TRANSPORTES
www.revistatransportes.org.br
anpet
associação nacional de pesquisa e ensino em transportes
www.anpet.org.br

# Evaluation of the permanent and resilient deformation of an alternative laterite used in road pavement

*Avaliação da deformação permanente e resiliente de uma laterita alternativa utilizada em base de pavimento rodoviário*

**Ana Carolina Duarte Bona**[1], **Antonio Carlos Rodrigues Guimarães**[2]

[1]Instituto Miliar de Engenharia, Rio de Janeiro – Brasil, acduartebona@gmail.com
[2]Instituto Miliar de Engenharia, Rio de Janeiro – Brasil, guimaraes@ime.eb.br

**Recebido:**
28 de junho de 2020
**Aceito para publicação:**
25 de maio de 2021
**Publicado:**
31 de dezembro de 2021
**Editor de área:**
Jorge Barbosa Soares

**Keywords:**
Laterita.
Permanent deformation.
Resilient modulus.
Shakedown.

**Palavras-chave:**
Laterita.
Deformação permanente.
Modulo resiliência.
*Shakedown.*

DOI:10.14295/transportes.v29i3.2404



**ABSTRACT**
This study presents the geotechnical characterization of a lateritic soil extracted from an existing pavement section with double surface treatment of the BR-163 highway located in the state of Pará, Brazil. Resilient modulus and permanent deformation characterizations were performed to investigate the lateritic soil behavior during the pavement rehabilitation. The lateritic soil presented CBR (California Bearing Ratio) values below the minimum acceptable to serve as base material of pavements according to DNIT ES 098/2007. However, analysis of pavement mechanics fundamental tests - resilient modulus and permanent deformation - showed excellent behavior. The resilient modulus of the material presented good agreement with several models available in the literature and the results were compared with international reference values. The permanent deformation analysis indicated practically zero contribution of the material to whole track sinking, and the shakedown survey of the occurrence showed a tendency to accommodation in all tests performed in this study. It was concluded that the lateritic soil studied, despite having a CBR value below minimum required standards, can be used as pavement base layer without presenting risk of unsatisfactory performance.

**RESUMO**
No presente trabalho são apresentados resultados dos ensaios de caracterização geotécnica, de módulo resiliente e de deformação permanente de um solo laterítico oriundo do pavimento de um trecho existente da rodovia BR-163/PA, revestido com tratamento superficial duplo, e em fase de execução de restauração do pavimento. Mostra-se que a laterita apresentou valores de CBR (*California Bearing Ratio*) inferiores ao mínimo aceitável para constituir material de base de pavimentos de acordo com a Norma DNIT ES 098/2007, porém quando se analisou os resultados dos ensaios fundamentais da mecânica dos pavimentos - módulo resiliente e deformação permanente - verificou-se que o material apresentou excelente comportamento. São apresentados enquadramentos do módulo resiliente do material em diversos modelos disponíveis na literatura e comparados os resultados com valores de referências internacionais. A análise da deformação permanente indicou contribuição praticamente nula do material para o afundamento de trilha de roda total, e a pesquisa de ocorrência do *shakedown* mostrou tendência ao acomodamento em todos os ensaios realizados. Conclui-se que a laterita estudada, apesar de apresentar valor de CBR inferior ao mínimo exigido por norma, pode compor a camada de base do pavimento sem risco de desempenho insatisfatório.

## 1. INTRODUCTION

The peculiarities of the behavior of tropical soils are well known and internationally consolidated in behalf of the International Conference on tropical soils held in the city of Brasilia in 1984, as stated in the document COMMITTEE ON TROPICAL SOILS OF THE ISSMFE (1985). A specific standard for laterite bases was published in 1974 being replaced more recently by the DNIT ES 098/2007 standard. However, the term laterite was first used by Bunchanan (1807) to designate a material whose appearance was described as "*that of a ferruginous deposit of vesicular structure, apparently not stratified, and occurring not far below the surface*".

