

Study of subgrade behavior and estimation of resilient modulus values from a pavement deflection database

Estudio del comportamiento de subrasantes y estimación del módulo resiliente a partir de una base de datos de deflexiones de pavimentos

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ABSTRACT: The Pavement Quality Control Laboratory (LCCP) of the Universidad de la República (UdelaR), Uruguay, has been using the Falling Weight Deflectometer (FWD) on pavements throughout the country since 2013. With this data, the LCCP created a georeferenced database that contains both measurement data and geological information on the location of each point according to the official geological map. Previous studies evaluated subgrade behaviors by analyzing the variation of the surface modulus (SM) with the distance from the load application point. This study proposes a criterion to determine when subgrade response can be considered as linear elastic, based on the percentage variation of the SM as a function of distance. The geological origin of the soils is also used to evaluate subgrade behavior. For typical geological units, SM values are determined, and ranges of resilient modulus (M_r) are estimated. These ranges are then compared with values deduced from the assumed soil classification groups of these units. As a result, differences are found when comparing calculated M_r values with theoretical ranges, mainly for granular soils, where the calculated values are lower. The importance of calibrating the C factor based on laboratory tests is emphasized to avoid a poor estimation of the design M_r value.

KEYWORDS: road geotechnics – FWD – database – resilient modulus - subgrade

1 INTRODUCTION

The condition of a pavement can be classified into two main categories: functional and structural (Park et al. 2007). Functional condition generally defines when a pavement has reached its service life or when intervention is necessary. Structural condition, which is not perceived by the user, is fundamental for the pavement to withstand the loads that transit over it, for a given service life and with appropriate comfort and safety conditions. Pavements with poorer structural conditions will experience faster rates of deterioration in functional conditions (Bryce et al. 2016).

The structural condition is assessed through the collection of pavement deflection data. The use of the Falling Weight Deflectometer (FWD) has become widespread worldwide since 1980 as a standard technique for determining pavement deflections. This equipment applies a load pulse onto a circular plate (usually 30 cm in diameter), which aims to simulate the passage of a loaded axle at a certain speed. The deflections generated by this load are determined using geophones, which measure the velocity of the pavement surface, placed at various distances from the load plate. From these measurements, the pavement deflection bowl can be constructed (Smith et al. 2017).

Since 2013, the Faculty of Engineering at the University of the Republic (UdelaR) in Uruguay has acquired technological equipment for pavement surveying through a Strengthening Technological Services project funded by the National Agency for Research and Innovation (ANII). One of the acquired pieces of equipment is a KUAB brand Falling Weight Deflectometer (FWD). Through agreements with the Ministry of Transportation and Public Works (MTOP) and services provided to the community, over 20,000 measurements have been taken on roads and highways throughout the country. Figure 1 shows the spatial distribution of the FWD measurements considered for this study.

In 2013, the Pavement Quality Control Laboratory (LCCP) working group was established, responsible for the management and maintenance of these equipments up to the present date.

Based on the collected data, a georeferenced database is created with the aim of facilitating its analysis and establishing representative parameters based on the type of pavement and the subgrade on which they are supported (Neme Gamarra et al. 2022).

This study focuses on the examination of deflection bowl parameters to evaluate the subgrade behavior.

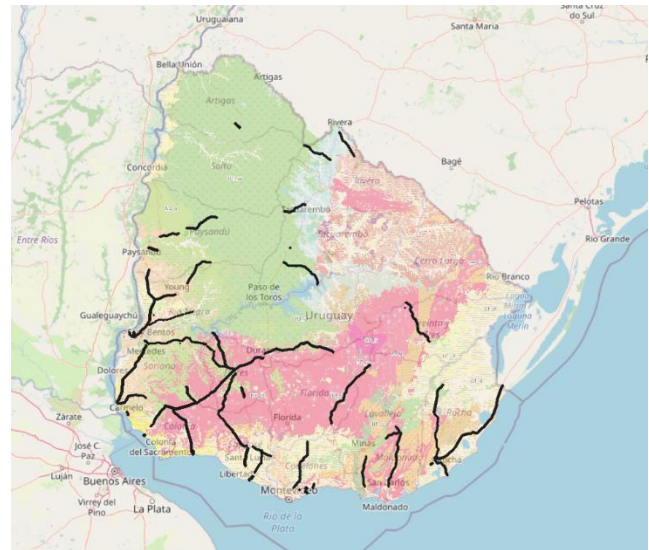


Figure 1. Geospatial distribution of FWD measurements over the Geological map of Uruguay

2 PAVEMENT SUBGRADE CHARACTERIZATION

2.1 Subgrade stiffness

The stiffness and behavior of the subgrade of a pavement are properties that must be determined for the structural design of the layers supported on it. The stress-strain behavior of soil typically exhibits a dependence on the stress state to which it is subjected. This results in the modulus of deformation acquiring different values depending on the stress state. The stresses reaching the subgrade depend on the applied load and the pavement structure above the soil under consideration (Angelone & Martínez 1999).

