, and the second is the disturbance caused by vehicle actions or climate effects (Andrews, Mathias, et Austroads 2009). For an unpaved road, the first sign of wear is the loss of surface fines, manifested by dust clouds and noise as vehicles pass. This changes the surface's particle size distribution, making it less homogeneous, with a grading curve showing fewer fine particles, leaving only coarser soil grains (Andrews, Mathias, et Austroads 2009).

This degradation is directly related to the health and safety of road users, as at advanced levels with high severity, dust clouds can obstruct drivers' visibility if two or more vehicles are in sequence, potentially triggering accidents. Additionally, raised particles are harmful to the respiratory system and may cause discomfort to the eyes of those exposed (ERA, 2016).

2.3.2.5. Loose Aggregates and Gravels

The loss of fine aggregates can lead to an excessive amount of loose gravel on the surface. Under the influence of traffic, this loose gravel may tend to accumulate between wheel paths and along the sides of the road, creating a driving hazard and reducing drainage capacity (Alzubaidi et Magnusson 2002).

Figure 18. Photograph of an unpaved road with loose aggregate on its surface (Australian Road Research Board. 2009)

2.3.2.6. Insufficient Drainage Capacity (Incorrect Geometry)

It is essential to construct pavements with a cross slope of approximately 3 to 6%, along with an effective drainage system (Jorge 2014). Figure 19 illustrates examples of correct and incorrect cross-sections.

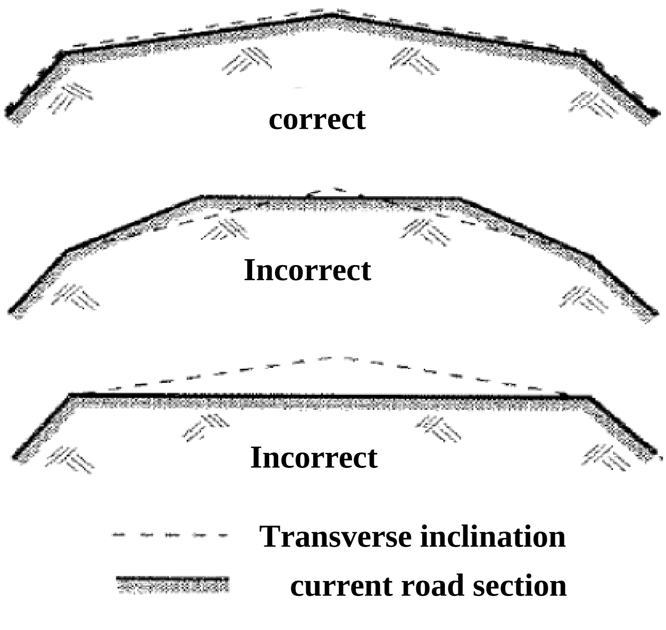


Figure 19. Examples of correct and incorrect slopes (Australian Road Research Board. 2009) and (Alzubaidi 1999).

The slope is designed to allow water to flow by gravity towards the shoulders and the pavement. The steeper the surface, the more quickly water will flow towards the sides. If the slope is inadequate, water will stagnate, creating puddles and, consequently, erosion. Wear gradually decreases the slope of the road. This wear is partly due to the transformation of finer materials into dust that disperses, while some coarser materials are moved by vehicle wheels towards the sides. If the slope is excessive, water flow will be insufficient, exposing the road to other types of damage. Additionally, overload can deform the road: if the load-bearing capacity is insufficient, the passage of very heavy vehicles can reduce the slope of the cross-section (Alzubaidi et Magnusson., 2002).

Wear and vegetation growth can obstruct the ditches on the shoulders, in addition to harming the geometry of the ditches. Figure 20 illustrates the appropriate geometry of a road for proper functioning.

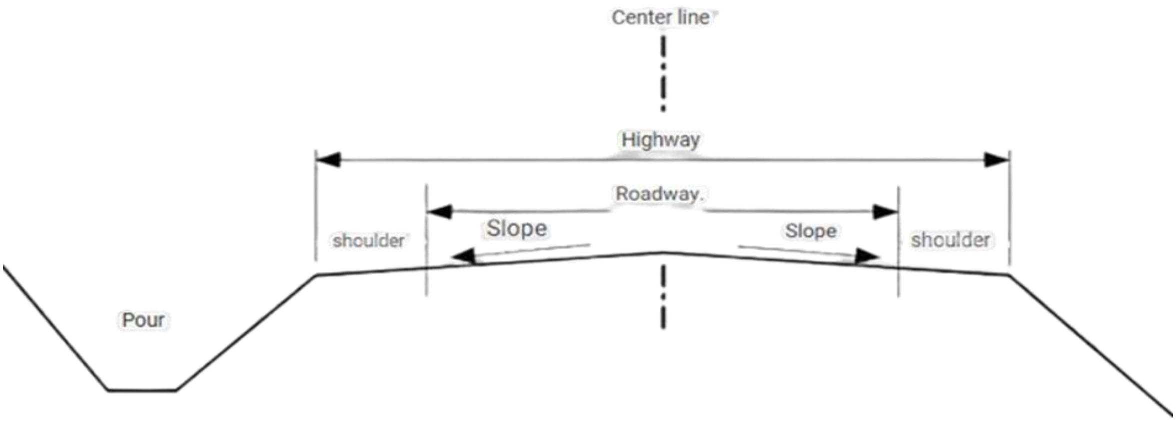


Figure 20. Correct geometry of a road profile.

If this occurs, it prevents water from flowing into the ditch and down onto the road. On slopes, water increases its speed and can cause erosion. If water remains on the road surface, potholes may form, as moisture somewhat reduces the load-bearing capacity of the road (Alzubaidi et Magnusson., 2002). In addition to potholes, water on the road surface can lead to hydroplaning, which can result in serious accidents for road users.

2.3.2.7. Erosion (scouring)

Erosion is an entirely natural process, independent of the action of traffic. It corresponds to the loss of surface material under the effect of water flow, which can occur on or along the road. This phenomenon can be caused by an excessive accumulation of gravel, loose aggregate or vegetation in drainage systems, preventing water from draining properly into ditches and stagnating on the road surface. Erosion can also result from insufficient compaction, making grains more vulnerable to being carried away by water, or from excessive irregularity of the longitudinal slope, which accelerates the flow of water towards the ditches and drags grains to the surface (Australian Road Research Board. 2009; Andrews, Mathias, et Austroads 2009)

Erosion can be transverse or longitudinal. Transverse erosion generally occurs when the level of compaction is low and extends towards the shoulder of the road, or when ditches are blocked, forcing water to divert its path, as mentioned previously (Figure 21). Longitudinal erosion, on the other hand, occurs when the main slope is oriented towards the road and has significant gradients, forcing water to flow along the carriageway rather than into the side ditches (Australian Road Research Board. 2009; Andrews, Mathias, et Austroads 2009)

Figure 21. Transverse erosion on the unpaved EBVT at Montesinho in 2023 (Kolodi 2023)

2.3.2.8. Drainage system

The main drainage systems found along a road are ditches, culverts and galleries. It is through these devices that water, which is responsible for some of the damage that can occur on a road, is collected and drained in such a way as not to compromise the structure of the road. It is therefore essential that they function correctly in order to preserve the integrity not only of an unpaved road, but also of paved roads.

Ditches are the elements that accompany the road in the longitudinal direction, receiving and channelling the water present on the road surface that tends to run off the sides of the road across the cross slope. Often, this device is sufficient to meet the need to channel the water without affecting the road, but there are cases where other devices are required, such as culverts and galleries placed at strategic points that require a greater effort to channel this water, situations that can be seen in Figure 22 (Ferreira., 2004).

Figure 22. Accumulated water next to the unpaved Montesinho EBVT in 2023 (Kolodi 2023)

For the ditch to function optimally, it is important to carry out maintenance and observations in order to monitor and intervene in the event of debris accumulation or vegetation growth at the bottom of the ditch. It should be pointed out that, especially on slopes, the presence of vegetation just above the ditches helps this system by slowing down the speed at which water from the slope reaches the ditch, and by totally or partially blocking debris that also comes from the sloping surface of the land and would fall into the drainage system.

2.3.3. Functional assessment

The functional evaluation of pavements aims to examine the surface condition of roads, taking into account how this condition affects ride comfort. It focuses on collecting data on surface degradation, such as potholes, erosion, depth of sags and undulations, measuring these degradations quantitatively, in addition to visual observations. (Ali., 2016 ; DNIT., 2006)

There are two main methods of functional assessment. The first is subjective: it is based on visual observation by qualified personnel who identify the type and severity of damage without the aid of measuring equipment. The second method, known as objective, combines visual observation with basic measurements, such as rulers and levels, to provide a more accurate description of the condition of the surface. Functional assessment is based on direct observations or impressions gathered when the vehicle passes over the segment under study, depending on the comfort perceived by the assessor (Brooks et al. 2016).

However, this assessment has limitations, as it can vary from one assessor to another, introducing variability into the records. To minimise these discrepancies, several standardised methods, such as the RCS/DVI, ERCI, URCI and RSMS methods, have been developed to provide a rating and ranking of deterioration based on precise criteria, incorporating the various road pathologies and drainage system characteristics. These methodologies help to reduce uncertainty and subjectivity by using standardised criteria for the functional assessment of unpaved roads.

2.3.3.1. AASHTO method

The evaluation method developed by AASHTO is one of the established methods for the subjective functional evaluation of a pavement section. It assigns a numerical score, known as the current serviceability value, on a scale of 0 to 5 (Table 2). This rating corresponds to the average of the assessments made by several observers, who rate the ride comfort of a vehicle travelling on a given section, at a given moment in the pavement's service life (Bernucci et al. 2007; DNIT 2006).

Table 2. Utility assessment (DNIT 2006)

Concepts	Limits
Excellent	4 to 5
Good	3 to 4
Regular	2 to 3
Bad	1 to 2
Very bad	0 to 1

2.3.3.2. Overall severity index (IGG) method

The objective method, defined in DNIT 006/2003-PRO (DNIT 2003), describes the process of inventorying and classifying visible degradations and permanent deformations of wheel tracks (measured using a ruler). It proposes a system for calculating a combined failure index, known as the Global Severity Index (IGG), which works in a similar way to the fitness-for-service method, as illustrated in Table 3.