Despite several advances in the study of pavement mechanics using tropical soils in Brazil, few fundamental aspects of the 1974 standard were changed or updated by the 2007 standard. In fact, the main aspects associated with the value of CBR, granulometry and consistency limits are strictly identical. The main changes refer to environmental issues, such as the compliance with the recommendations in the DNER-ES 281/97 standard. In the manual for road environmental activities, there is also in the 2007 standard a concern related to the exploitation of deposits of materials, which did not exist in the 1974 standard.

Consequently, the occurrence of systematic disposal of good paving materials or unnecessary stabilization is possible, which would increase the cost of the works that are generally high. In this case study, the cost of granulometric stabilization of a ferruginous laterite at the base of the pavement track of the BR-163/PA highway was avoided, by the demonstration of the excellent mechanical behavior of the material evaluated by pavement mechanics tests, despite the non-compliance with the CBR test. Thus, in this work, the resilient modulus (RM) and permanent deformation (PD) tests were included, as well as shakedown testing by observation of field behavior.

Furthermore, despite the fact that the pavement base presented a CBR value lower than the required by DNIT standard (80%), the RM tests indicated that the material presents high values for several stress states and the PD indicated a minimal plastic deformation even for high stresses. Therefore, this material was considered suitable as a pavement base without the need for any type of chemical or granulometric stabilization.

## 2. SOME CONSIDERATIONS ABOUT LATERITES USED IN PAVEMENTS

Since the original use of the term laterite by Buchanan (1807), several other terms have been used to refer to this type of material, such as *Carapace* or *Cuirasse* in France, *Eisenkruste* or *Krusteneisenteine* in Germany, Iron Clay or Brickstone in India, and *Pizarra* or *Canga* in Brazil, including virtually all reddish iron-rich soils, whether capable of hardening or not (Charman *et al.*, 1988).

Due to this number of terms to describe laterite, regardless of the geotechnical characteristics and behavior of the material, there is a certain confusion of terms regarding the correct definitions for laterite and laterite soils. In this research, the following definitions were used:

- Laterite: soil with variable particle size distribution, predominantly pebbly, red-brown in color, composed of iron and aluminum oxide-hydroxides, such as goethite, hematite and gibbsite, and eventually quartz, formed by the total hydrolysis reaction in silicate rocks rich in alkali feldspars;
- Lateritic Fine Sandy Soil: soil with almost all of its passing mass in the 2.00 mm sieve (nº 10), classified as having a lateritic behavior by the MCT methodology (Miniature Compacted Tropical).

The observations regarding the use of laterites for road construction made by Hammond (1970) suggested that the material was subject to fragmentation during compaction. However, this material has characteristics that vary according to the percentage of minerals present in its composition. Ackroyd (1967) apud Netterberg (2014) showed that the resistance of laterite grains and their presumable field performance is directly linked to the sesquioxide content in their structure, especially of iron and aluminum oxide.

Several authors have described aspects of the mechanical behavior of laterites, as can be seen in Indraratna and Nutalaya (1991) who studied laterites from Southeast Asia and concluded that such materials have a promising use in geotechnical works in general. Mahalinga-Iyer and Williams (1996) studied laterites from Southeast Queensland, Australia, and stated that laterites behave well as a paving material, proposing a criterion for material selection. The main aspect of this criterion is the flexibility of the CBR value from 80 to 40%, but it also includes a minimum value of cohesion obtained in conventional triaxial testing (> 50kPa). Furthermore, it is only applicable to lateritic residual soil from sandstone, which was the material effectively studied by the authors.

Santos (1998) described physical and mechanical characteristics of laterites used on highways in Mato Grosso, which presented high RM and low PD. Vertamatti (1998) studied laterites used in several aerodromes in the Amazon region, showing that such material was suitable for application as pavement base layers. Guimarães (2001) found that a laterite from Brasília had a high RM and low PD, testing up to 1,000,000 charge cycles.

Guimarães and Motta (2008a and 2008b) studied a laterite from Acre which did not meet the criteria provided in the DNIT specific standard for laterites (DNIT-ES 098/07) and showed that this material had an average RM of 650 MPa, higher than that of some simple graded gravel, and total PD less than 2.0 mm, for specimens with 200 mm in height, considering different stress states.