The parameter commonly used in design methods is the resilient modulus (M_r). This is obtained from laboratory tests where the soil is subjected to cyclic loads, and M_r values are determined for different stress levels. This allows the consideration of the nonlinear behavior of soils in evaluating the subgrade response under deviator stresses generated by the loads applied in the upper layers.

The test to determine the resilient modulus (M_r) is not economical compared to other routine tests: it requires sophisticated equipment and trained personnel to interpret results properly. This means that it is still not a commonly used test in the country, and when dimensioning a pavement, estimates must be made based on correlations with other tests such as the California Bearing Ratio (CBR) test. This often results in working with a constant value of M_r , without considering its variation with the stress state, which is inappropriate in a considerable number of cases (Cornelius et al. 2023). Designing under these assumptions can lead to deficient designs, with a relatively higher cost considering the service life of the designed pavement (Cauhapé et al. 2010).

2.2 Surface modulus

The definition of the surface modulus (SM) is often used when studying the deflection bowl of a pavement, which can be calculated from the deflection recorded by each of the sensors. A simplified method for performing this calculation was proposed by Ullidtz (1987). It is observed that, as distance away from the load increases, compression of the layers above the subgrade becomes less significant to the measured deflection at the pavement surface (AASHTO & NCHRP 1993). Therefore, it could be assumed that from a certain distance, the deflections measured on the pavement surface depend entirely on the elastic properties of the subgrade. The calculation of the SM is based on the adaptation of Boussinesq's equations, assuming a linear and homogeneous elastic medium.

According to this author, the SM can be calculated using Eq. 1.

$$SM_r = \sigma_0 \cdot (1 - \nu^2) \cdot a^2 / (r \cdot D_r) \quad (1)$$

Where a : load plate radius, r : distance at where the deflection is measured ($r \neq 0$), σ_0 : applied stress, ν : Poisson modulus, D_r : deflection measured at r distance, SM_r : SM calculated at r distance.

The variation of this modulus with distance can be used to identify whether the behavior of the subgrade is linear elastic or responds with a modulus that depends on the stress state (Ullidtz 1987). This modulus generally decreases as a sensor further away from the load is considered. Additionally, it tends towards an

asymptotic value when the subgrade has a "semi-infinite" depth extension and its behavior is linear elastic, implying an elastic modulus that does not vary with depth.

Ullidtz (1987) defines different types of trends in the variation of SM with distance to evaluate the linearity of the subgrade. Three types of subgrade behavior are established based on these trends (see Figure 2):

- Linear elastic (SM reaches an asymptotic value with distance)
- Nonlinear with increasing M_r as stress levels increase (SM decreases with distance)
- Nonlinear with decreasing M_r as stress levels increase (SM increases with distance).

To classify the behavior of the subgrade into these categories, Horak (2008) defines a surface modulus differential (SMD), obtained as the difference between the SM values considered at distances of 600 and 1200 mm respectively (based on equation 1). The importance of being able to establish the behavior of the subgrade for its characterization is emphasized as a starting point for the structural analysis of a pavement.

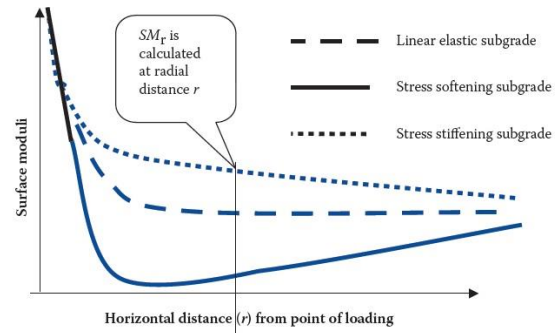


Figure 2. Surface modulus behavior over the deflection distance (Horak 2008).

Horak (2008) establishes that if the absolute value of the SMD is less than 20 MPa, it can be assumed that the behavior of the subgrade is linear elastic. If this condition is not met, the material will exhibit nonlinear behavior, where two cases are distinguished:

- The SMD is less than -20 MPa
- The SMD is greater than 20 MPa

The first case indicates that the modulus of response of the subgrade increases with depth, implying that the material has a higher modulus as the deviator stress decreases. The second case, on the other hand, occurs when this modulus decreases with depth or with the level of loading. Generally, in fine-grained soils, M_r decreases as the deviator stress increases. For granular soils, the tendency is for M_r to increase as the trace of the stress tensor increases (sum of principal stresses).

