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Lateritic soil deformability regarding the variation of compaction energy in the construction of pavement subgrade

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Article

Keywords
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Undisturbed samples
Resilient modulus
Permanent deformation
Repeated load triaxial tests

Abstract

The performance of the subgrade towards the main deterioration mechanisms must be considered in the pavement structure design. Thus, this paper discusses the resilient modulus and permanent deformation evaluation of a pedological horizon of a Brazilian lateritic soil deposit, comparing samples compacted in the laboratory at the three compaction energies (standard, intermediate and modified) and undisturbed samples. Physical, chemical, and mechanical characterization tests were conducted. The cyclic tests were performed in repeated load triaxial tests and according to the current Brazilian standards. Five mathematical models widely used were employed to verify the resilient modulus behavior of the sample conditions, in which the Compound and Universal models showed the best correlations. For permanent deformation, the model was used, which well-expressed the plastic behavior of the analyzed conditions. Although all cases appeared to attend the shakedown criteria, some samples did not reach the deformation rate required by the standard. As the compaction energy was increased, the resilient modulus increased, and the permanent deformation decreased. Therefore, there is a substantial modification of the material behavior by increasing the compaction.

1. Introduction

Pavement consists of a system of elastic layers with finite thicknesses, composed of varied materials intended to distribute stresses vertically and horizontally until it reaches the subgrade, which receives a portion of the stresses. Pavements must present satisfactory performance even with the appearance and accumulation of defects during the service life, which are related to loss of serviceability and bearing capacity (Yoder & Witczak, 1975; Balbo, 2007; Papagiannakis & Massad, 2008; Medina & Motta, 2015).

The proper design of a pavement by the mechanistic-empirical methodology relates the selection of materials and their thicknesses to assure that, with the repeated passage of vehicles, does not occur excessive fatigue cracking of the surface does not occur (resilient deformation) and ensure that the effects of permanent deformation associated with rutting are minimized (Cerni et al., 2012; Erlingsson et al., 2017; Nazzal et al., 2020).

For paving structures, soils go through the compaction process, which consists of reducing their void content by the action of a mechanical force, increasing shear strength and stability, and reducing deformability, permeability, and erodibility (Lambe & Whitman, 1969; Van Impe, 1989; Crispim et al., 2011; Kodikara et al., 2018). In the case of tropical soils, their performance in the field is influenced by the genesis, degree of weathering, morphological characteristics, mineralogical and chemical composition, among others. The use of these soils is feasible to pavements (Guimarães et al., 2018; Lima et al., 2020; Pascoal et al., 2021; Coutinho & Sousa, 2021).

The performance of the subgrade regarding the main pavements' deterioration mechanisms should be considered when designing by the Brazilian mechanistic-empirical design guide (Brazilian Pavement Design Method – Medina). Thus, it is necessary to obtain resilient parameters and permanent deformation, in addition to physical and chemical characterization tests, considering the peculiarities of tropical soils (Medina & Preussler, 1980; Nogami & Villibor, 1995; Camapum de Carvalho et al., 2015; Freitas et al., 2020).

Resilience is the ability of a material to not keep deformations after loads cease. The resilient parameters of soil can be obtained from the resilient modulus (*RM*) test and from the models and mathematical equations. The non-linear behavior of soils is associated with several factors, such as: soil nature and grain size composition, physical state, loading condition, histcory and state of stresses, number of repetitions of deviator stress, degree of saturation, density and

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moisture of compaction, compaction method, among others (Hicks & Monismith, 1971; Bayomi & Al-Sanad, 1993; Li & Selig, 1994; Guimarães et al., 2001; Ceratti et al., 2004; Buttanaporamakul et al., 2014; Soliman & Shalaby, 2013; Biswal et al., 2016; Razouki & Ibrahim, 2017; Rahman & Gassman, 2017; Bhuvaneshwari et al., 2019; Jibon et al., 2020; Tamrakar & Nazarian, 2021; Byun & Kim, 2022).

Permanent deformations (PD) are small non-recoverable deformations accumulated by the pavement throughout its service life caused by external loads cyclically applied. The PD parameters can be obtained following DNIT 179 standards (DNIT, 2018b), supported by the model proposed by Guimarães (2009). Generally, the factors that influence the permanent deformation of soils are related to stress, loading, physical state and material type (Bayomi & Al-Sanad, 1993; Núñez et al., 2011; Salour & Erlingsson, 2015; Lima et al., 2019a, b; Zago et al., 2021; Zhang et al., 2020; Ackah et al., 2020; Silva et al., 2021).

Permanent deformation has two distinct behaviors. The deformation can increase until the material's rupture, or it can increase until an equilibrium state is reached and then the increase ceases. When the permanent deformation stabilizes and the material presents only an elastic behavior, a phenomenon called shakedown occurs (Dawson & Wellner, 1999; Werkmeister et al., 2001). This investigation is important to identify if the pavement will present progressive accumulation of plastic deformation, leading to stabilization, rupture or collapse. Quian et al. (2016) emphasize that the shakedown theory is an effective way to predict the material behavior and estimate the number of loading cycles that the material can be subjected to, without excessive rutting or rupture.

This research aims to evaluate the resilient behavior, the permanent deformation and the occurrence of shakedown of tropical lateritic soil from a deposit located in Cruz Alta, state of Rio Grande do Sul (Brazil). The material was compacted in the laboratory at standard, intermediate and modified compaction energies, and comparing it with undisturbed samples, extracted from the top layer of the embankment, employed in the field at intermediate energy.