Table 3. Overall severity index (DNIT 2003)

Concepts	**Limits**
Excellent	$0 < IGG \leq 20$
Good	$20 < IGG \leq 40$
Regular	$40 < IGG \leq 80$
Bad	$80 < IGG \leq 160$
Very bad	$IGG > 160$

This index is calculated using equation 5.

$$IGG = \Sigma IGI \qquad \textbf{[5]}$$

Where IGI is the individual severity index, defined in equation 6.

$$IGI = fp \cdot fr \qquad \textbf{[6]}$$

Where fp is the damage weighting factor, provided in DNIT 006/2003-PRO (DNIT., 2003), and fr is the relative frequency, i.e., the number of times the damage was checked relative to the total number of stations. The relative frequency is defined by Equation 7.

$$fr = (fa \cdot 100/)n \qquad \textbf{[7]}$$

where fa is the absolute frequency, which corresponds to the number of times the damage was checked, and n is the number of stations inventoried.

Chapter III: Methodology

3. Design of a Low-Traffic Unpaved Road

Maintaining social distancing measures has become a priority to preserve the health and safety of all. As a result, the solutions explored in this study rely on data obtained from previous works by Cabette (2018) and Freitas (2019). These studies provided a solid foundation for applying internationally recognized design methods, such as those used in the United States, Australia, and the United Kingdom. According to Jorge (2014), low-traffic roads are defined by an annual average daily traffic (AADT) of less than 400 vehicles per day, with a speed limit below 80 km/h. Although the road studied did not undergo a specific traffic survey, Cabette (2018) observed low traffic volumes in the region, a finding confirmed by Freitas, who used the lowest traffic class described in the ERA Manual (2011). Both studies conducted laboratory and field tests to assess the geotechnical parameters of the materials composing the road. These results enabled the segmentation of the road into homogeneous sections, meaning segments with similar characteristics and, therefore, comparable mechanical behavior.

This study does not include the functional evaluation according to the PASER Manual (Walker, 2002). To ensure the reliability of the results, a complete analysis of the entire road would be required, as some classification parameters depend on the proportion of defects present along the entire route. However, in previous research, the data collection on defects only covered a limited portion of each subsection, and the defects along the full route were estimated based on these partial data. Similarly, the evaluation and design of the drainage system will not be addressed here, due to the lack of information regarding precipitation periods in the studied area.

3.1. Location of the Study

The low-traffic unpaved road (EBVT) analyzed in this study is located in the northeast of Portugal, within a mountainous region near the village of Montesinho, on the border with Spain. More specifically, it is situated within the Montesinho Natural Park, a protected area classified as a natural reserve.

The solutions proposed in this research take into account the preservation of the area's natural characteristics, thus excluding any possibility of adding an asphalt or concrete surface to the road. With a total length of 3.06 km, this road serves as the main access route to two dams located in the municipality of Bragança: the Veiguinhas Dam and the Serra Serrada Dam. The route begins at the coordinates Latitude 41° 57' 44" N and Longitude 6° 48' 28" W, and ends at Latitude 41° 56' 7" N and Longitude 6° 48' 50" W. Figure 23 provides an illustration of the road's location.

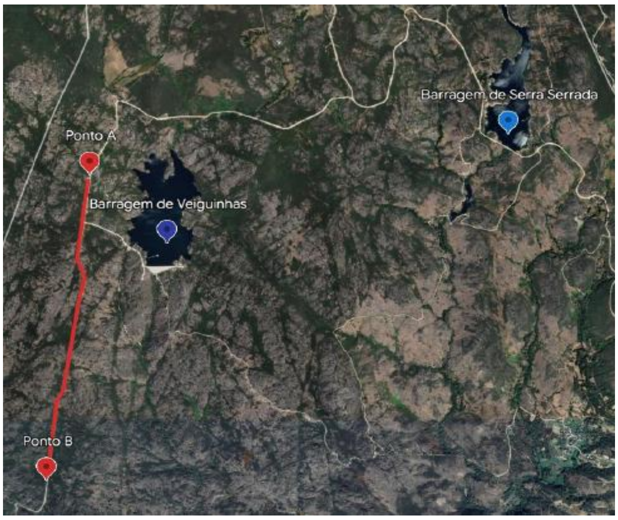


Figure 23. Location of the road segment studied as part of this study

By using the Google Earth map, it was possible to draw the longitudinal profile corresponding to the extent of the studied low-traffic unpaved road (EBVT), as shown in Figure 24.

Figure 24. Longitudinal profile of the road

3.2. Analyses conducted by Cabette (2018)

In his study, Cabette (2018) divided the road section into 31 segments, each spaced 100 meters apart, to identify the homogeneous sections and representative points. He conducted tests using a Light Dynamic Penetrometer (LDP) and Plate Load tests (PL), in accordance with the EM ISO 22476-2 and NF P94-117-1 standards. The data obtained were then analyzed using the Cumulative Differences Method, following the guidelines of the American Association of State Highway and Transportation Officials (AASHTO). As a result of his analysis, five homogeneous sections were defined, and samples were collected at the following representative points: point 1 (0 m), point 3 (300 m), point 14 (1,342 m), point 21 (2,056 m), and point 31

(3,056 m). The examination of samples taken from these points revealed the presence of two distinct soil layers, as shown in Figure 25.

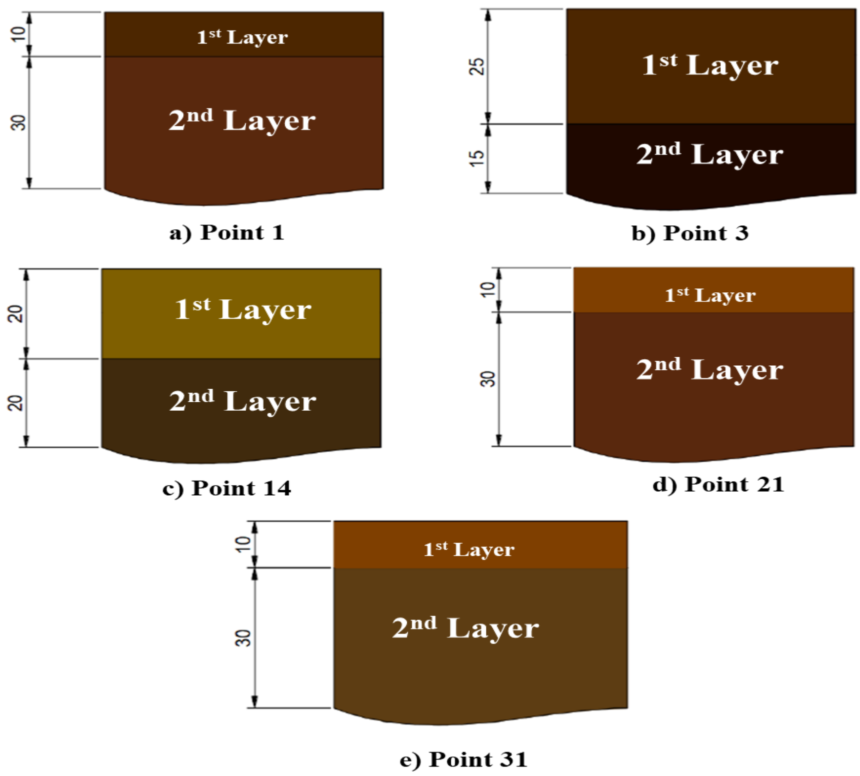


Figure 25. Geotechnical profile of the points where samples were collected (dimensions in cm).

After defining the homogeneous sections and their representative points, several tests were carried out:

a. Grain size analysis – performed according to the LNEC E 239-1970 standard;
b. Normal Proctor test – following ASTM D-698-07 standard to determine the optimum moisture content required for compaction;
c. CBR (California Bearing Ratio) – according to ASTM D-1883-07 standard, to assess the material resistance;
d. Light Dynamic Penetrometer (LDP) – EM ISO 22476-2 standard, used to measure soil penetration resistance. This test also helped identify the five homogeneous sections;
e. Compaction control using gammasonde – according to ASTM D-2922-01 standard, to determine dry and wet bulk densities, as well as moisture content;
f. Plate load test (CP) – performed according to NF P94-117-1 standard, to evaluate the bearing capacity of the soil. This test was also used for the identification of the five homogeneous sections;

g. In situ CBR – performed following BS 1377:9 standard, to assess material resistance directly on the field.

For the design of the unpaved road (EBVT), only the results of the CBR values and soil classification obtained from identification tests will be considered, as shown in Tables 4 and 5.

Table 4. Soil Classification at Representative Points (adapted from Cabette, 2018).

Representative points	Layers	Soil Type
Point 1 (0 m)	Layer 1	Well-graded sand with silt and gravel
	Layer 2	Well-graded gravel with silt and sand
Point 3 (300 m)	Layer 1	Well-graded sand with silt
	Layer 2	Poorly graded sand with silt
Point 14 (1342 m)	Layer 1	Silty sand with gravel
	Layer 2	Poorly graded sand with gravel
Point 21 (2056 m)	Layer 1	Silty sand
	Layer 2	Well-graded gravel with silt and sand
Point 31 (3056 m)	Layer 1	Silty sand
	Layer 2	Poorly graded gravel with silt and sand

Table 5. CBR Values at Representative Points (Adapted from Cabette, 2018).

Representative Points	Layers	Laboratory CBR (%)	CBR "in situ" (%)
Point 1 (0 m)	Layer 1	39	37
	Layer 2	30	
Point 3 (300 m)	Layer 1	37	40
	Layer 2	23	
Point 14 (1342 m)	Layer 1	18	24
	Layer 2	38	
Point 21 (2056 m)	Layer 1	29	31
	Layer 2	21	
Point 31 (3056 m)	Layer 1	30	28
	Layer 2	40	

The points 1 and 3 show quite similar CBR values, both when compared to each other and to the "in situ" measured values, and also indicate good soil resistance. On the other hand, point 14 shows the lowest CBR value, thus exhibiting the weakest resistance among the observed points. As for points 21 and 31, their CBR values for the first layer are close to those obtained in field tests, suggesting a satisfactory soil resistance as well. The other results from the tests conducted by Cabette (2018) are detailed in Tables 6, 7, and 8.

Table 6. Results of the Standard Proctor Test (Cabette, 2018).

Points	Layer	Apparent Specific Weight Optimal Dry Density (kN/m³)	Optimal Water Content (%)
1	1	20,1	9,2
	2	18,2	13,0
3	1	19,3	9,7
	2	17,3	14,0
14	1	18,6	13,5
	2	18,2	12,5
21	1	19,0	10,5
	2	17,0	14,0
31	1	18,2	13,0
	2	18,9	10,2

The values of the dry apparent specific gravity obtained in the laboratory were mostly very close to each other for the different representative points of the study. The same observation was made for the optimal moisture values. This suggests that the soil samples collected exhibit similar behavior.