In subgrade soils, at a depth where the influence of the applied overload tends to be less than the effect of geostatic stresses, increasing depth results in a decrease in the deviator stress and an increase in the trace of the stress tensor (due to the increase in geostatic stresses). Figures 3 and 4 present typical behaviors for fine-grained and granular soils (Al-Refeai & Al-Suhaibani 2002). Based on this, it is expected that the most typical behavior in subgrades, if not linear elastic, is that the SMD is less than -20 MPa.

Jung & Stolle (1991) studied the behavior of the surface modulus obtained by FWD deflections in asphalt concrete

pavements. In many cases they observed a nonlinear behavior, obtaining an increase of SM with distance, beyond a minimum value. They concluded that the deviation of field behavior from elastostatic modeling, can be caused by some of these effects: an increase of subgrade stiffness with depth; the presence of an often-unrecorded bedrock face or the presence of discontinuities of bounded or unbounded granular materials. In these cases, it is not recommended to estimate the subgrade moduli from the surface modulus directly.

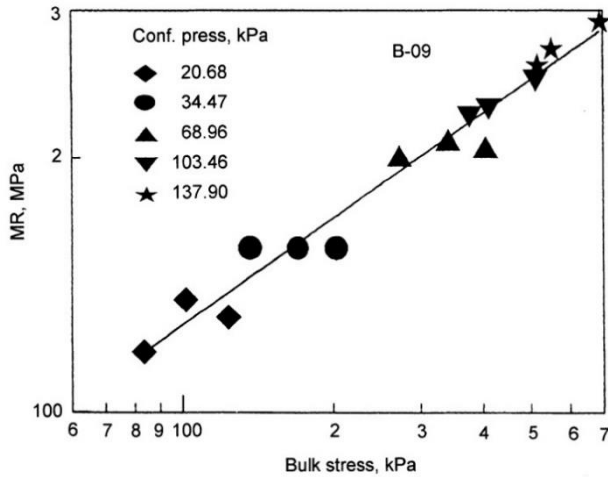


Figure 3. Typical Mr behavior of granular soils (under 20 % of fines) (Al-Refeai & Al-Suhaibani 2002).

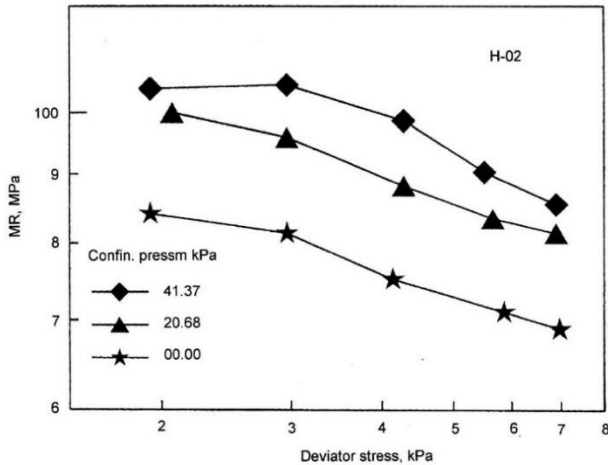


Figure 4. Typical Mr behavior for fine grained soils (Al-Refeai & Al-Suhaibani 2002).

2.3 Resilient modulus

The methodology proposed in the design guide of the American Association of State Highway and Transportation Officials (AASHTO) in 1993 (AASHTO & NCHRP 1993) adopts the definition of SM to estimate a back-calculated resilient modulus, referred to as M_{rr} . This is obtained by using equation (1) and considering a Poisson's modulus of 0.5 (incompressible material) (see Eq. (2), where P : applied load). Based on the thickness of the structural layer, a minimum distance is established sufficiently far

from the application of the load, to consider that the modulus calculated there can be associated with the response of the subgrade. It is suggested not to use a distance too far from the load so that low deflection values do not result in less accurate modulus calculation.

$$M_{rr} = P \cdot (1 - \nu^2) \cdot a^2 / (\pi \cdot a^2 \cdot r \cdot D_r) \cong 0.24 \cdot P / (r \cdot D_r) \quad (2)$$

This method correlates the M_{rr} modulus with the laboratory-derived resilient modulus M_r by multiplying the value of M_{rr} by a factor C , which depends on the type of soil and the structural layer. Generally, C takes values less than 1.

The design guide recommends calibrating this factor in case the subgrade type is different from that used in the test pavements, which was a fine-grained soil classified as A-6 with M_r values less than 20 MPa. For flexible pavements on such subgrades, a C value of 0.33 is recommended. For previously fractured portland cement concrete (PCC) pavements, due to the higher stiffness of the structure, a C value of 0.25 is recommended.