The characterization of field compaction and the comparison with laboratory compaction energies is extremely relevant, since the compaction energy is correlated to deformations by modifying the interaction between the particles, resulting in increased density of the material, improving strength and deformability. To prove this, analysis of a pavement structure with the application of different compaction conditions will be presented, using the Brazilian mechanistic-empirical design guide (MeDiNa).

2. Lateritic soil from the northwest region of the state of Rio Grande do Sul

The soil object of this research was used in road subgrade and has tropical origin and lateritic behavior. It was extracted from a soil deposit in the municipality of Cruz Alta, northwest of the state of Rio Grande do Sul, Brazil (Figure 1a), at the margins of RS-342 highway, and was used to compose the pavement subgrade. In terms of pedological and geological aspects, the studied area presents dark red latosols of medium clayey texture and quite deep, resulting from the weathering process at the upper portions of the Paraná Basin basalt effusion, Serra Geral formation (Lemos, 1973).

The soil used in the laboratory tests was collected in the pedological horizon B, the same material used in the road embankment, as shown in Figure 1b. The undisturbed

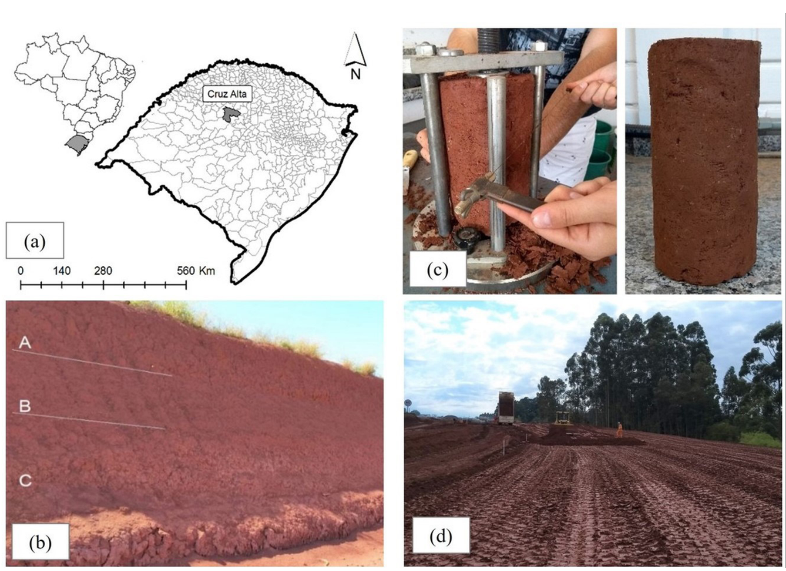

Figure 1. (a) Location of the municipality of Cruz Alta, State of Rio Grande do Sul, Brazil; (b) deposit in the margins of RS-342; (c) undisturbed sample after measures reduction process before testing; (d) road subgrade in the construction process.

samples were extracted in the top layer of the embankment (TL) and compacted in the field at intermediate energy. The extraction of the undisturbed samples from the top layer of the embankment was performed by driving cylindrical steel samplers with 150 mm diameter and 300 mm height. The procedure of extraction and reduction of the dimensions can be consulted in Pascoal et al. (2021). In Figure 1c, it is possible to observe an undisturbed sample sculpted in the dimensions ready to be tested. This procedure occurred to avoid any alteration in the structure of the specimen due to the contact between soil and sampler. Figure 1d demonstrates the road embankment in the construction phase.

In the laboratory, the soil was physically characterized by traditional Soil Mechanics tests, following the Brazilian standards (ABNT, 2016b; ABNT, 2016c; ABNT, 2016d; ABNT, 2016a). Table 1 shows the average values of the physical and chemical characterization and classification of the soil. Regarding the granulometry, about 67% of the soil particles are smaller than 0.06 mm, which means, silt and clay fractions. According to the MCT classification (DNER, 1996), the soil has clayey lateritic behavior (LG'), presenting characteristics such as high bearing capacity, low expansion, and permeability, which can provide excellent behavior for application in pavement subgrade. The comparison between MCT classification and traditional classifications of AASHTO and USCS highlights the importance of the classification methodology for tropical soils, since, according to traditional classifications, the soil under study would exhibit poor behavior towards pavement structures.

Still referring to Table 1, the presence of iron hydroxide and aluminum hydroxide, evidenced by the X-Ray Fluorescence test, is consistent with the MCT classification and the physical characteristics of soils with lateritic behavior. Also, according to the chemical analysis, the soil presented low organic matter in its composition and cation exchange capacity (CEC) lower than 6, characterizing it as clay of low activity. The presence of aluminum, higher clay content and CEC indicate the presence of the mineral kaolinite (Camapum de Carvalho et al., 2015).

As shown in Table 1, note that as the compaction energy is increased, the optimum moisture content (OMC) is reduced and there is an increase in the value of maximum dry density (MDD) of the soil. As for the undisturbed samples, the OMC and MDD values were obtained from the bulk density test using the cutting cylinder method, performed when the samples were collected. All details of the experimental tests can be consulted in Pascoal et al. (2021).