Table 7. Results of the Plate Load Test (Cabette, 2018).

Points	EV1 (MPa)	EV2 (MPa)	EV2/EV1
1	108,6	157,2	1,448
3	124,9	142,6	1,142
14	37,9	63,0	1,662
21	124,5	101,8,	0,818
31	67,1	99,6	1,484

According to Table 8, it is noticeable that the results obtained for the dry apparent specific gravity in the laboratory via the Standard Proctor Test are comparable to those measured in the field using the radioactive load cell (gama-densimeter).

Table 8. Result of the Gamma-Densimeter Test. (Cabette, 2018).

Points	Apparent Dry Specific Weight (Lab.) (kN/m³)	Dry Specific Weight "in situ" (kN/m³)	Degree of Compaction (%)
1	20,1	20,3	100,98
3	19,3	20,5	105,89
14	18,6	18,4	98,63
21	19	18,1	94,95
31	18,2	17,7	97,04

The soil compaction rate corresponds to the ratio between the dry apparent specific gravity measured in the field and that obtained in the laboratory. Based on the results, it can be concluded that the soil in the studied area is well compacted.

3.2.1. Austroads Method (2009)

To apply the design according to the method described in the Austroads (2009) manual, it is necessary to consider local traffic without buses, for a design period of 20 years, which corresponds to an average daily traffic of 400 vehicles. This traffic class was selected following the recommendations of Jorge (2014), which explains why lower traffic classes, present in the manual, are not used for this design. Regarding the subgrade strength, the lowest CBR value measured in the laboratory, 18% at point 14, will be considered.

Thus, the input parameters for the application of this method are as follows:

- Traffic: 4×10^4
- CBR: 18%.

By applying these values to the abacus, the design according to the Austroads (2009) method indicates that a 10 cm base layer is required, leading to the road structure configuration shown in Figure 26.

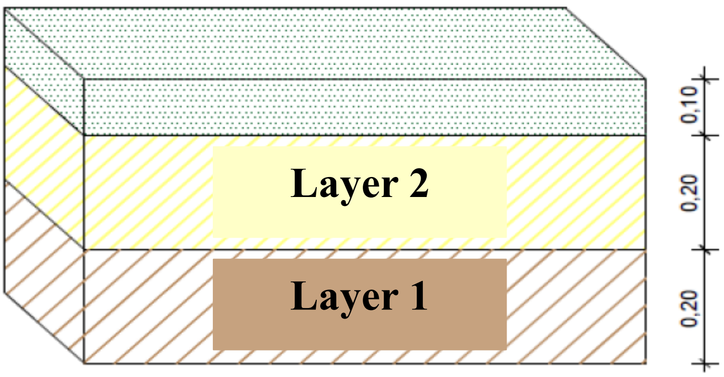


Figure 26. Design (cm) Austroads Method (2009). (Cabette, 2018).

The manual recommends adding a "protection" layer to the pavement, in addition to the designed base layer, because the surface layer is likely to deteriorate over time due to wear caused by traffic and weather conditions.

3.2.2. FCE Method (Brito, 2011)

For the dimensioning according to the FCE manual (Brito, 2011), only the soil classification and the obtained CBR value are taken into account. As with the previous method, the point with the lowest CBR value, i.e., point 14 with a CBR of 18%, is selected. This point, defined by granulometric analysis, is classified as silty sand with gravel. The parameters used for this calculation are:

- Soil type: silty sand with gravel;
- CBR : 18%.

Using Table 8, the FCE Manual (Brito, 2011) recommends a base layer thickness of 15 cm, as shown in Figure 27.

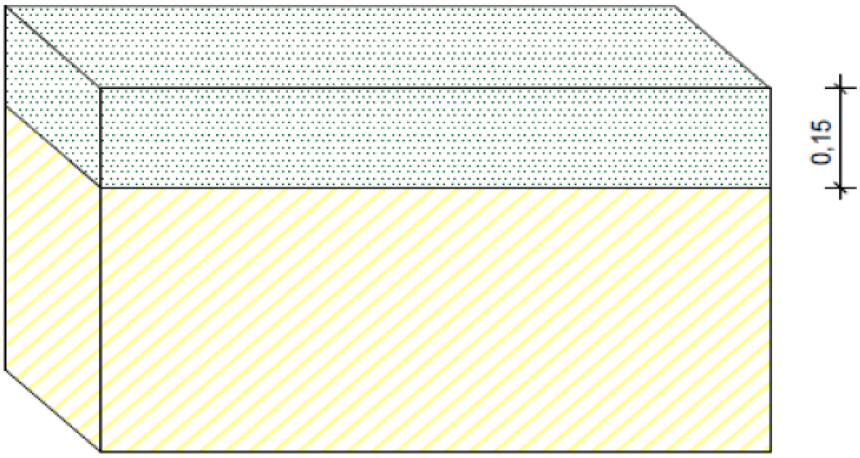


Figure 27. Dimensioning (cm) FCE Method (Brito, 2011). (Cabette, 2018).

3.3. Freitas (2019) Studies

In Freitas' (2019) study, the procedure to determine homogeneous sections and their representative points followed a similar approach to Cabette (2018). The road was divided into 31 sub-segments spaced 100 meters apart, but points 1 and 31 were excluded from the analysis of the homogeneous sections. Using the results from the Light Falling Weight Deflectometer (LFWD) and the AASHTO cumulative difference method, the homogeneous sections of the studied segment were identified.

Freitas (2019) defined six homogeneous sections, from which soil samples were collected to perform tests to determine the geotechnical characteristics of the road materials.

The representative points for each homogeneous section are: 3 (200 m), 9 (796 m), 12 (1103 m), 17 (1623 m), 24 (2319 m), and 28 (2719 m) (distance from the starting point).

Once these points were defined, several geotechnical tests were performed, including:

- Grain size analysis – LNEC E 239-1970 standard;
- Standard Proctor test – ASTM D-698-07, to determine the optimum compaction moisture;
- California Bearing Ratio (CBR) – ASTM D-1883-07 standard, to evaluate the resistance of the materials;
- Light Falling Weight Deflectometer (LFWD) – ASTM E-2835-11 standard, used to measure surface deformation and dynamic modulus of elasticity. This test was conducted at all test points.

The results from these tests were used to determine the soil classification and CBR values, as shown in Tables 9 and 10.

Table 9. Soil Classification at representative points (adapted from Freitas, 2019).

Representative Points	Soil type
Point 3 (200 m)	Well graded sand with silt
Point 9 (796 m)	Well graded sand with silt
Point 12 (1103 m)	Well graded sand with silt
Point 17 (1623 m)	Well graded sand with silt
Point 24 (2319 m)	Silty sand
Point 28 (2719 m)	Well graded sand with silt

Table 10. CBR Values at Representative Points (Adapted from Freitas, 2019).

Representative Points	Laboratory CBR (%)
Point 3 (200 m)	37
Point 9 (796 m)	36
Point 12 (1103 m)	34
Point 17 (1623 m)	34
Point 24 (2319 m)	20
Point 28 (2719 m)	25

Points 3 and 9 show similar CBR values, as do points 12 and 17. However, points 24 and 28 show lower values compared to the others. To ensure the safety of the project and for the EBVT dimensioning calculation, the CBR value from point 24 will be used.

The results of other tests conducted by Freitas (2019) are summarized in Tables 11, 12, 13, and 14.

Table 11. Results of the Standard Proctor Test (Freitas, 2019).

Points	Dry Apparent Specific Weight (kN/m3)	Optimum water content (%)
3	19,4	9,7
9	18,7	11,5
12	18,7	11,5
17	18,9	12
24	17	16,5
28	18,4	12,5

The results from the Standard Proctor Test allowed the determination of both the dry bulk density and the optimum moisture content for the representative points. Generally, the values for these two parameters are quite similar, suggesting that the road's soil behaves in a relatively uniform manner.

Table 12. Results of the LFWD Test. (Freitas, 2019).

Points	Evd (MPa)
3	20,27
9	24,85
12	21,9
17	20,11
24	12,44
28	21,56

The LFWD test provides the dynamic deformation modulus of the soil. The measured values are mostly similar, except for point 24, where the modulus is significantly lower compared to the other points. The manual for the device used in the Light Falling Weight Deflectometer test provides a table showing the correlation between the deformation modulus and the soil's compaction degree. Table 13 shows this correlation.

Table 13. Relationship between the Evd modulus and soil compaction (Zorn, 2016).

Soil Types (DIN 18 196)	Compaction Rate (%)	Static Elasticity Modulus (EV2) (MPa)	Dynamic Elasticity Modulus (Evd) (MPa)
GW, GI	≥ 103 ≥ 100 ≥ 98 ≥ 97	≥ 120 ≥ 100 ≥ 80 ≥ 70	≥ 60 ≥ 50 ≥ 40 ≥35
GE, SE, SW, SI	≥ 100 ≥ 98 ≥ 97	≥ 80 ≥ 70 ≥ 60	≥ 40 ≥35 ≥32
Mixed and Granular Soils Fine	≥ 100 ≥ 97 ≥ 95	≥ 45 ≥ 30 ≥ 20	≥ 25 ≥ 15 ≥ 10

Table 14. Relationship Between Evd and Degree of Compaction. (Freitas, 2019).

Points	Evd (MPa)	Degree of Compaction (%)
3	20,27	≥ 97
9	24,85	≥ 97
12	21,9	≥ 97
17	20,11	≥ 97
24	12,44	<95
28	21,56	≥ 97

Based on this relationship, the compaction degree of the representative points of each homogeneous section was estimated. Generally, it was observed that the soil is well compacted.

3.3.1. Austroads Method (2009)

According to the method presented in the Austroads manual (2009), the lowest CBR value, which is from point 24 at 20%, will be used. The traffic category chosen corresponds to a local access road without buses, for a design period of 20 years, with an estimated average daily traffic of 400 vehicles.

The input parameters for applying this method are therefore:

- Traffic: $4x10^4$ vehicles;
- CBR: 20%.

When these values are applied to the graph in Figure 5, the Austroads (2009) method recommends a base layer thickness of 10 cm. Figure 28 illustrates the structure of the designed layer.