In these tests, pavements supported by granular subgrades were not studied. The guide mentions that in this type of subgrade, corrections of smaller magnitude are expected, meaning C values closer to 1.

Kam et al. (2016) summarizes different calibrated C values for flexible pavements in different states (see Table 1). It is observed that all values are greater than 0.33, and in the case of Utah DOT, which evaluated different types of soils, higher values are obtained for granular soils.

Table 1. C-Factors obtained by several US DOT (Kam et al. 2016)

Agency	C-Factor
AASHTO	0.33
Colorado DOT	0.52
Idaho DOT	0.35
Missouri DOT	0.35
Montana DOT	0.50
Utah DOT	0.55 (fine grained soil) 0.67 (coarse grained soil)
Wyoming DOT	0.645

3 SOIL CHARACTERISTICS

3.1 Geological information

Five typic geological units are selected: Dolores (DOL), Libertad (LIB), Fray Bentos (FB), Asencio (AS) and Mercedes (MER) formations. Preciozzi et al. (1985) presents the typical descriptions for these units.

DOL and LIB are Pleistocene formations and composed generally of Mudstones, loess, associated with continental sedimentation.

FB is an Oligocene formation composed of very fine sandstones and loess with variable percentage of fines, associated with a continental peridesert sedimentation.

AS is a Cretacic formation, composed of fine well sorted sandstones, argillaceous, with secondary processes of ferrification and silicification of rust color, associated with desertic continental sedimentation.

MER is a Cretacic formation composed of medium to conglomerate sandstones, badly sorted, with intercalations of limestones and lutites. MER is associated with fluvial torrential continental sedimentation.

3.2 Geotechnical properties

DOL and LIB formations present similar properties, with soils having more than 50 % of fine particles (under ASTM #200 sieve). Veroslavsky et al. (2009) collect some typic geotechnical properties of these geological units presented by Goso (2000) (see Table 2).

Table 2. Typic geotechnical properties for some geological units (Veroslavsky et al. 2009). Mean value and typical range.

Geological unit	Pass #40 (%)	Pass #200 (%)	LL*	PI**
DOL	98 (91-100)	77 (39-95)	36 (26-51)	17 (11-25)
LIB	97 (72-100)	91 (51-99)	53 (37-65)	21 (9-32)

*LL: liquid limit; **PI: plasticity index

Based on Table 2, according to AASHTO M 145-91, these soils normally classify as clayey materials ($PI > 10$), in the A-6 or A-7 groups. As the LIB Atterberg limits values are slightly greater than DOL Atterberg limits values, in general, DOL would classify as A-6 or A-7-6 and LIB as A-7-5 or A-7-6.

FB, AS and MER formations are composed of more consolidated units and present cementation in the less weathered horizon in comparison to DOL and LIB formations.

Soil from FB formation is normally composed of very fine sand and silty soils, presenting very low plasticity. Soils from AS and MER formations could present some plastic soils in the weathered horizon. The soils of these three units can contain certain percentage of gravel associated with particles of the original sedimentary rocks less affected by the weathered conditions. This variable percentage of gravel, based on the weathered condition of the unit, would change the classification group of these soils, ranging from A-1 to A-4 groups. As FB soils normally do not present plasticity but present some percentage of silts, it can be assumed that they would be classified as A-2-4 or A-2-5 depending on the percentage of fines. AS and MER soils could present certain plasticity; therefore, the classification groups of these units would be more variable, ranging from A-2-4 to A-2-7.

AASHTO (2015) recommends Mr ranges for different types of soil. Based on the assumed classification groups for each unit, Mr ranges are assigned as presented in Table 3. It is important to note that these Mr values are estimated considering the soil with a water content near the optimum water content.

Table 3. Expected soil classification and Mr range values for geological units.

Geological unit	Expected classification group	Mr range (MPa)
FB	A-2-4 or A-2-5	165 – 259
AS and MER	A-2-4 to A-2-7	148 – 259
DOL	A-6 or A-7-6	34 – 165
LIB	A-7-5 or A-7-6	34 – 121

4 ANALYSES

4.1 Objectives

The aim of this study is to analyze measurements made on asphalt concret (AC) pavements, with known base material, within the selected geological units, exhibiting a linear elastic behavior.

To achieve this, a stabilization criterion is established to classify the behavior of the subgrade. For cases with linear elastic behavior, the SM value is defined (according to the stabilization distance) and outliers are filtered out. Subsequently, C factors are adopted and the calculated values of Mr are analyzed for each geological unit.

4.2 Stabilization criterion

In a previous study, the behavior of the SM was analyzed as a function of the deflection measurement distance from the load plate for the complete database (Cornelius et al. 2023). In the study, the criterion established based on the SMD is evaluated to assess the linearity of the behavior of different subgrades. It is concluded that this criterion does not fit well for all types of soils and pavements analyzed, and that a criterion based on relative percentage variation would be more appropriate.