3. Methodology

The experimental program of this investigation was divided into five stages: a collection of disturbed and undisturbed samples; physical, chemical, and mechanical characterization tests; triaxial tests of repeated loads for *RM* and PD, with an analysis of five models to investigate the resilient behavior and with the model by Guimarães (2009) to obtain the permanent deformation parameters; investigation of shakedown based on the model developed by Dawson & Wellner (1999); and finally, analysis of a pavement structure with the application of different compaction conditions, using the Brazilian mechanistic-empirical design guide (MeDiNa).

Repeated load triaxial tests (RLT) were performed on samples with 100 mm diameter and 200 mm height. The equipment used for the RL and PD tests was the triaxial repeated load equipment, which has the purpose of reproducing in the laboratory the cyclic loading conditions of traffic loads on the pavement structure. The elastic deformations are measured by two Linear Variable Differential Transformer (LVDT) transducers, allowing readings up to 5 mm and used

Table 1. Physical and chemical characterization of Cruz Alta's soil.

Physical characterization		Chemical analysis and Classifications	
Liquid Limit, w_L (%)	55	MCT - Brazilian Classification	LG'
Plastic Limit, w_P (%)	44	AASHTO Classification	A-7-6
Plasticity Index, *PI* (%)	11	USCS Classification	MH
Specific Weight, γ (kN/m^3)	27.8	CEC	1.8
% coarse sand (0.6-2.0mm)	0	Basic cations Ca/K/Mg (Cmol$_c$dm^3)	0.3/0.02/0.4
% medium sand (0.2-0.6mm)	8	Saturation - Al/bases (%)	55.6/9.2
% fine sand (0.06-0.2mm)	25	Organic matter (%)	0.2
% silt (2µm-0.06mm)	26	pH	5.8
% clay (%< 2µm)	41	EDXRF - Most frequent chemical components	Fe2O3/SiO2/ Al2O3/TiO2

Compaction tests		
Energy / Condition	MDD (kg/m^3)	OMC (%)
Standard Energy (SE)	1550	28.90
Intermediate Energy (IE)	1625	25.60
Modified Energy (ME)	1652	22.60
Undisturbed Samples (TL)	1645	21.52

Note: Results of granulometry refer only to analysis with dispersant.

inside the triaxial chamber, supported under a top cap that receives the action of the deviator stress through a load cell and a piston. To study the plastic deformation, the equipment has a Rectilinear Displacement Transducer (RDT) that enables readings up to 25 mm.

Thus, the disturbed samples of horizon B were compacted at standard, intermediate, and modified energies, while the undisturbed samples of TL, compacted in the field at intermediate energy, had their dimensions reduced to the required size. The criterion for considering the sample moldings valid was the maximum variation of ± 0.5% about the OMC, as established by DNIT 134 and DNIT 179 standards (DNIT, 2018a; DNIT, 2018b). These standards do not determine an acceptable variation for MDD, thus the limit of ± 1.0% was adopted.

The stiffness of the compacted soil samples at the three energies and the undisturbed samples were evaluated by testing the resilient modulus (DNIT, 2018a). Thus, after the conditioning phase 12 pairs of deviator and confining stresses were applied, according to the standard. The test was performed in triplicate for each of the four specimen conditions, at a repeated load application frequency of 1 Hz with a 0.1 second load pulse and 0.9 seconds of rest.

The plastic deformation and subsequent obtaining of permanent deformation parameters were considered the precepts of the DNIT (2018b). After the conditioning procedure, each specimen was subjected to at least 150,000 cycles of a pair of confining and deviator stresses at a frequency of 2 Hz. Testing was performed at the following stress conditions (confining × deviator): 0.04 × 0.04 MPa, 0.04 × 0.12 MPa, 0.08 × 0.08 MPa, 0.08 × 0.24 MPa, 0.12 × 0.24 MPa e 0.12 × 0.36 MPa (Lima et al., 2019a).

Table 2 shows the mathematical models for obtaining the resilient parameters that were considered for analyzing the results, as well as the Guimarães' model used to acquire the permanent deformation parameters (Guimarães et al., 2018; Lima et al., 2020). The criterion used to evaluate the models was the best fit of the coefficient of determination (R^2).

Non-linear multiple analysis were performed using the method of minimizing the sum of squares of residuals, using the software Statistica v.10. Although there are other models for permanent deformation in the literature (Barksdale, 1972; Monismith et al., 1975; Tseng & Lytton, 1989), it was decided to use only the model of Guimarães (2009), since the current Brazilian regulations take into account the use of this model, besides it presents good correlations to the characterization and peculiarities of tropical soils (Guimarães et al., 2018; Lima et al., 2019b; Lima et al., 2020; Lima et al., 2021; Zago et al., 2021).

Then, the occurrence of shakedown was identified based on the model developed by Dawson & Wellner (1999). The results of permanent deformation were analyzed to relate the accumulated vertical permanent deformation to the rate of increase of deformation at each load cycle, following the classification of levels A (shakedown), B (plastic creep), C (incremental collapse) or AB (significant initial deformations and accommodation in the sequence).

With the parameters from the Guimarães' model and considering the compound model for resilient modulus, simulations of a typical road pavement structure were performed using the MeDiNa v 1.1.5.0 program. For this, the following conditions were considered:

- Medium traffic (*N*: $5x10^6$) e heavy traffic ($1x10^7$) (Ceratti et al., 2015);
- Primary Arterial System - maximum of 30% of cracked area and 10 mm of permanent deformation to the end of pavement life;
- Standard road axis of 8.2 tf, tire pressure 0.56 MPa;
- Project period of 10 years, average growth rate of 3.0%.