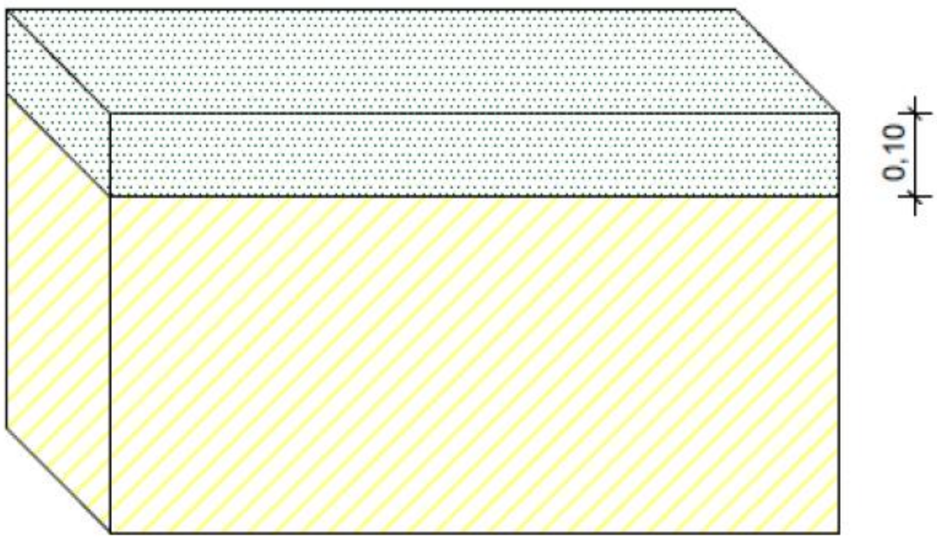


Figure 28. Design (cm) Austroads Method (2009). (Freitas, 2019).

The manual recommends adding an additional "sacrificial" layer to the pavement, in addition to the calculated base layer, as the surface layer undergoes material loss over time due to traffic and climatic conditions.

3.3.2. FCE Method (Brito, 2011)

According to the method described in the FCE manual (Brito, 2011), the geotechnical characteristics of point 24, consisting of silty sand with a laboratory CBR of 20%, are considered for the design. The parameters used to determine the thickness of the base layer are as follows:

- Soil type: silty sand;
- CBR: 20%.

By applying these values to Table 8, the FCE method (Brito, 2011) suggests a thickness of 15 cm for the base layer, as shown in Figure 29.

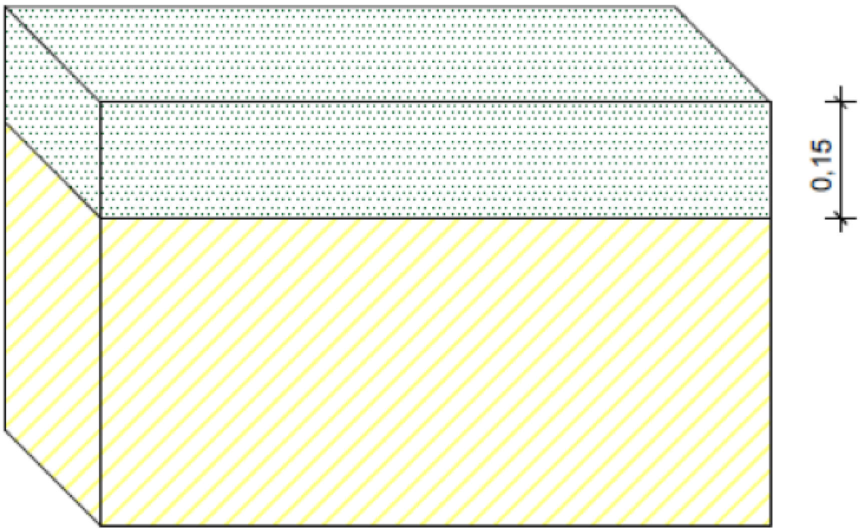


Figure 29. FCE Design (cm) (Brito, 2011). (Freitas, 2019).

Chapter IV: Comparative study

4. Comparative analysis

Many guides and manuals are available to assist designers in the construction of low-traffic unpaved roads. These documents provide detailed recommendations covering various aspects such as site selection, materials, drainage systems, and the final design of the road.

This study focuses specifically on the structural design of these unpaved roads using two design methods presented in this study, one Australian and the other British, in order to identify the most appropriate methodology to apply to the studied site. This chapter aims to compare these two methods, analyze the results of previous research, and conduct a sensitivity study on the sizing procedures.

4.1. Comparative Analysis of Tests

Despite the distinct tests conducted by Cabette (2018) and Freitas (2019) to characterize soils, certain parameters obtained are shared between both studies. The purpose of the comparative analysis is to evaluate the impact of traffic, weather conditions, and a specific intervention carried out by the Bragança municipality on the results obtained. However, the representative points selected differ in each study. Therefore, a more precise comparative analysis must be performed, taking into account the subsection of the homogeneous section represented by each point.

4.1.1. Homogeneous Sections

Cabette (2018) and Freitas (2019) applied the cumulative differences method, as recommended by AASHTO, to define homogeneous sections. Cabette (2018) based this delineation on the results of PDL and CP tests, while Freitas (2019) utilized data from LFWD tests. Figures 30, 31, and 32 illustrate the homogeneous sections identified along the road's topographic profile.

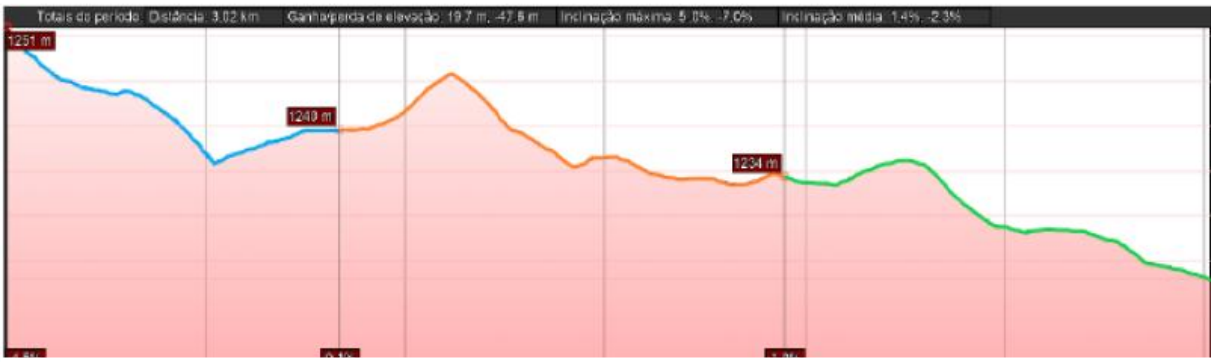


Figure 30. Homogeneous sections using the Cabette PDL test (Freitas, 2019).

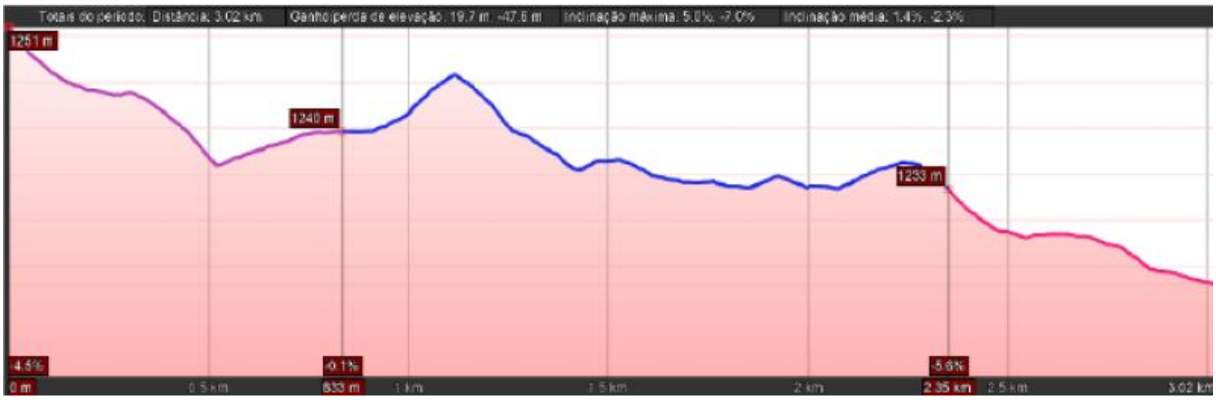


Figure 32. Homogeneous sections using the CP - Cabette test (Freitas, 2019).

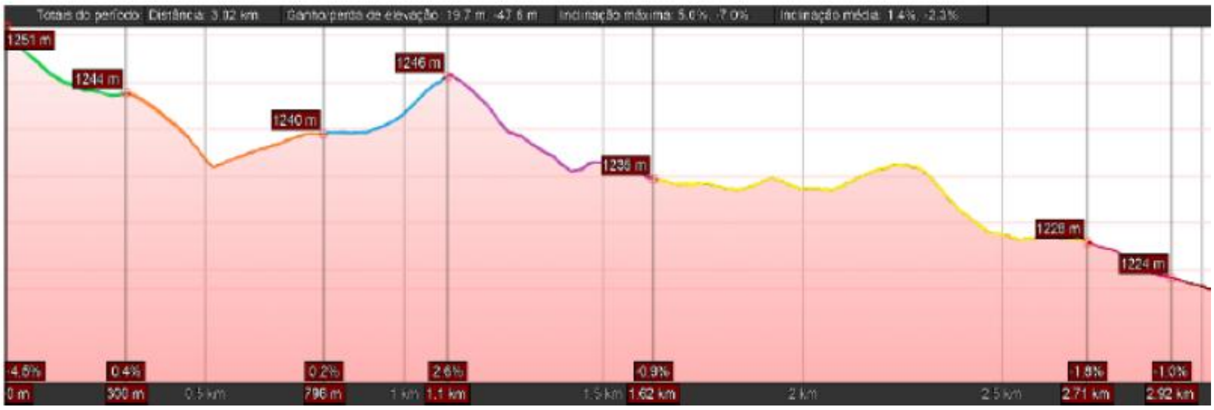


Figure 31. Homogeneous sections using the LFWD test - Freitas (Freitas, 2019).

Although the method used was identical and the study focused on the same road, the defined homogeneous sections differ. This can be attributed to several factors, such as the time gap between the two studies, the impact of traffic and weather conditions, the type of test used to identify the homogeneous sections, or the intervention performed on the road. It is worth noting that Freitas (2019) identified a greater number of subsections, although the sections remain broadly similar across the three figures presented. This suggests that the road retains characteristics comparable to those observed in the earlier study. Additionally, the use of the LFWD test allowed for a more precise segmentation of the studied segment.

4.1.2. CBR Value

In both studies mentioned, the laboratory CBR test was conducted. However, only Cabette (2018) also performed an in situ CBR test. To enable a comparison of the results obtained, the CBR values determined in the laboratory will be considered. The comparison is presented in Table 15.

Table 15. Comparison of CBR values.