It is proposed to adopt a criterion to evaluate this behavior based on the percentage variation of the SM as a function of distance. If the relative variation of the surface modulus between consecutive sensors is less than 10 %, it is considered that stabilization is achieved, which is associated with linear elastic behavior of the subgrade. The classifications obtained based on this criterion are presented in Figure 5, dividing the cases according to the type of pavement base: cemented (C) or granular (G).

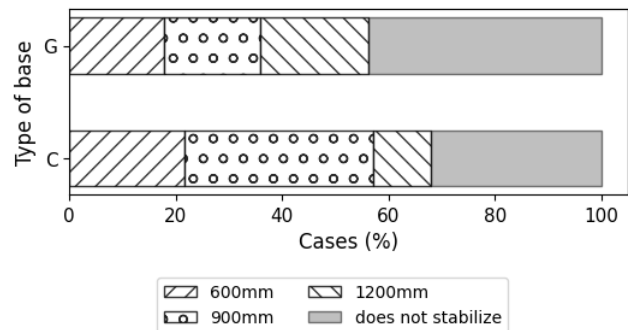


Figure 5. Stabilization distance of the data studied by type of base.

It is observed that stabilization is not achieved in 42 % of the cases. Pavements with cemented bases have a higher percentage of stabilized cases than those with granular bases. This result is consistent, since cemented bases could be associated with pavements with greater stiffness, which distribute stresses better, generating less deformations in the subgrade.

4.3 Data processing

For the identification and filtering of outliers, the criterion defined based on the interquartile range is utilized. To do so, the 25th and 75th percentiles of the SM distribution for each unit, denoted as Q1 and Q3 respectively, are calculated. The interquartile range (IQR) is defined as the difference between Q1 and Q3. The upper (UL) and lower (IL) limits are then defined using the following expressions:

$$IL = Q1 - 1.5 \cdot IQR \quad (3)$$

$$UL = Q3 + 1.5 \cdot IQR \quad (4)$$

The data outside the range established by those limits are considered outliers. The Figure 5 presents the boxplots for each unit and the outliers values outside the range.

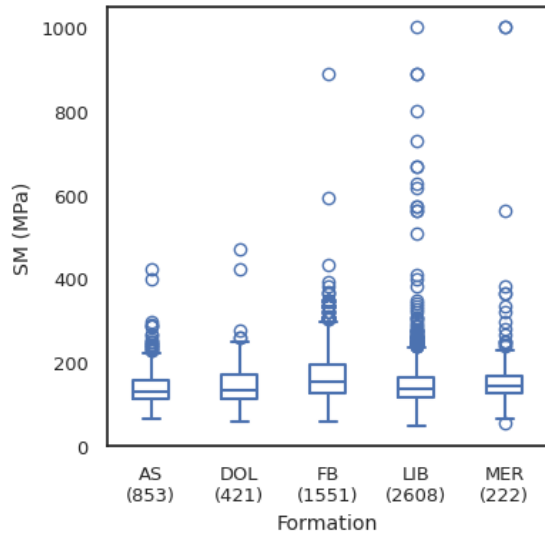


Figure 6. Outlier detection in SM values. The number under the geological unit indicates the total number of points considered.

In general, the quantity of detected outliers is less than 3%, and these values surpass the upper limit. However, in the case of the MER formation, the quantity of outliers exceeds 7%, which may be due to an insufficient amount of data for this unit. Outliers could be associated with erroneous measurements: measurements in areas of contact between geological units, or the presence of a shallow, stiffer layer beneath the subgrade. Outliers are discarded to prevent them from influencing statistical analyses.

4.4 SM values

Figure 7 depicts the distribution of SM values for each unit. In some cases, the distribution of values is not entirely symmetric

(e.g., FB and DOL formations). Table 4 presents statistical values of these distributions, dividing the cases according to the type of pavement base (cemented or granular). In all cases, low coefficients of variation (CV) are obtained (below 30 %).

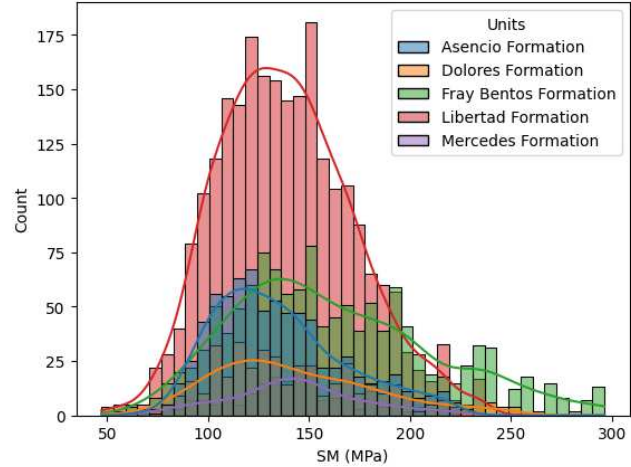


Figure 7. Histogram of SM by geological unit.