The structure considered in the analysis is composed of 5 cm asphalt concrete pavement (MeDiNa default, Class 2, RM: 6743 MPa) and of 15 cm granular material of base (MeDiNa default, RM: 381 MPa) and subgrade with variable conditions, considering the four conditions evaluated in this research.

Table 2. Models were used to obtain the resilient parameters and permanent deformation.

Properties	Models	Equation
Resilient Modulus	Confining Stress (Biarez, 1962)	$RM = k_1.\sigma_3^{k_2}$
	Stress Invariant	$RM = k_1.\theta^{k_2}$
	Deviator Stress (Svenson, 1980)	$RM \quad k_1.\sigma_d^{k_2}$
	Compound (Pezo et al., 1992)	$RM = k_1.\sigma_3^{k_2}.\sigma_d^{k_3}$
	Universal (AASHTO, 2004)	$RM = k_1.\rho_a\left(\theta / \rho a\right)^{k_2}.\left(\tau_{oct} / \rho a + 1\right)^{k_3}$
Permanent Deformation	Guimarães (2009)	$\varepsilon_{p(\%)} = \psi_1\left(\sigma_3 / \rho a\right)^{\psi_2}.\left(\sigma_d / \rho a\right)^{\psi_3}.N^{\psi_4}$

Where: *RM*: resilient modulus; σ_3: confining stress; σ_d: deviator stress; θ: principal stress; τ_{oct}: octahedral stress; ρ_a: atmospheric pressure; k_1, k_2 and k_3: resilient parameters experimentally determined; ε_p (%): plastic specific deformation; ψ_1, ψ_2, ψ_3, ψ_4,,,: parameters experimentally determined; *N*: number of load application cycles.

4. Results

4.1 Resilient modulus

The resilient modulus results were analysed by five *RM* prediction models. For this, multiple non-linear regression of the results for each test condition was performed. Each sample tested results in 12 values of resilient modulus, one for each pair of stresses. Since the test was conducted in triplicate, there are 36 values of *RM* for each sample condition. Table 3 shows the average resilient parameters of this set of 36 resilient modulus values for each sample condition.

The models that best represented the four groups of samples were the Universal (AASHTO, 2004) and the Compound (Pezo et al., 1992), both models that consider the confining and deviator stresses act on the samples. Regarding the Compound model, as the deviator stress increases, the *RM* decreases; The opposite happens for the confining stress, which, when increased, increases the *RM* value. The Biarez (1962) and Stress Invariant models showed similar behavior, because for low energies, they did not show good correlations, while for modified energy, the R^2 values were high. Svenson's model (1980) was found to be unsatisfactory for all sample conditions. Figure 2 shows the modelling of the resilient modulus for the four test conditions for confining stress and for deviator stress.

The increase in *RM* as a function of compaction energy is evident, as shown in Figure 3, which presents results of average, minimum and maximum *RM* for the Compound model. Considering the average resilient modulus, a 60.2% gain of RM was observed for samples compacted at intermediate energy compared to those compacted at standard energy.

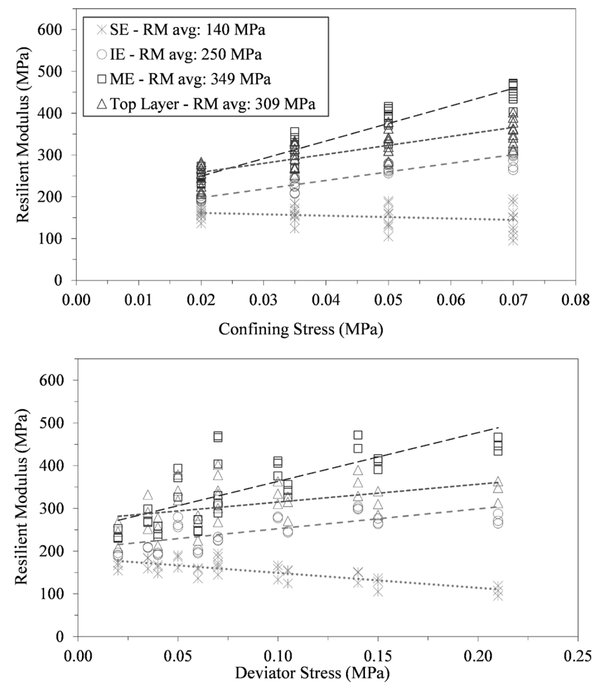

Figure 2. Comparison between deviator stress and confining stress for the four resilient modulus conditions.

Table 3. Resilient parameters for five mathematical models.