Medina *et al.* (2006) presented results of RM and PD tests of laterites from eight pavement sections in Brazil, resulting in mean RM values for pavement base above 500 MPa. The total accumulated PD was less than 1.0 mm, considering 100,000 load application cycles with a deviatoric stress of 309 kPa and a confining stress of 103 kPa. This pair of stresses is compatible with what is expected in the field for stresses at the top of the base layers, considering the application of the standard axle load of 8.2 tf per axle, a traffic volume greater than or equal to $5 \times 10^6$ and asphalt coating minimum of 5 cm.

In Brazil, the Interrelated Survey of Road Costs (ISRC), conducted by Queiroz (1981), analyzed 45 sections in the Midwest and Southeast regions of Brazil between 1975 and 1979. This survey evaluated, among other aspects, the track-of-wheel sinking of pavements. The mean value obtained was 2.53 mm, ranging between 0.4 and 7.4 mm, and the standard deviation was 0.90. The average age of the pavements was 7.71 years, ranging between 1.5 and 20.5 years, with a standard deviation of 4.80 years. The mean of the decimal logarithm of the number of repetitions of the standard axis was 5.56, ranging between 3.20 and 7.23, with a standard deviation of 0.75. Two types of base materials were identified: laterite gravel (most common) and simple graded gravel.

Geological aspects of laterites from the Amazon can be seen in Costa (1997) and Costa (1991), in which Costa (1997) identified two distinct periods of laterization in the Amazon region. This study verified that both can be applicable in paving as long as they meet the granulometric classification and other criteria provided for in the DNIT ES 098/07 standard.

In some cases, concretions generate crusts of more than 50 cm in diameter, therefore requiring prior crushing for use on pavements.

The CBR values of laterites that meet the granulometry of the DNIT 098/2007 standard are generally higher than 80%, with cases such as the one presented in this work, which are a little lower, but still high. In general, for conventional soils, the higher the percentage of fines, the lower the CBR value. Such a relationship cannot be made in the case of RM values – which vary between 550 and 1,100 MPa – and PD which are low. The nature of the type of fines – whether of lateritic or even non-laterite behavior – also influences the RM and PD values.

## 3. PROBLEM DESCRIPTION

The restoration project of a segment of the BR-163 highway in the state of Pará in Brazil initially provided for the recycling of the coating - a compound with a highly compromised double surface treatment. The pavement showed degradation of the coating, with loss of aggregates, and detachment of plates. In the project, the incorporation of laterite in the pavement base with the addition of gravel was foreseen to increase the California Bearing Ratio (CBR) greater than 80%.

However, the high stiffening of the laterite base combined with the absence of pathologies, such as plastic deformation behavior or disaggregation, made it possible to maintain the existing base, despite the CBR value being less than 80%. Therefore, significant cost savings are obtained, without loss of the useful life of the pavement.

Figures 1 and 2 show, respectively, a general aspect of the track studied, being possible to verify the pavement condition before the restoration and the open trench for the removal of the laterite base material, highlighting the stony and reddish aspect of the laterite. The difficulty in excavating the base was associated with the cohesion of this type of soil, observed after compaction and due to the presence of iron and aluminum sesquioxides, confirmed by mineralogical analysis of the material through Energy Dispersive Spectroscopy (EDS) and and X-ray diffraction.

**Figure 1.** Track analyzed before restoration

**Figure 2.** Open trench for sample removal

## 4. METHODOLOGY ADOPTED

Tests were carried out for geotechnical characterization and determination of conventional properties, as well as fundamental tests of pavement mechanics (RM and PD) from samples of base material in trenches opened on the pavement of the BR-163/PA highway.

### 4.1. Resilient modulus assessment

The resilient modulus (RM) tests followed DNIT 134/2018-ME standard, with load application frequency of 1 Hz. The models used for framing, and their respective references, are described in Table 1.