Table 4. SM statistical values.

Geological unit	Base	Count	Mean (MPa)	Min (MPa)	Max. (MPa)	CV (%)
AS	C	149	139.1	90.4	216.2	21.8
	G	606	134.3	65.6	222.2	24.3
DOL	C	0	-	-	-	-
	G	409	142.7	59.0	250.0	27.9
FB	C	182	196.0	99.7	296.3	26.7
	G	1067	155.8	57.6	296.3	29.9
LIB	C	919	157.3	82.1	235.3	18.2
	G	1461	126.9	47.2	235.3	24.2
MER	C	3	161.5	129.0	177.8	17.5
	G	202	142.8	65.8	228.6	22.8

For DOL and MER formations, there are insufficient data on pavements with a cemented base. In other cases, it can be observed that the SM value obtained is higher when a cemented base is present. This is consistent with the previous explanation because a pavement with higher stiffness distributes stresses more effectively and generates less deformation in the subgrade, resulting in a stiffer response under these conditions.

The Figures 8, 9 and 10 displays the distributions of SM for these three units. In the case of AS and FB formations, the distribution of SM values for pavements with a cemented base could be influenced by the quantity of available data. For LIB formation, distributions for both types of bases resemble normal distributions, and the notable difference in SM magnitude is evident.

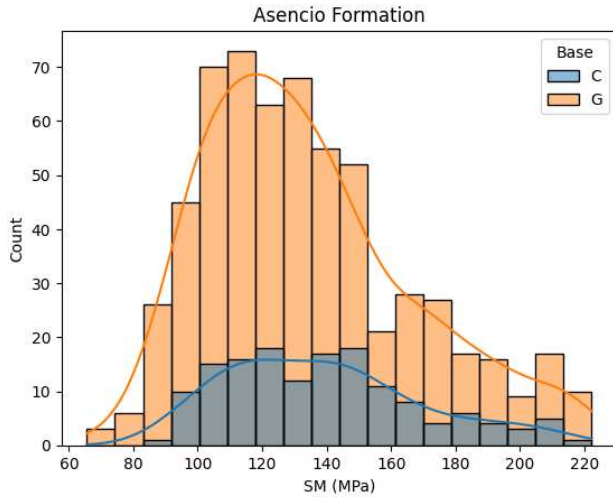


Figure 8. Histogram of SM by type of base for AS formation.

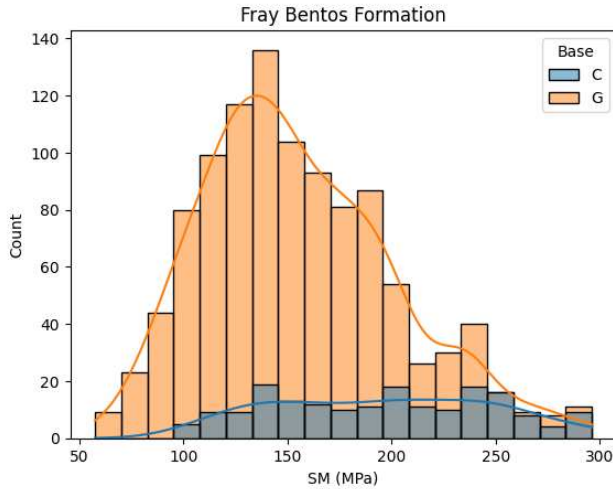


Figure 9. Histogram of SM by type of base for FB formation.

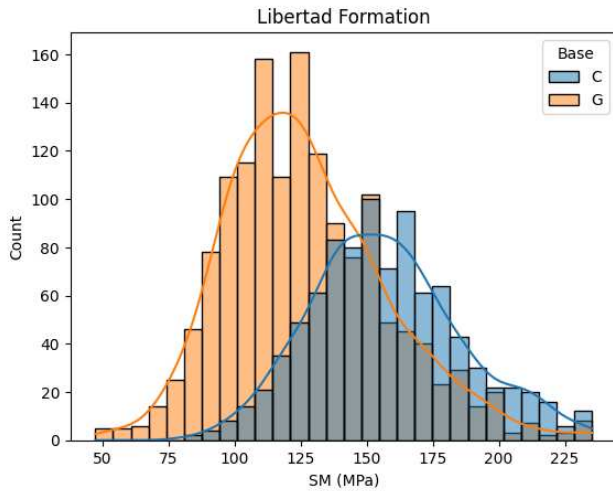


Figure 10. Histogram of SM by type of base for LIB formation.