Condition	Model	k_1	k_2	k_3	R^2	*RM* avg (MPa)
Standard Energy	Confining Stress	109.76	-0.08	-	0.05	140
	Stress Invariant	122.03	-0.14	-	0.20	153
	Deviator Stress	102.98	-0.15	-	0.41	153
	Compound	144.30	0.18	-0.24	0.55	153
	Universal	391.49	0.28	-1.45	0.74	138
Intermediate Energy	Confining Stress	751.38	0.34	-	0.90	250
	Stress Invariant	397.98	0.30	-	0.80	247
	Deviator Stress	387.90	0.17	-	0.48	247
	Compound	739.20	0.34	0.00	0.90	247
	Universal	727.29	0.53	-0.69	0.93	247
Modified Energy	Confining Stress	1724.20	0.50	-	0.91	349
	Stress Invariant	721.03	0.46	-	0.89	349
	Deviator Stress	729.83	0.29	-	0.62	349
	Compound	1640.16	0.43	0.08	0.93	348
	Universal	1185.26	0.65	-0.55	0.94	349
Undisturbed Samples - TL	Confining Stress	775.59	0.29	-	0.68	309
	Stress Invariant	454.16	0.24	-	0.55	309
	Deviator Stress	433.04	0.13	-	0.28	309
	Compound	792.87	0.32	-0.03	0.69	309
	Universal	875.11	0.48	-0.76	0.72	309

With the addition of standard to modified compaction, the resilience gain became 127%. If the intermediate and modified energy are compared, an increase of 41.8% is noted. The set of undisturbed samples TL, compacted in the field at intermediate energy, presented results above expectations, since their *RM* were higher than the samples molded at intermediate energy in the laboratory, thus they were close to those obtained by the modified energy. The material was compacted at a lower than OMC, evidencing the suction effect. In summary, increasing the compaction energy reduces the voids content of the samples, increasing the *RM*, this typical behavior can result in less deformable pavements, contributing to reducing stresses in the upper layers.

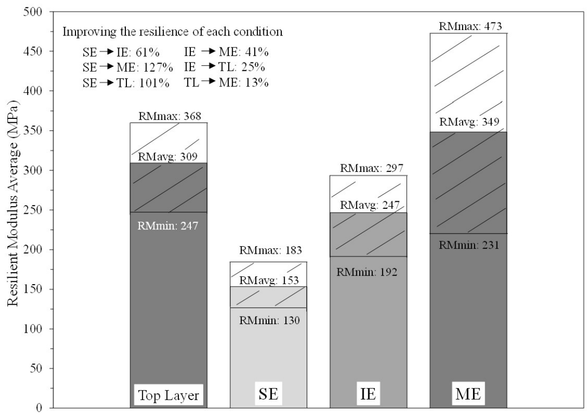

Figure 3. Resilience improvement as the compaction energy increases from average *RM*.

4.2 Permanent deformation

The permanent deformation tests were conducted for the three compaction energies and for the undisturbed samples. In Figure 4 it is possible to observe the permanent deformation accumulated versus the number of cycles of load application (150,000 cycles) for the four sample conditions. It is clear that as the stresses increased, the deformations also increased and, as the number of cycles go up, the deformations tend to stabilize. Higher stresses are generally the result of pavements with low thickness where traffic stresses are closer to the subgrade.

As expected, in all cases, the largest deformation occurred for the largest pair of stresses (σ_3: 120 and σ_d: 360 kPa), due to the magnitude and to the fact that the higher the ratio σ_1/σ_3, the larger the deformations. As the deviator stress is raised,