**Table 1** – RM framing models used in this work

| Models | Authors |
|---|---|
| $Rm = k_1 \left(\frac{\sigma_3}{P_a}\right)^{k_2}$ | Dunlap (1963) |
| $Rm = k_1 \left(\frac{\sigma_{sum}}{P_a}\right)^{k_2}$ | Seed *et al.,* (1967) |
| $Rm = k_1 P_a \left(\frac{\sigma_{sum}}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3}$ | Uzan (1985) |
| $Rm = k_1 P_a \left(\frac{\sigma_{sum}}{P_a}\right)^{k_2} \left(\frac{\tau_{oct}}{P_a}\right)^{k_3}$ | Witczak e Uzan (1988) |
| $Rm = k_1 \left(\frac{\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1}{\tau_{oct}}\right)^{k_2}$ | Johnson *et al.,* (1986) |
| $Rm = k_1 \left(\frac{\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}}{\sigma_d}\right)^{k_2}$ | Tam e Brown (1988) |
| $Rm = k_1 P_a \left(\frac{\sigma_3}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a}\right)^{k_3}$ | Pezo (1993) |
| $Rm = k_1 \left(\frac{\sigma_3}{P_a} + 1\right)^{k_2} \left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | Hopkins *et al.,* (2001) |
| $Rm = k_1 P_a \left(\frac{\sigma_3}{P_a} + 1\right)^{k_2} \left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | Ni *et al.,* (2002) |
| $Rm = k_1 P_a \left(\frac{\sigma_{sum}}{P_a} + 1\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$ | NCHRP1-37A (2004) |
| $Rm = k_1 P_a \left(\frac{\sigma_{sum}}{P_a}\right)^{k_2} \left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | NCHRP1-28A (2004) |
| $Rm = k_1 P_a \left(\frac{\sigma_{sum}}{P_a} + 1\right)^{k_2} \left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | Ooi *et al,.* (2004) |
| $Rm = k_1 P_a \left(\frac{\sigma_{sum}}{P_a} + 1\right)^{k_2} \left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$ | Ooi *et al.,* (2004) |

Legend:
$Rm$ = Resilient modulus (MPa);
$k_1, k_2$ $e$ $k_3$= Regression coefficients;
$\sigma_{sum}$ = sum of principal stresses/stress invariants (kPa);
$\tau_{oct}$= Octahedral stress (kPa);
$P_a$= Reference stress (=100 kPa);
$\sigma_3$ = Confining stress (kPa);
$\sigma_d$= Deviatoric stress (kPa).

### 4.2. Permanent deformation evaluation and shakedown survey

For permanent deformation (PD) studies, specimens measuring 10 cm in diameter and 20 cm in height were molded, compacted with energy equivalent to the Intermediate Proctor test, following the procedures of DNIT 179/2018-IE and Guimarães (2009) standards. However, the stress states of the permanent deformation tests were a little different, as shown in Table 2. The stresses adopted were compatible with those expected in the field for the case of performance of the standard road axle of 8.2 tf per axle with 6.5 × $10^6$ traffic volume.

An approach similar to that presented by Guimarães (2009), Lima (2020) and Werkmeister (2001) was used for shakedown survey or accommodation of permanent deformations. In this type of shakedown approach, in which data from PD tests are used, the variation in the rate of increase in DP in function of the N number of load application cycles is analyzed.

**Table 2 –** List of triaxial repeated load tests and respective stress states used in this work to perform the PD test

| Tests | $\sigma_d$ (kPa) | σ3 (kPa) |
|---|---|---|
| 1 | 70 | 70 |
| 2 | 210 | 70 |
| 3 | 200 | 100 |
| 4 | 300 | 100 |
| 5 | 360 | 120 |

In the original approach of Werkmeister (2001), adopted by the British standard BS EM 13286-7:2004 in which it is suggested that a value of $\{\varepsilon p^1{}_{5000} - \varepsilon p^1{}_{3000}\} = 0{,}045 \times 10^{-3}$ defines the "shakedown limit" for uncemented granular materials and that a value of $\{\varepsilon p^1{}_{5000} - \varepsilon p^1{}_{3000}\} = 0{,}4 \times 10^{-3}$ defines the plastic limit, i.e its contribution to track-of-wheel sinking is not null. It should be noted that εp is the permanent deformation obtained in the DP test.