Representative Points	Cabette (2018)		Freitas (2019)	
	Layers	Laboratory CBR (%)	Representative Points	Laboratory CBR (%)
Point 1	1	39	**Point 3**	37
	2	30		
Point 3	1	37	**Point 9**	36
	2	23		
Point 14	1	18	**Point 12**	34
	38			
Point 21	1	29	**Point 17**	34
	2	21		
Point 31	1	30	**Point 24**	20
	2	40		
			Point 28	25

The comparison of CBR values reveals few significant differences. Point 3, common to both studies, shows an identical CBR value. The road continues to exhibit relatively high CBR values, indicating good platform strength.

4.1.3. Optimum Dry Unit Weight and Optimum Moisture Content

The standard Proctor test was conducted to determine the optimum dry unit weight and the optimum moisture content of the soil, thus identifying the amount of water required for optimal compaction. Table 16 compares the optimum dry unit weight values, while Table 16 presents the comparison of optimum moisture content values.

Table 16. Comparison of dry apparent specific weight.

Points	Cabette (2018)		Freitas (2019)	
	Layers	Optimum Dry Apparent Specific Weight (kN/m3)	Points	Optimum Dry Apparent Specific Weight (kN/m3)
1	1	20,1	**3**	19,4
	2	18,2		
3	1	19,3	**9**	18,7
	2	17,3		
14	1	18,6	**12**	18,7
	38	18,2		
21	1	19	**17**	18,9
	2	17		
31	1	18,2	**24**	17
	2	18,9		
			28	18,4

The results revealed similar values for apparent dry unit weight and optimum moisture content between the two studies. At point 3, the dry unit weight and optimum moisture content values were perfectly identical.

Table 17. Comparison of optimum moisture content.

Points	**Cabette (2018)**		**Freitas (2019)**	
	Layers	Optimum water content (%)	**Points**	Optimum water content (%)
1	1	9,2	**3**	9,7
	2	13		
3	1	9,7	**9**	11,5
	2	14		
14	1	13,5	**12**	11,5
	38	12,5		
21	1	10,5	**17**	12
	2	14		
31	1	13	**24**	16,5
	2	10,2		
			28	12,5

4.1.4. Modulus of Deformability

To evaluate the modulus of deformability, plate load tests and LFWD tests were conducted. The results obtained by Cabette (2018) and Freitas (2019) are summarized in Table 18.

Table 18. Comparison of the modulus of deformability.

Points	**Cabette (2018)**		**Freitas (2019)**	
	EV_1 (MPA)	EV_2 (MPA)	**Points**	EV (MPA)
1	108,6	157,2	**3**	20,27
3	124,9	142,6	**9**	24,85
14	37,9	63	**12**	21,9
21	124,5	101,8	**17**	20,11
31	67,1	99,6	**24**	12,44
			28	21,56

The plate load test provided EV_1 and EV_2 values, corresponding to the soil's static modulus of deformability, while the LFWD test yielded the Evd value, representing the dynamic modulus of deformability.

According to Moraes (2018), in static tests, the modulus of deformability is calculated based on the stress-strain relationship of the material under progressively applied loads, resulting in a series of successive equilibrium states. In contrast, dynamic tests determine the deformability constant from the material's continuous and dynamic response to rapidly applied cyclic loads. Since static and dynamic deformability moduli differ, their results cannot be directly compared. Freitas (2019) was unable to establish a correlation between EV_2 and Evd values due to the high dispersion of data obtained from the tests. This discrepancy appears to be primarily attributable to the time gap between the tests and the intervention performed on the studied section.

The Zorn manual (2016) proposes a correlation between the deformability moduli EV2 and Evd using the following equation:

$$EV_2 = 2 * Evd \qquad \textbf{[7]}$$

By applying this formula to the Evd values obtained, the calculated results are presented in Table 19.

Table 19. Evd and Ev_2 ratio

Freitas (2019)		
Points	EVd (MPA)	EV_2 (MPA)
3	20,27	40,54
9	24,85	49,7
12	21,9	43,8
17	20,11	40,22
24	12,44	24,88
28	21,56	43,12

The EV_2 values estimated using the correlation proposed by Zorn (2016) were significantly lower than those obtained by Cabette (2018) during the plate load test. This discrepancy can be attributed to several factors: the use of the road for nearly a year, the intervention performed in the interim, and the differing nature of the tests, which were conducted at different times and under varying conditions. Consequently, directly comparing the EV_2 values is not appropriate.

To more accurately assess the validity of the proposed formula, it would be advisable to perform plate load tests and LFWD tests simultaneously on the same site and under identical conditions. This would ensure more reliable and comparable results.

4.1.5. Soil Compaction

Cabette (2018) assessed the soil compaction degree by calculating the ratio between the in situ apparent dry unit weight and the laboratory-determined dry unit weight. In contrast, Freitas (2019) determined the compaction degree using the ratio between the Evd value and the compaction degree specified in the LFWD equipment manual table. The results obtained by both authors are compared in Table 20.

Table 20. Soil compaction

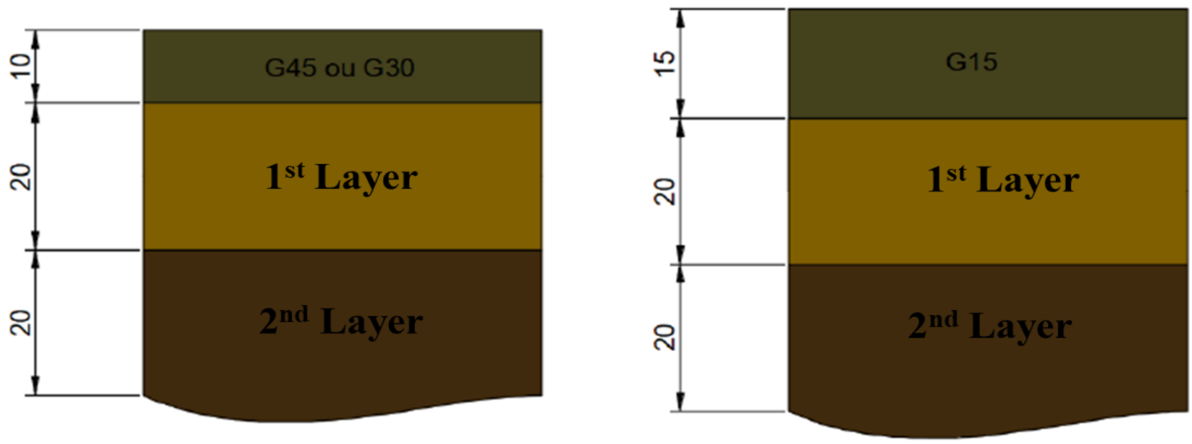
Cabette (2018)		Freitas (2019)	
Points	Degree of compaction (%)	**Points**	Degree of compaction (%)
1	100,98	**3**	≥ 97
3	105,89	**9**	≥ 97
14	98,63	**12**	≥ 97
21	94,95	**17**	≥ 97
31	97,04	**24**	≥ 95
		28	≥ 97

Based on the compaction degree estimates provided by Cabette (2018) and Freitas (2019), it can be concluded that the soil was well compacted in both studies.

4.2. Comparative Analysis of Methodologies

The pavement structure design will be analyzed by comparing the results obtained by Cabette (2018) and Freitas (2019) with the values presented in the previous chapter.

In the 2018 study, Cabette (2018) applied the method proposed by the Ethiopian ERA manual (2011) and determined the design, as illustrated in Figure 33.



(a) G45 ou G30 (b) G15

Figure 33. EBVT sizing (cm) - Cabette (2018).

The ERA (2011) manual considers the CBR value of the gravel used for the base layer during the design process. This method classifies G45 and G30 gravels as natural gravels, with CBR values of 45% and 30%, respectively, while G15 corresponds to a soil/gravel mixture with a CBR value of 15%.

For the estimated CBR value of the subgrade, Cabette (2018) designed a gravel base layer with a thickness of 10 cm for gravels with CBR values of 45% and 30%, and a thickness of 15 cm for gravels with a CBR of 15%. Regarding the subgrade, classified as silty sand with gravel and a laboratory CBR value of 18%, the thickness of the base layer ranges between 10 and 15 cm, as shown in Table 21.

Table 21. Comparison - Cabette (2018).

Comparison - Cabette (2018)			Base Layer Thickness (cm)		
Point 24	Soil	CBR (%)	ERA Manual	Austroads Manual	FCE Manual
	Silty sand with pebbles	18	10 - 15	10	15

Freitas (2019) a également opté pour la méthodologie éthiopienne proposée par le manuel de l'ERA (2011). Cependant, contrairement à l'étude de Cabette (2018), la route avait été utilisée pendant près d'un an lorsqu'il a collecté ses données. En raison de l'impact du trafic, des conditions météorologiques défavorables et d'une intervention du conseil municipal de Bragança, les valeurs obtenues par Freitas (2019) diffèrent de celles trouvées par Cabette (2018). En 2019, Freitas a proposé le dimensionnement présenté à la figure 34.

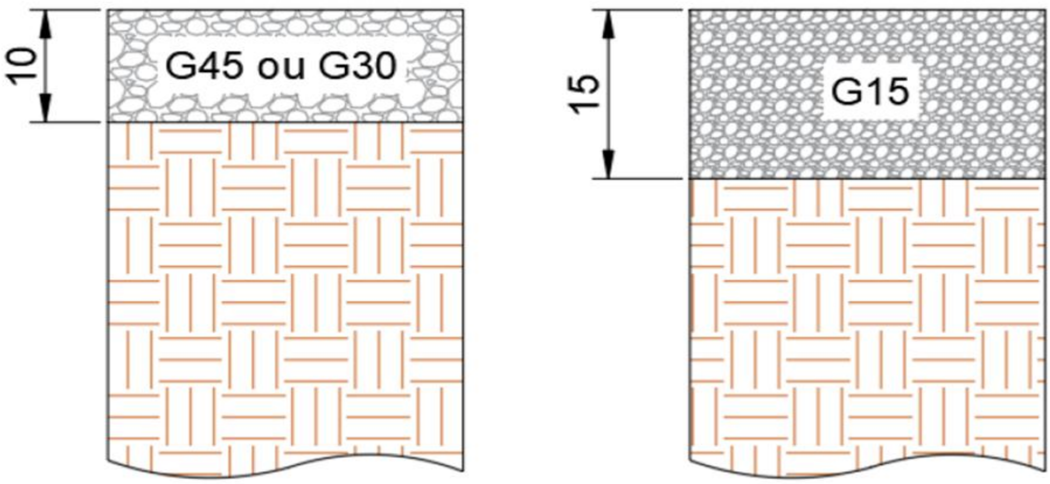


Figure 34. Dimensions (cm) of the EBVT - Freitas (2019)

The input variables for the design process according to the ERA (2011) manual included a CBR value of 20% and soil classified as silty sand. Although the input data differed, the determined thickness remained the same due to the minimal variation in the CBR value and the use of the same traffic class for the design.