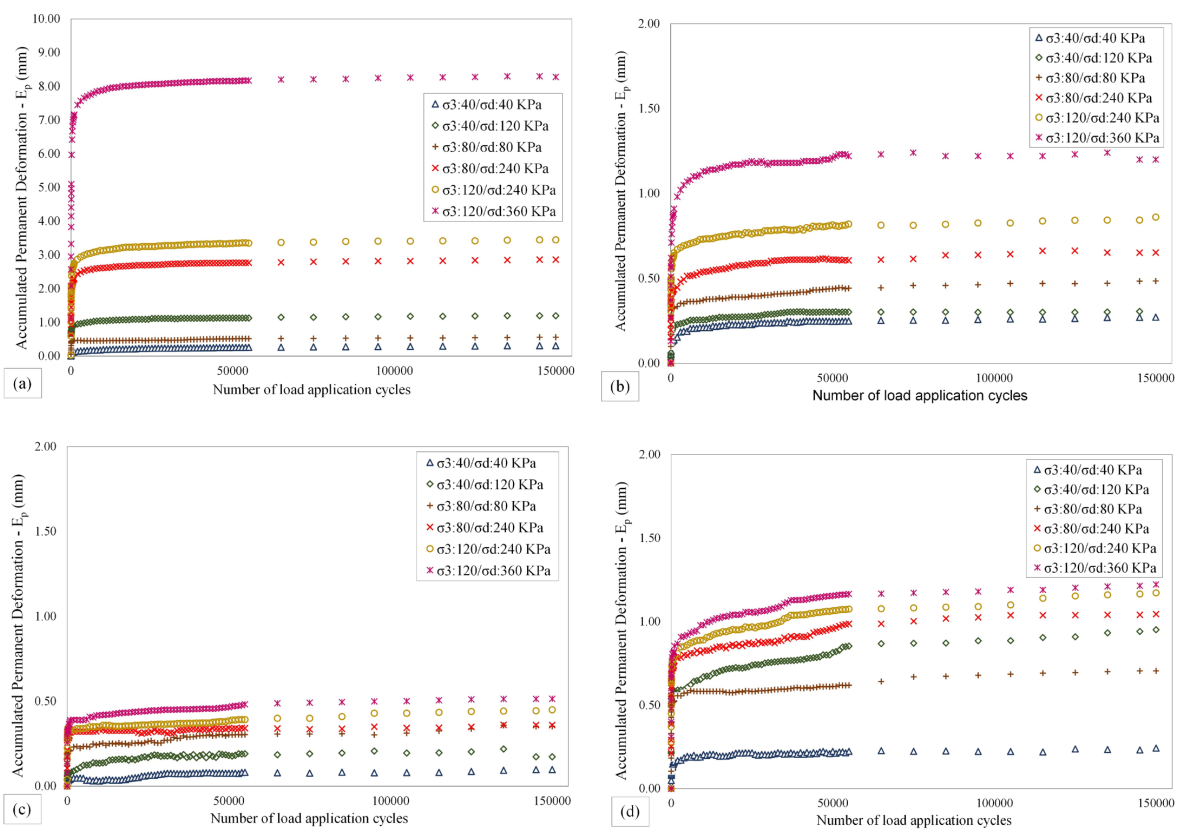

Figure 4. Accumulated permanent deformation of (a) standard energy, (b) intermediate energy, (c) modified energy and (d) undisturbed from top layer.

the permanent deformation is increased. As cycles increase, the rate of PD addition decreases. The final deformation of the specimen subjected to the highest pair of stresses was: 0.83 cm at standard energy, 0.12 cm at intermediate energy, 0.05 cm at modified energy, and 0.13 cm in the TL (compacted in the field at intermediate energy). Unlike the *RM* result, in this case, regarding accumulated deformations, the undisturbed samples behaved similarly to the samples molded in the laboratory at intermediate energy, reinforcing the need to investigate the stiffness and resistance to plastic stress separately.

In Figure 4, a similar behavior is observed in all test conditions, where the samples show a tendency to the accommodation of permanent deformation, fitting in Type I or Type II, according to the considerations of the DNIT 179 standard (2018b). Type I and II refer to materials that tend to stabilize PD with a number of loading cycles. The difference between them is that Type II presents a high value of cumulative permanent displacement before accommodation. Guimarães et al. (2001) explains that the rate of change of PD can be null with increasing cycles of loading or tends to decrease greatly when the material is close to accommodation.

After a certain number of cycles of load application, the deformations tend to become constant, presenting small variations. To investigate this phenomenon, Figure 5 shows the total deformation portion for each specimen at 1,000, 10,000, 50,000, 100,000, and 150,000 cycles of repeated loading.

Regarding the samples compacted at standard energy, except for the sample submitted to the lowest pair of stresses, at the end of the initial 1,000 cycles, the specimen had already deformed 75% to 85% of its final deformation. By the time it reached 10,000 cycles of repeated loading, the accumulated permanent deformation was already approaching 90%. A similar fact occurred for the specimens compacted at intermediate energy, which at 1,000 cycles of PD testing reached 67% to

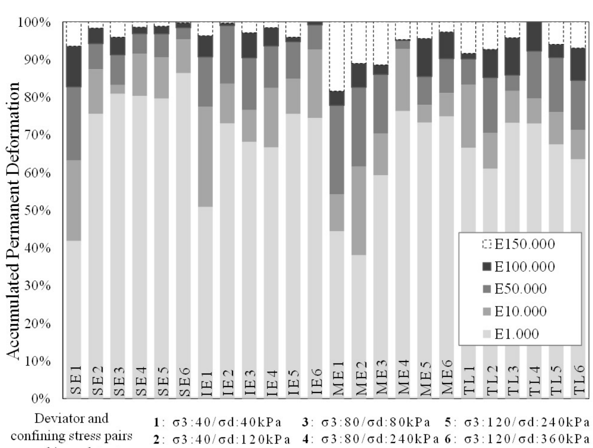

Figure 5. Part of accumulated permanent deformation as cycles increase.

76% of total accumulated deformation, and upon reaching 50,000 cycles, the samples showed more than 90% to total sample deformation.

For the samples compacted at the modified energy, if compared to the other energies, the PD rate associated with the initial PD of the samples was lower, which occurred due to the high compaction energy applied to these specimens. However, when reaching 50,000 cycles, the set of modified samples presented from 78% to 95% of the total deformation of each sample. The deformations in the initial cycles of this set of samples were smaller than the others, given the higher stiffness.

The undisturbed samples from the TL showed similar behavior to the samples compacted in the laboratory at intermediate energy. At the end of 50,000 cycles, they had already deformed 85% to 92% of the total PD. In summary, it is concluded that the undisturbed samples have behavior similar to the samples compacted in the laboratory at intermediate energy towards permanent deformation.

The data from the sets of each compaction energy and undisturbed samples were submitted to non-linear regression in order to obtain the parameters of the model by Guimarães (2009), presented in Table 2. Table 4 summarizes the parameters obtained considering the value of ρ_a equal to 0.1 MPa, considering all permanent deformations with their respective pairs of stress and load repetition cycles. The values of the coefficients of determination show that the model is adequate for the deformability of the materials under study.

With the permanent deformation values and the load application cycles, it was possible to obtain the permanent deformation variation rate and plot the graphs to analyze the occurrence of shakedown conditions in the soil, as presented in Figure 6. The material can be considered in shakedown state if the increase rate of PD reaches the order of 10^{-7}x10^{-3} meters per load application cycle.