However, Lima (2020) showed that such criterion is not adequate because in the case of tropical soils there is a fourth type of behavior pattern, identified by Guimarães (2009), in which the material deforms significantly in the initial cycles, but then goes into shakedown. This pattern of behavior was called AB. Thus, it is possible that the material will not go into shakedown up to 10,000 cycles (pattern of the British standard cited), but between 10,000 and 100,000 cycles. This occurs because up to 10,000 cycles the accumulation of permanent deformation is due to the deformation of the fine fraction of the laterite, and from then on, with the gravel-gravel contact, the large fraction prevents further deformations, as shown by Guimarães (2009).

## 5. RESULTS
### 5.1. Geotechnical characterization

Table 3 presents the results of the geotechnical characteristics of the collected soil sample, as well as the compaction and CBR tests.

**Table 3** – Geotechnical characteristics of the laterite of the BR-163/PA highway

| | **Proctor Intermediate** | | **CBR and Expansion** | | **Atterberg Limits** | | | |
|---|---|---|---|---|---|---|---|---|
| Sample | Hot (%) | MDAS (g/cm³) | Expansion (%) | CBR (%) | Gs | LL (%) | PL (%) | PI (%) |
| 1 | 14.0 | 1.971 | 0.24 | 62.8 | 2.878 | 44 | 17 | 27 |

Legend:
$H_{ot}$: Optimum humidity (%);
MDAS: Maximum Dry Apparent Specific Mass (g/cm³);
CBR: California Bearing Ratio (%);
Gs: Real density;
LL: Liquid Limit (%);
PL: Plastic Limit (%);
PI: Plasticity Index (%).

As can be seen in Table 3, the pavement base laterite had a CBR value less than 80%, which is considered the minimum to constitute base material when the traffic is greater than $5\times10^6$ according to the specific standard for lateritic soils, previously mentioned, and for this reason the laterite base was denominated "alternative laterite". The CBR value below the lower limit is probably due to the percentage of fines in the natural soil in the order of 30%. Thus, once immersed in water, this fraction of fines can facilitate the penetration of the rod during the CBR test decreasing its value.

Regarding the granulometry (Figure 3), it can be seen that the material can be considered a clayey gravel with a poorly graduated curve, being composed of 41% of gravel, 31% of clay and the rest of sand and silt.