4.5 M_r values

4.5.1 C factor

To estimate values of M_r , according to what is presented in 2.2, it is decided to adopt the values of C presented in Table 5, based on the types of subgrades and pavements.

Table 5. C factors adopted

Subgrade type	Pavement type	C factor
Fined grained soil (LIB and DOL formations)	AC with granular base	0.33
	AC with cemented or stabilized base	0.30
Coarse grained soil (AS, FB and MER formations)	AC with granular base	0.67
	AC with cemented or stabilized base	0.60

For granular soils under pavements with a granular base, the value obtained by the Utah DOT is adopted, which is consistent with expectations according to AASHTO (values closer to 1). For cemented bases, a lower value of 0.6 is adopted.

For fine soils beneath pavements with a granular base, the value of 0.33 proposed by AASHTO is adopted. For pavements with a cemented base, which have higher stiffness, a value of 0.3 is chosen as an intermediate value between the recommended values for a granular base (0.33) and those recommended for previously fractured concrete pavements (0.25).

4.5.2 Statistical values

Applying the adopted C factors yields the distributions of M_r for each unit (see Figure 11 and Table 6).

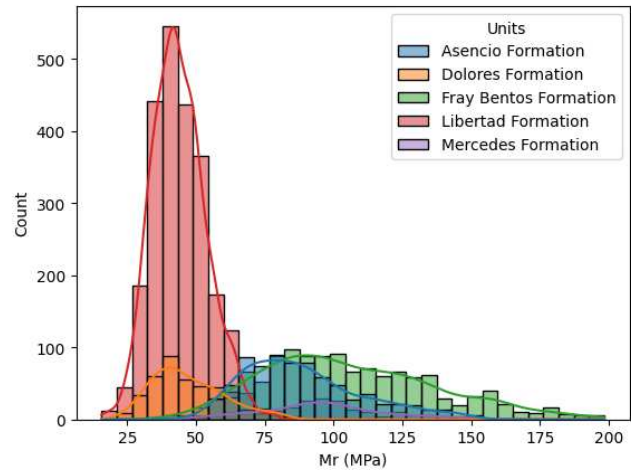


Figure 11. Histogram of M_r by geological unit.

Table 6. Mr statistical values.

Geological unit	Base	Count	Mean (MPa)	Min (MPa)	Max. (MPa)	CV (%)
AS	C	149.0	83.4	54.2	129.7	21.8
	G	606.0	90.0	44.0	148.9	24.3
DOL	C	0	-	-	-	-
	G	409.0	47.1	19.5	82.5	27.8
FB	C	182.0	117.6	59.8	177.8	26.7
	G	1067.0	104.4	38.6	198.5	29.9
LIB	C	919.0	47.2	24.6	70.6	18.2
	G	1461.0	41.9	15.6	77.6	24.1
MER	C	3.0	96.9	77.4	106.7	17.4
	G	202.0	95.7	44.1	153.2	22.8

It is observed that the difference between the values of Mr for the two types of pavements is smaller than in the case of SM.

4.5.3 Mean values

The values of Mr for each formation do not necessarily follow a normal distribution, therefore the Central Limit Theorem is used to estimate confidence intervals (CI) for its mean values. This theorem states that, under certain conditions, when sample size is sufficiently large, the sampling distribution of the mean follows a normal distribution, even if the population is not normally distributed. The sample size (n) is the same for all samples; by convention, 30 is the minimum value to consider n large enough to apply the theorem. Sample size is important because it affects the standard deviation of the sampling distribution (s): the larger the n, the smaller s.

For each formation, random sampling is made to construct the sampling distribution of the mean. An example is shown in Figure 12. The adopted sampling size is 100 and the number of samples is 10,000. Based on our database and the C factors adopted, a confidence interval (CI) for the mean value of Mr is estimated with a 95 % confidence level in Table 7.

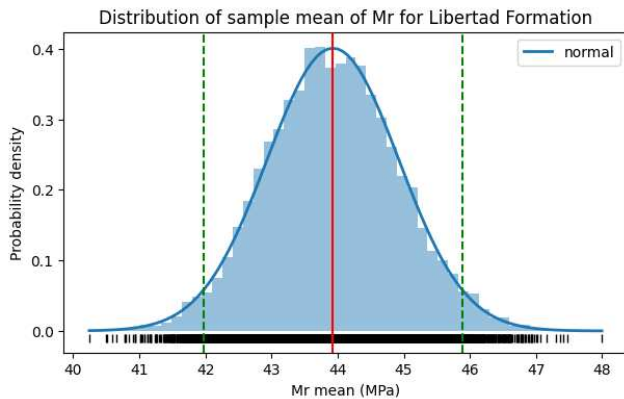


Figure 12. Sample mean distribution of Mr for LIB formation.