Although all cases appeared to go into shakedown, most samples did not reach the PD rate required by the DNIT 179 standard (2018b). Possibly, with the application of a higher number of load repetition cycles, all samples would reach the rate of 10^{-7}x10^{-3} meters per cycle. However, it is not considered necessary to apply a higher number of cycles, since about 90% of the total deformation occurred in the 50,000 cycles, as shown in Figure 6.

Except for a sample of standard energy, which was subjected to the highest pair of stresses, all samples subjected to the PD test showed behavior A indicating that this layer would not contribute to the subsidence in the pavement layer (Dawson & Wellner, 1999). The sample considered as an exception fits the AB behavior (Guimarães, 2009), which encompasses materials that present significant initial deformations, followed by plastic accommodation.

It is evident that with the increase of the compaction energy, there is a substantial modification of the material's behavior when subjected to cyclic loading, and this resource can be applied more often to optimize pavement designs,

Table 4. Parameters of permanent deformation by the Guimarães model (2009).

Condition	Ψ_1	Ψ_2	Ψ_3	Ψ_4	R^2
Standard Energy	0.089	0.225	2.161	0.095	0.97
Intermediate Energy	0.083	0.699	0.517	0.103	0.93
Modified Energy	0.064	0.663	0.027	0.078	0.89
Undisturbed sample - TL	0.115	-0.042	0.516	0.092	0.87

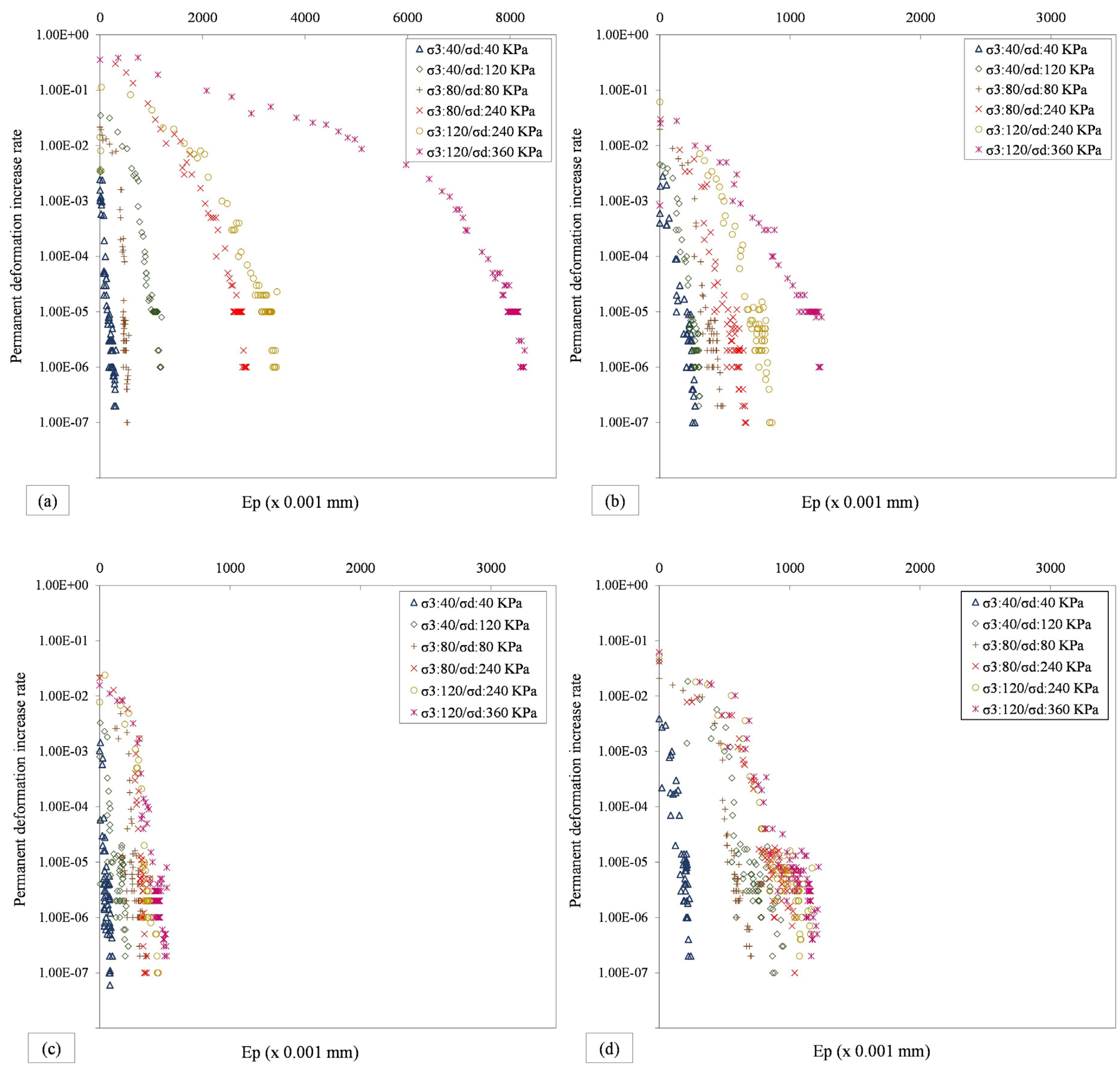

Figure 6. Shakedown occurrence research according to Dawson & Wellner (1999) of the (a) standard energy, (b) intermediate energy, (c) modified energy and (d) undisturbed of top layer.

if field compaction limitations and financial aspects are observed, which deserve specific studies.