Table 22 provides a comparison between the results from Freitas (2018) and those obtained in this study.

Table 22. Comparison - Freitas (2019)

Comparison - Freitas (2019)			Base Layer Thickness (cm)		
Point 24	Soil	CBR (%)	ERA Manual	Austroads Manual	FCE Manual
	Silty sand	20	10 - 15	10	15

The thicknesses of the designed base layers were found to be identical for the data from Cabette (2018) and Freitas (2019). This observation, along with the other comparisons conducted in this chapter, indicates that the condition of the road remained stable. In other words, after one year of use and an intervention carried out by the Bragança City Council, the road exhibited performance similar to that observed the previous year, prior to the new tests.

4.3. Sensitivity Analysis

A sensitivity analysis of the variables used in the different methodologies will be conducted by assigning arbitrary values to the input variables. This approach will allow for the evaluation of their influence on the design of an unpaved pavement of the EBVT type.

4.3.1.ERA Manual (2011)

In the framework of pavement design according to the ERA Manual (2011), the main variables considered are the traffic class and the classification of the subgrade soil. The graphs shown in Figures 35, 36, and 37 highlight the influence of input variables on the layer thickness. Moreover, the thickness is affected by the strength of the gravel used in the design. The ERA Manual (2011) defines three types of gravel for the designed layer, namely classes G45, G30, and G15, corresponding to CBR values of 45%, 30%, and 15%, respectively.

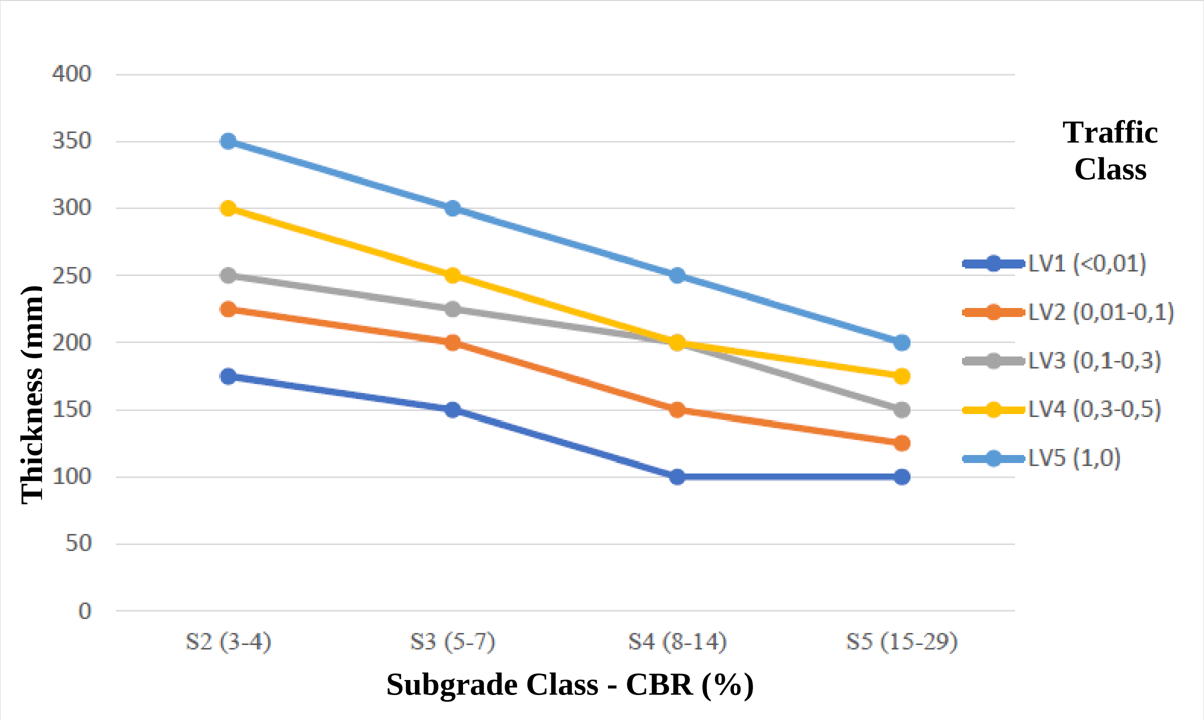


Figure 35. Influence of CBR and traffic on pavement thickness - ERA Manual (2011) CBR 45%.

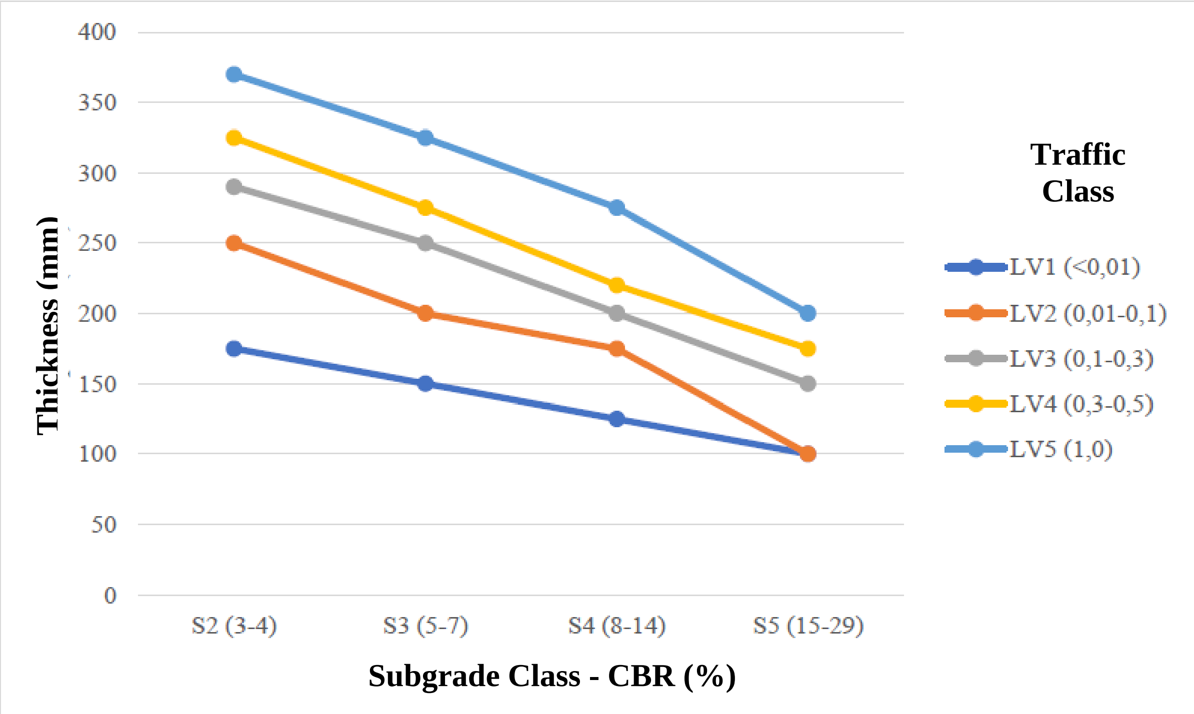


Figure 36. Influence of CBR and traffic on pavement thickness - ERA Manual (2011) CBR 30%.

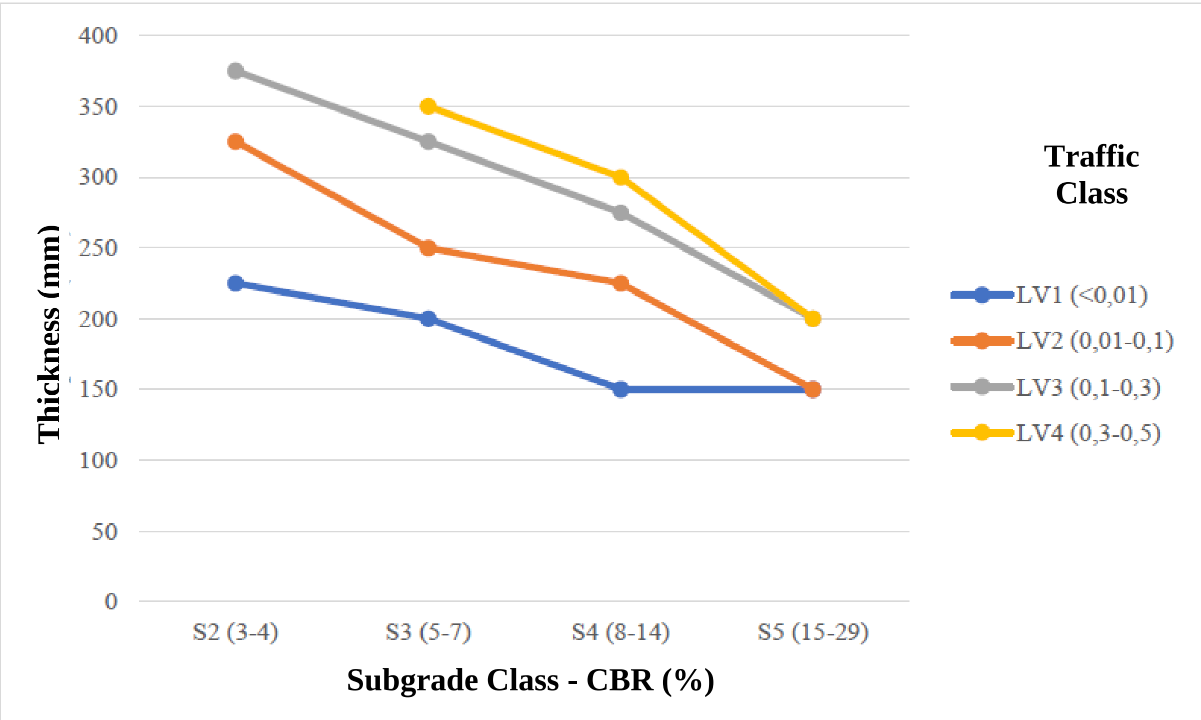


Figure 37. Influence of CBR and traffic on pavement thickness - ERA Manual (2011) - CBR 15%.

The first observation when analyzing the graphs above is that the thickness of the designed layer decreases as the strength of the gravel to be used increases. For instance, for a gravel layer with a CBR of 15%, it is not possible to design a road with a traffic class of LV5, as the low strength is not sufficient to reduce the stresses transmitted to the subgrade, which could lead to pavement failure.