Table 7. CI for the mean value of Mr.

Geological unit	Mean (MPa)	s (MPa)	95% CI (MPa)
AS	88.7	2.1	[84.5, 92.9]
DOL	47.1	1.3	[44.5, 49.7]
FB	106.3	3.2	[100.1, 112.5]
LIB	43.9	1.0	[42.0, 45.8]
MER	95.7	2.2	[91.4, 100.0]

A comparison between theoretical Mr ranges and the Mr estimated from the database is presented in Figure 13. The theoretical values are presented as shaded areas; the ranges of calculated Mr values are shown by a dotted line, from the minimum to the maximum value; the CI for the mean value is indicated by black dots.

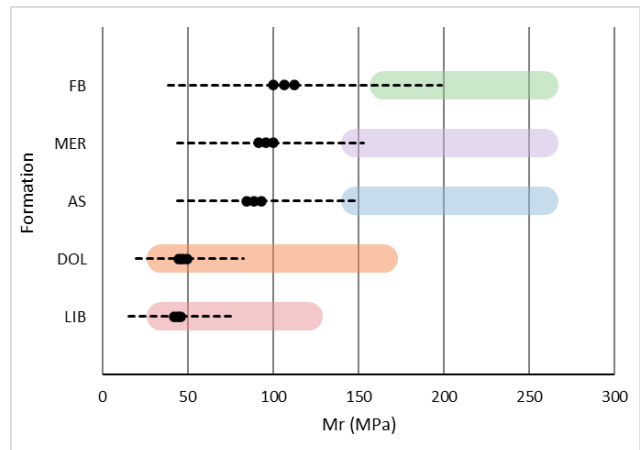


Figure 13. Mr values for each geological unit. The dotted line indicates the range of calculated values and the dots the CI for the mean value. The shaded areas show the ranges estimated in 3.2.

It is worth noting that the literature ranges are associated with soils having optimal water content. For subgrades, it is expected that they contain higher moisture content, resulting in a lower values of Mr.

For fine grained soils (DOL and LIB), the calculated ranges are almost completely on the lower side of the theoretical range. The variance could be associated with the difference between the moisture content of each case, or with the C factor value assumed. The mean values obtained are greater than 20 MPa; according to AASHTO, if this happens, it is recommended to calibrate C factor.

For the granular soils (FB, MER and AS), the calculated range is generally outside the theoretical range, obtaining lower values. The difference could also be associated with the moisture content of the soil. The theoretical values are related to a moisture content near the optimum water content, which is not necessarily equal to the natural soil moisture. The C value assumed for these soils is based on the Utah DOT, which is calibrated for coarse grain soils. Probably, the C value should be greater than the one assumed for the soils analyzed in this work.

Considering only the calculated Mr values, comparable Mr distributions are obtained for the fine grained soils. The expected

mean values of Mr are slightly higher for the DOL formation, which is somehow consistent, considering that this unit generally exhibits slightly lower plasticity than the LIB formation. For granular soil units similar ranges are obtained, with higher mean Mr values for the FB formation and lower for the AS formation.

5 CONCLUSIONS

A criterion is established to classify the behavior of subgrades under AC pavements based on deflections obtained through FWD. This criterion is defined from the relative percentage variation of the SM for consecutive sensors. It seems to be better suited for the analyzed data, compared to the criterion based on the SMD.

For the data considered (measurements made on AC pavements, with known base material, within the selected geological units), it is found that 58 % can be considered to exhibit a linear elastic behavior. Distributions of SM and Mr values are obtained for the selected geological units, which present acceptable CV values, considering that the data comes from different regions of the country and under different pavement structures.

It is observed that the SM values under pavements with cemented base (stiffer structure) are higher compared to pavements with granular base, for the same geological unit.

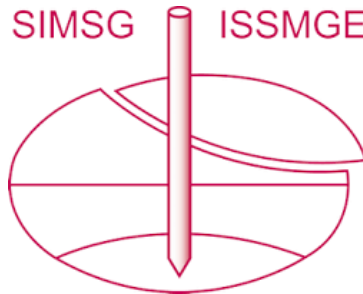
Differences are found when comparing calculated Mr values with theoretical ranges based on expected soil classifications. These differences are greater for granular soils, where the calculated values are lower.

The importance of calibrating the C factor based on laboratory tests is emphasized. Given that this test is not available in the country, it is common to assume a C value based on the bibliography, which could lead to a poor estimation of the design Mr value.

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