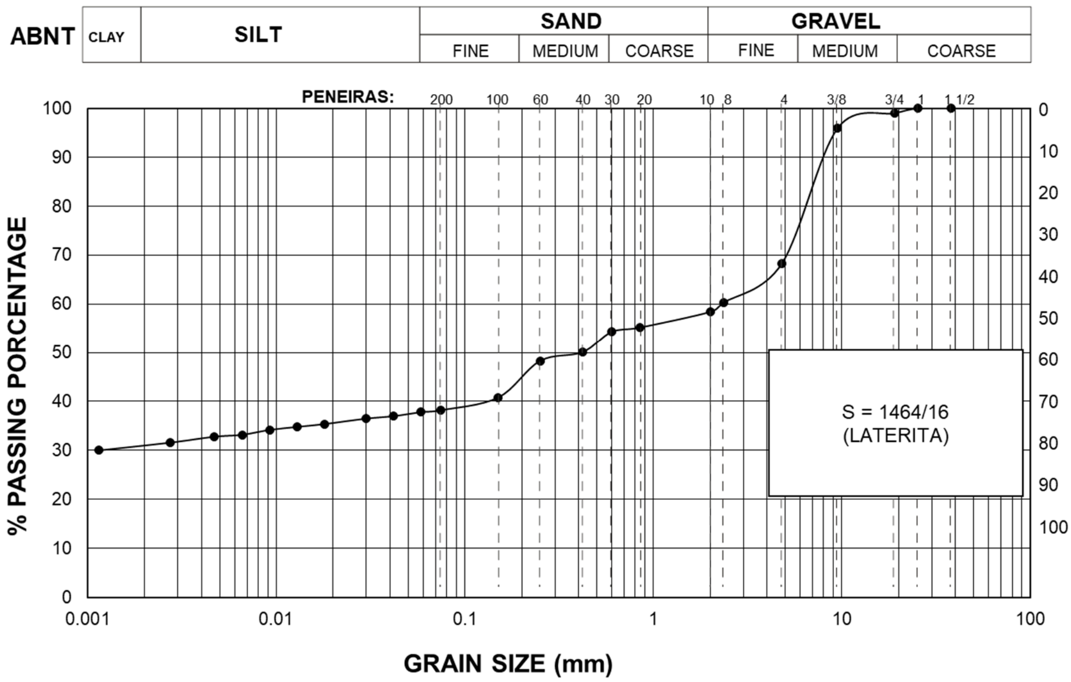


**Figura 3.** Granulometric curve

In accordance with the data presented in Table 1, the sample is classified as a clayey soil belonging to class A-7-6 in agreement with AASHTO HRB (Highway Research Board) classification. According to Santos (1998), the MCT classification is not effective to characterize soils with a high percentage of grains retained in the 2 mm sieve, so this methodology was not adopted. The MCT classification is only valid for soils that passes almost completely (95%) through a #10 sieve.

## 5.2. Resilient Modulus

Approximately 50 kg of laterite were sent to COPPE/UFRJ (Institute Alberto Luiz Coimbra of Post-Graduation and Research in Engineering / Federal University of Rio de Janeiro) laboratory to realize RM and PD tests. The mean RM values obtained in this test was 522 MPa, which can be considered high for base material. Figures 4 and 5 show the graphs of variation of RM values as a function of confining and deviatoric stresses, respectively, and in none of the cases there was a good correlation. However, it was possible to notice that there is a significant increase in the RM value as a function of the confining stress with values varying between 400 and 700 MPa. The models as a function of simultaneously both stresses, exhibited in Table 4, presented better framing as will be discussed later at the end of this item.

Table 4 presents the results of the framing of the RM values obtained for the studied laterite, considering several models available in the literature, in order to verify the best framing of the resilient behavior.

An excellent result obtained for some models is observed from data shown in Table 4 in which the value of $R^2$ was greater than 0.9. This result is superior to those calculated for the RM models as a function of only one of the stress states, as shown in Figures 4 and 5.

The composite model presented by Macedo (1996), and which is equivalent to that attributed to Pezo (1993) in Table 4, has been used in Brazil. The $R^2$ value was also higher than 0.9, but it was not the best among those analyzed.