Additionally, it can be observed that the higher the traffic class, the greater the thickness of the designed layer. Conversely, with higher CBR values of the subgrade, the thickness of the layer will be lower.

Finally, analyzing the variation in the base layer thickness for G45 gravel, the results are shown in Figure 38.

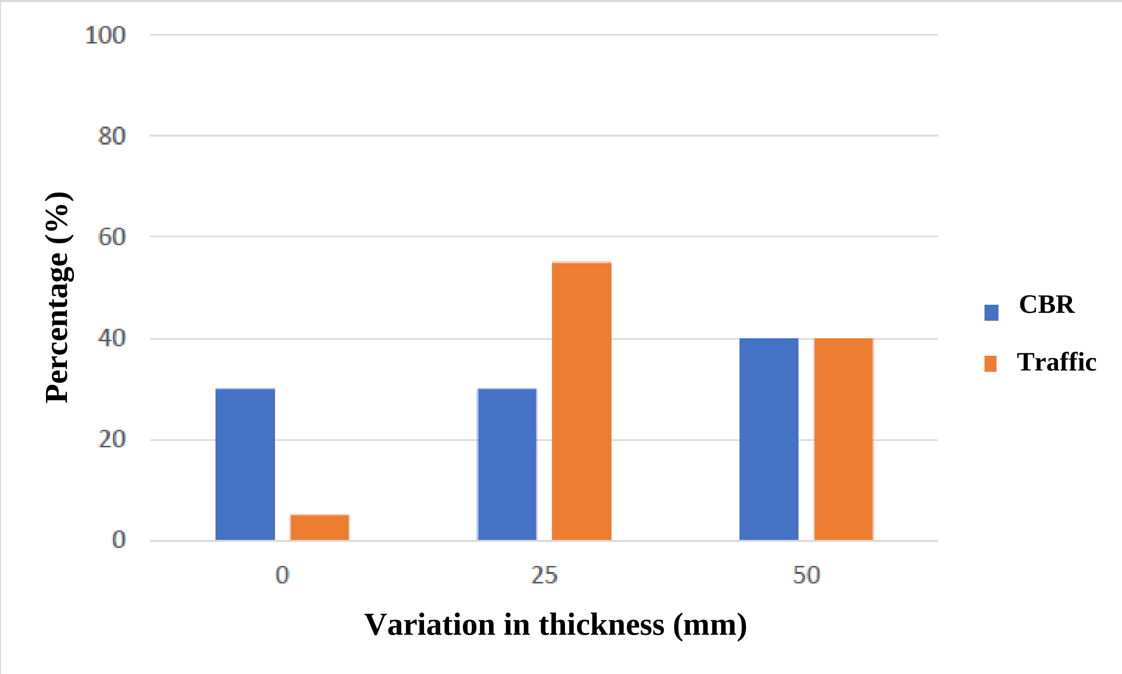


Figure 38. Variation in the thickness (%) of the base layer - ERA Manual (2011) - CBR 45%.

Regarding the variation in the CBR value, 70% of the results showed a change in the thickness of the designed layer. In contrast, when the traffic class varied, 95% of the results showed a change.

For the base layer with G30 gravel, the variation in layer thickness is shown in Figure 39.

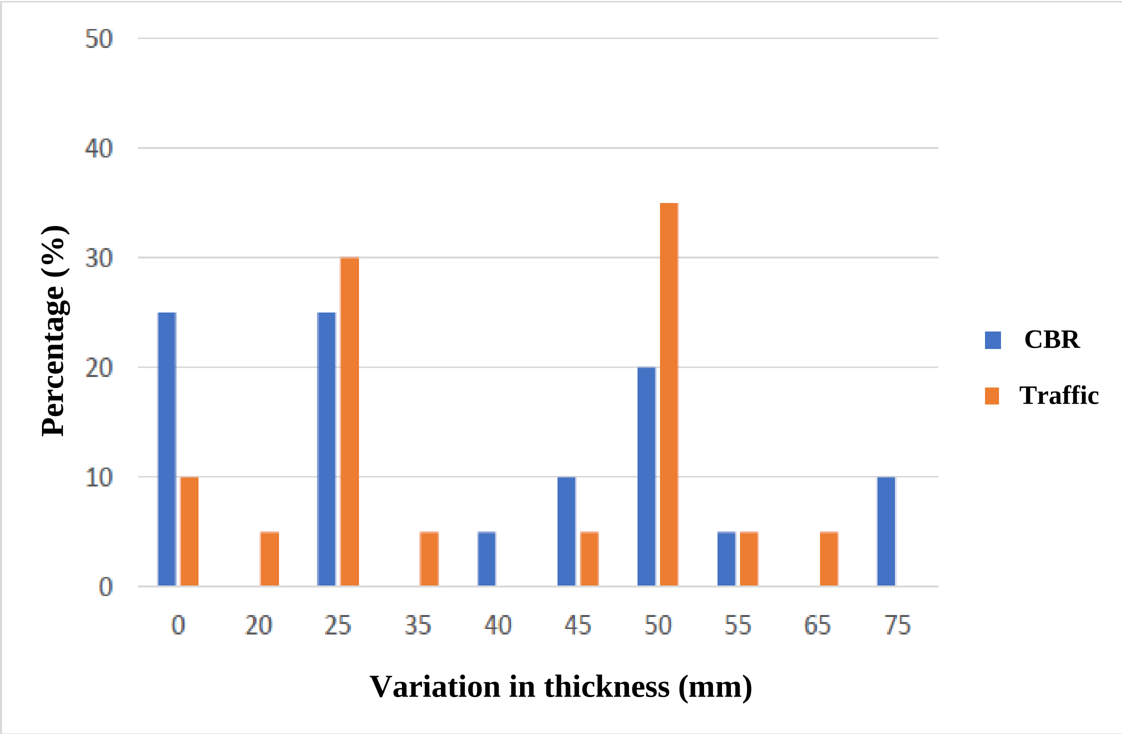


Figure 39. Variation in the thickness (%) of the base layer - ERA Manual (2011) - CBR 30%.

When the CBR value changes, 75% of the results show a variation in the thickness of the base layer. However, when the traffic class changes, 90% of the results lead to a variation in the layer thickness. Finally, for the base layer with G15 gravel, Figure 40 shows the variation in the base layer thickness.

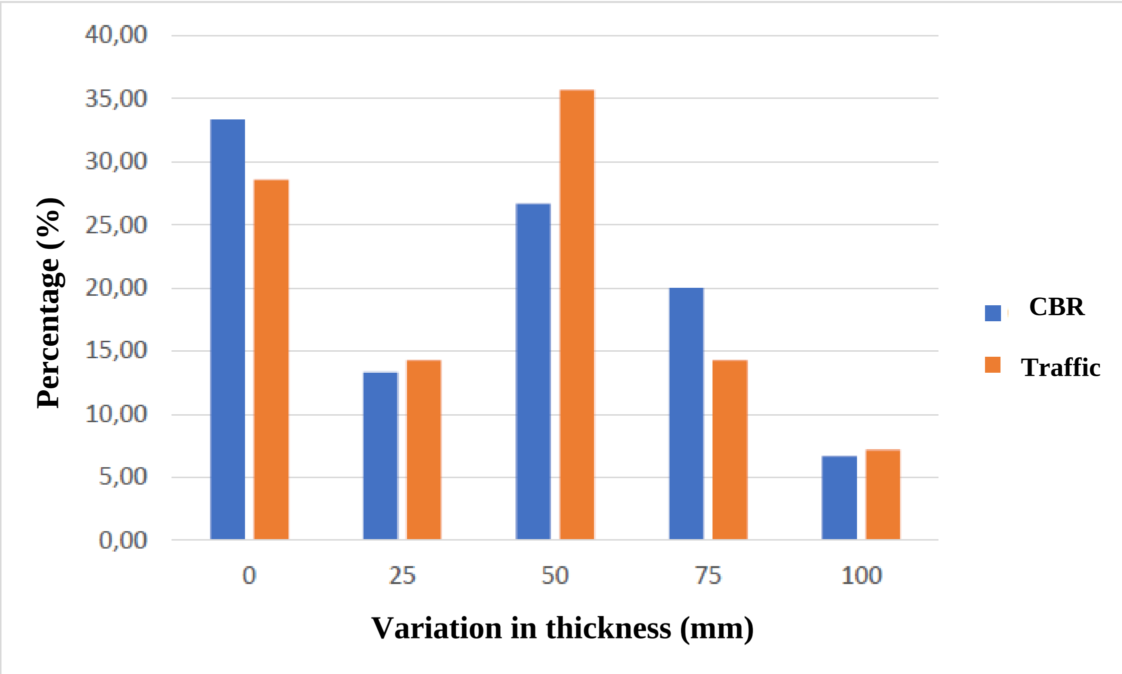


Figure 40. Variation in the thickness (%) of the base course - ERA Manual (2011) CBR 15%.

For G15 gravel, when the CBR value is changed, 66.6% of the results show a variation in the layer thickness, while for the variation in traffic class, 71.4% of the results show a change. It is clear that the estimated thickness of the designed layer varies more when the traffic class changes compared to the variation in the CBR value. In conclusion, it can be stated that, for this method, the most influential variable is the traffic class.

4.3.2. Austroads (2009)

Similar to the methodology outlined in the ERA Manual (2011), the main variables influencing the thickness of the layer to be designed according to the Austroads Manual (2009) are the foundation strength, CBR value, and traffic class. Figure 41 illustrates the behavior of the curve when the CBR value and traffic class are varied.