4.3 Analysis with de Brazilian pavement design method

When analysed the application of different compaction energies of the subgrade, when dealing with cracked area, the thicknesses of the asphalt concrete pavement and granular base were fixed, to demonstrate the performance of soil conditions over the period of 10 years. The cracked area decreases as the compaction energy increases, regardless of the number of stresses on the structure (Figure 7). The rutting caused by permanent deformations decreases considerably with increasing compaction energy. Within the rutting value is encompassed the behavior of the subgrade and the granular material used in the base.

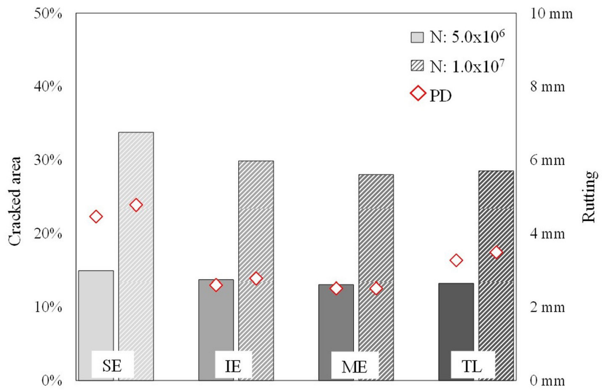

Figure 7. Comparison of four structural performance in the face of permanent deformation (rutting) and fatigue (cracked area) submitted to two *N*.

By increasing the compaction from SE to ME, the rutting throughout the design period, decreases about 43% and 48% of the permanent deformation for N of $5.0\text{x}10^6$ and $1.0\text{x}10^7$, respectively. The evaluation of the undisturbed TL samples shows that the analysed condition is similar to the conditions reproduced in the laboratory.

By AASHTO classification, this fine-grained soil would present poor to poor behavior. However, the feasibility of its use, regardless of the compaction energy, was proven from this analysis, corroborating the performance prediction previously obtained by the MCT classification.

5. Final considerations

This investigation aimed to perform the elastic and plastic characterization of a tropical and lateritic soil deposit, compacted at the three energies, and to compare these with the results of undisturbed samples from the top layer of an embankment compacted with soil from the same deposit. The comparison with undeformed samples to evaluate if what was reproduced in the laboratory is really representative of the field, in view of the importance of investigating the deformability and characterization of soils at the new Brazilian mechanistic-empirical design guide. The main conclusions were:

- According to the MCT classification, the deposit presents clayey soil with lateritic behavior (LG'). Thus, this material can be considered good for application in pavement structures, since it is well executed. Regarding the chemical characterization from the X-ray fluorescence test and chemical analysis, it was found that there is a predominance of silicon dioxide, iron oxide and aluminum oxide, which corroborates the results of the MCT classification.
- The moisture content of the undisturbed samples, compacted at the intermediate energy in the field, was close to the optimum moisture content obtained in the laboratory at the modified energy, while the maximum dry density has a value similar to the intermediate energy.
- Regarding the resilient behavior of the soils, the Compound and Universal models presented the best correlations. The increase of the *RM* with the elevation of the compaction energy was relevant. The resilient behavior of the samples compacted in the field at intermediate energy was close to the samples compacted at modified energy.
- Compaction energy is correlated with deformation. As the energy is increased from standard to intermediate or modified energy, there is a gain in the value of *RM* and a reduction in PD. With this increase in energy, the confining stress begins to influence the behavior of the soil more, something that does not occur when compacted at standard energy.
- Regarding permanent deformation, it was found that the analyzed materials tend to accommodate PD, fitting into Type I or Type II. The model of Guimarães (2009) proved to be appropriate to the deformability of the materials in question, showing good correlations. When the occurrence of shakedown was evaluated, it was found that although all cases appeared to enter plastic accommodation, some samples did not reach the PD rate required by Brazilian standards. Thus, the samples presented behavior A almost in their total, indicating that there will be no significant contributions by the soil to surface rutting.
- It is evident that increasing the energy modifies the behavior of the material, and this resource can be applied more often in pavement projects, provided that field compaction limitations and the financial aspects are observed, which deserve specific studies. The simulations by MeDiNa for two different *N* proved the effectiveness of varying the compaction energy, in relation to the performance, especially, of rutting.

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Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Paula Taiane Pascoal: conceptualization, data curation, visualization, methodology, and experimental procedures, writing – original draft, review & editing. Amanda Vielmo Sagrilo: validation, experimental procedures, writing – review & editing, validation, methodology, and experimental procedures writing – review & editing. Magnos Baroni: supervision, validation, writing – review & editing. Luciano Pivoto Specht: supervision, funding acquisition, project administration, writing – review. Deividi da Silva Pereira: funding acquisition, project administration, writing – review.

Data availability

Data is available upon request to the corresponding author.

List of symbols

k_1, k_2, k_3	resilient parameters experimentally determined
N	number of loading cycles
PD	permanent deformation
R^2	coefficient of determination
RM	resilient modulus
RLT	repeated load tests
ε_p	specific permanent deformation
θ	principal stress
ρ_a	atmospheric pressure
σ_3	confining stress
σ_d	deviator stress
σ_d / σ_3	stress ratio
τ_{oct}	octahedral stress
ψ_n	permanent deformation parameters experimentally determined

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