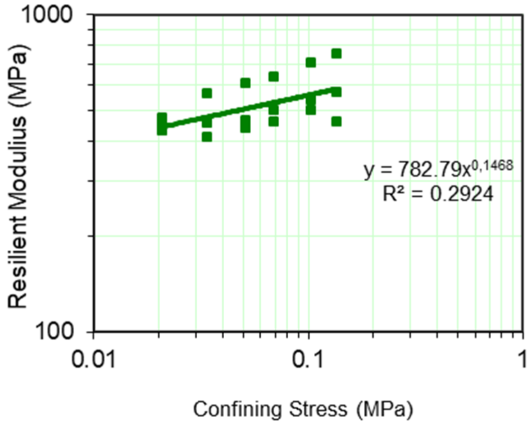


**Figura 4.** BR-163 laterite RM variation in function of confining stress

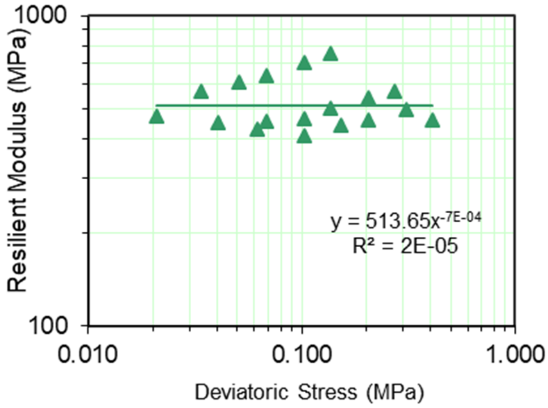


**Figura 5.** BR-163 laterite RM variation in function of deviatoric stress.

**Table 4** – Model parameters obtained from resilient modulus for the laterite of BR-163/PA highway

| Models | Authors | Correlation (R²) | Regression Constants |
|---|---|---|---|
| $Rm = k_1\left(\frac{\sigma_3}{P_a}\right)^{k_2}$ | Dunlap (1963) | 0.5408 | $k_1 = 567.01$<br>$k_2 = 0.0218$ |
| $Rm = k_1\left(\frac{\sigma_{sum}}{P_a}\right)^{k_2}$ | Seed *et al.*, (1967) | 0.3269 | $k_1 = 475.16$<br>$k_2 = 0.0895$ |
| $Rm = k_1P_a\left(\frac{\sigma_{sum}}{P_a}\right)^{k_2}\left(\frac{\sigma_d}{P_a}\right)^{k_3}$ | Uzan (1985) | 0.9690 | $k_1 = 2.4222$<br>$k_2 = 0.7465$<br>$k_3 = -0.5805$ |
| $Rm = k_1P_a\left(\frac{\sigma_{sum}}{P_a}\right)^{k_2}\left(\frac{\tau_{oct}}{P_a}\right)^{k_3}$ | Witczak e Uzan (1988) | 0.9638 | $k_1 = 2.4222$<br>$k_2 = 0.7465$<br>$k_3 = -0.5805$ |
| $Rm = k_1\left(\frac{\sigma_1\sigma_2 + \sigma_2\sigma_3 + \sigma_3\sigma_1}{\tau_{oct}}\right)^{k_2}$ | Johnson *et al.*, (1986) | 0.7755 | $k_1 = 136.62$<br>$k_2 = 0.2173$ |
| $Rm = k_1\left(\frac{\frac{\sigma_1 + \sigma_2 + \sigma_3}{3}}{\sigma_d}\right)^{k_2}$ | Tam e Brown (1988) | 0.7694 | $k_1 = 542.43$<br>$k_2 = 0.4733$ |
| $Rm = k_1P_a\left(\frac{\sigma_3}{P_a}\right)^{k_2}\left(\frac{\sigma_d}{P_a}\right)^{k_3}$ | Pezo (1993) | 0.9630 | $k_1 = 6.7956$<br>$k_2 = 0.4794$<br>$k_3 = -0.3131$ |
| $Rm = k_1\left(\frac{\sigma_3}{P_a} + 1\right)^{k_2}\left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | Hopkins *et al.*, (2001) | 0.9774 | $k_1 = 450.78$<br>$k_2 = 1.2042$<br>$k_3 = -0.5944$ |
| $Rm = k_1P_a\left(\frac{\sigma_3}{P_a} + 1\right)^{k_2}\left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | Ni *et al.*, (2002) | 0.9775 | $k_1 = 4.5078$<br>$k_2 = 1.2041$<br>$k_3 = -0.5943$ |
| $Rm = k_1P_a\left(\frac{\sigma_{sum}}{P_a} + 1\right)^{k_2}\left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$ | NCHRP1-37A (2004) | 0.9129 | $k_1 = 5.1751$<br>$k_2 = 0.5677$<br>$k_3 = -1.2815$ |
| $Rm = k_1P_a\left(\frac{\sigma_{sum}}{P_a}\right)^{k_2}\left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | NCHRP1-28A (2004) | 0.9449 | $k_1 = 5.7284$<br>$k_2 = 0.6229$<br>$k_3 = -0.9588$ |
| $Rm = k_1P_a\left(\frac{\sigma_{sum}}{P_a} + 1\right)^{k_2}\left(\frac{\sigma_d}{P_a} + 1\right)^{k_3}$ | Ooi *et al.*, (2004) | 0.9796 | $k_1 = 3.2228$<br>$k_2 = 0.9412$<br>$k_3 = -1.0611$ |
| $Rm = k_1P_a\left(\frac{\sigma_{sum}}{P_a} + 1\right)^{k_2}\left(\frac{\tau_{oct}}{P_a} + 1\right)^{k_3}$ | Ooi *et al.*, (2004) | 0.9569 | $k_1 = 3.0198$<br>$k_2 = 0.8682$<br>$k_3 = -1.4392$ |