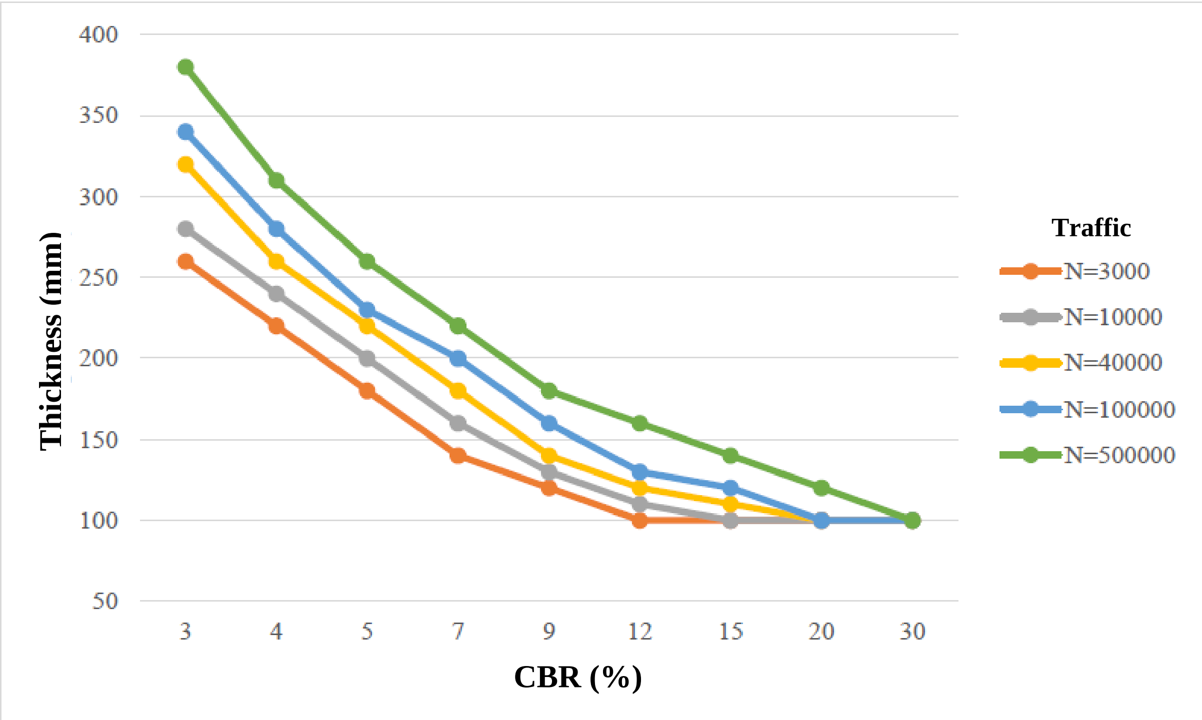


Figure 41. Influence of CBR and traffic on pavement thickness - Austroads (2009).

It is observed that the curves converge to a thickness of 100 mm, which corresponds to the minimum thickness required for the EBVT base layer according to the Austroads Manual (2009).

By analyzing the impact of the variation in CBR values and traffic on the designed base layer thickness, the following graph is obtained, see Figure 42.

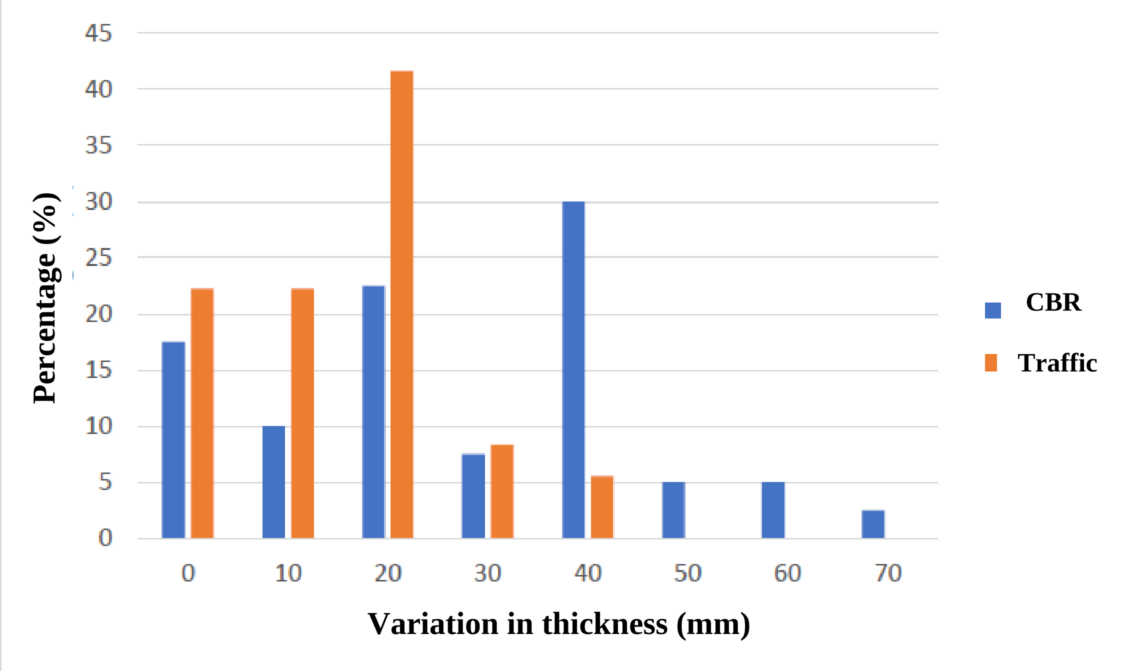


Figure 42. Variation in the thickness (%) of the base layer - Austroads (2009).

When observing the variation in the CBR value, 82.5% of the results show a change in the base layer thickness. In contrast, for variations in traffic volume, 77.8% of the results also indicate a change in thickness.

It follows that the base layer thickness varies less significantly when traffic is altered compared to the variation in CBR. This suggests that the subgrade strength is the most influential variable for this method, as stated in the Book of Proceedings (2015).

4.3.3. FCE (Brito, 2011)

The British design method, shown in Figure 43, only considers the subgrade strength and soil type as factors influencing the thickness of the designed layer. As with other approaches, the higher the CBR value of the foundation soil, the thinner the required base layer will be. Consequently, the strength of the material in the subgrade and the CBR value are the most significant variables in this method.

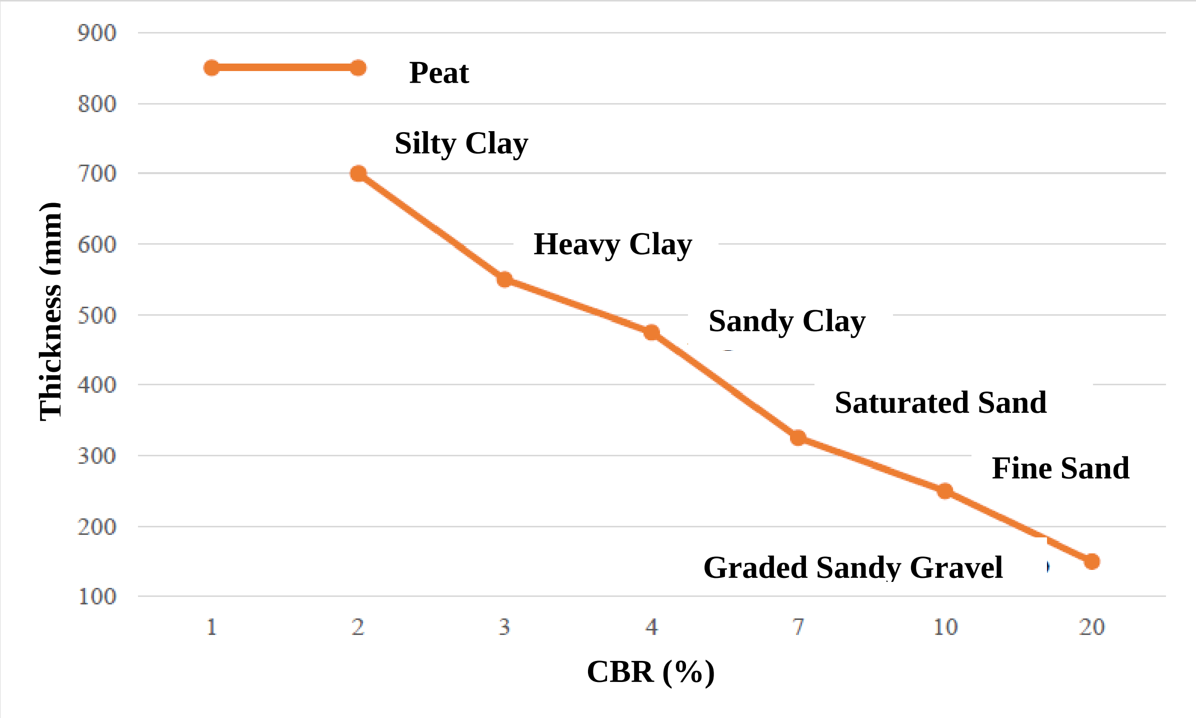


Figure 43. Influence of CBR and material type on pavement thickness - FCE (Brito, 2011).

Conclusions and future work

The aim of this work was to highlight the importance of low-traffic unpaved roads for the economic development and the movement of people and goods within a country's road network. Acknowledging their essential role, it becomes clear that it is crucial to design these roads properly and efficiently, so that they fulfill their structural function while ensuring safety and comfort for users throughout their lifespan.

There are several studies and guidelines that assist engineers in the design and construction of these roads. Many factors can influence the quality of these infrastructures, which underscores the need for more in-depth tests and studies on construction techniques, maintenance, and the most sensitive factors in the design of low-traffic roads.

Although laboratory and field tests were not conducted in this study, a comparison between three existing methodologies was made to analyze the differences between them. The results obtained were as expected: the three methods yielded similar results regarding the thickness of the base layer. The common variable in these methods is the CBR, i.e., the subgrade strength, which highlights the importance of the foundation and its strength in the road's behavior. Indeed, the higher the subgrade strength, the thinner the required base layer. A disadvantage of the methods proposed by the Austroads Manual (2009) and the FCE Manual (Brito, 2011) is the lack of specification regarding the type of soil to be used for the base layer construction. Both methods only suggest the granulometric curve for the base and wearing layers, without providing a specific CBR value for the base layer, unlike the ERA Manual (2011). However, these methods have the advantage of being relatively simple to implement and easy to understand. The comparison of data from the studies by Cabette (2018) and Freitas (2019) showed similar results, indicating that the road exhibits similar behavior, even after almost a year of use and following an intervention by the Bragança Municipality. This suggests that the road's condition has remained stable during this period, without significant deterioration or improvement. One of the key factors in the development of unpaved roads is the availability of local materials, as these roads are often built with the soils found on-site.

To better assess the condition of the road, a functional and structural analysis is needed. This would help identify defects and their probable causes, allowing for the most appropriate intervention to be defined. There is still much to analyze in the case study. The more data collected about the road, the more can be understood about its behavior, and the more parameters can be used to evaluate the road, in addition to providing a historical record for future projects.

Recommendations for future work include

- Adopting a different methodology for determining homogeneous sections;
- Collecting data on defects to refine the determination of homogeneous sections;
- Conducting new tests to better characterize the road's physical and mechanical properties, such as dynamic CBR.
- Collecting data on rainfall periods to better evaluate the drainage system.
- Developing a project for the design of the drainage system.
- Comparing results with the relationship proposed by Zorn (2016) between the EV_2, CBR, and Evd values.

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