The results obtained for the first specimen in the RM test ranged from 409 to 750 MPa with an average of 522 MPa. Replicas were performed and the results showed a variation of less than

10%, which can be considered as the repeatability rate, therefore the others were canceled for presentation in the article.

To perceive the order of magnitude of the RM value of the BR163 laterite, a comparison was made between the value obtained for this material and some well-graded gravel from quarries in Rio de Janeiro (Ramos and Motta, 2004), as shown in Table 5. For this comparison, the RM model was used as a function of the confining stress.

**Table 5** – Comparison of RM values of some well-graded gravel and BR-163/PA laterite (Ramos and Motta, 2004)

| Quarries | Rocks | $R_m = K_1(\sigma_3)^{K_2}$ | RM for $\sigma_3$ = 100 (MPa) |
|---|---|---|---|
| Concrebrás | Quartz-monzonite | $R_m = 782.9(\sigma_3)^{0.37}$ | 334 |
| Vigné | Trachyte | $R_m = 1018.1(\sigma_3)^{0.42}$ | 387 |
| Bangu | Granite | $R_m = 731.5(\sigma_3)^{0.37}$ | 312 |
| Pena Branca | Quartz-diorite | $R_m = 896.8(\sigma_3)^{0.41}$ | 349 |
| BR163 Laterite | | $R_m = 782.9(\sigma_3)^{0.146}$ | 559 |

As can be seen from the data in Table 5, the BR163 laterite presented a higher RM than the well-graded gravel used as reference. In general, laterites have a much higher RM than graded gravel (Guimarães, 2009; Santos, 1998).

Typical values for RM are recommended to assist pavement designers. For example, Table 6 shows typical RM ranges and values of granular materials with optimum moisture content according to the unified soil classification system SUCS (NCHRP 1-37A, 2004).

**Table 6 –** Typical ranges of values of resilient modulus of granular materials with optimum moisture content (NCHRP 1-37A, 2004)

| Description | Resilient modulus (MPa) | | Average (MPa) |
|---|---|---|---|
| | Minimum | Maximum | |
| Gravel or sandy gravel, well-graded | 272 | 290 | 283 |
| Gravel or sandy gravel, poorly graded | 245 | 276 | 262 |
| Clay gravel or gravelly sandy clay | 166 | 259 | 148 |
| Sandy or sandy gravel, well-graded | 193 | 259 | 221 |
| Sandy or sandy gravel, poorly graded | 166 | 228 | 193 |
| Sedimentary sand or sedimentary gravelly sand | 193 | 259 | 221 |
| Sandy clay or sandy gravelly clay | 148 | 193 | 166 |

The analyzed laterite can be classified as gravel or sandy gravel, and so forth would present RM values between 245 and 290 MPa, but test results presented values between 400 and 700 MPa, which are much higher. This fact indicates that sometimes international references may not be suitable for framing the behavior of tropical soils.

The Pavement Infrastructure Guide of AUSTROADS (2012) also provides the recurrent RM values for granular materials as indicated in Table 7. The guide still suggests using maximum RM values when no more reliable information is available.

**Table 7** – Recurring values for uncemented granular materials (AUSTROADS, 2012)

| Materials | Resilient Modulus (MPa) | | Average (MPa) |
|---|---|---|---|
| | Minimum | Maximum | |
| High standard crushed rock base | 300 | 700 | 500 |
| Normal standard crushed rock base | 200 | 500 | 350 |
| Gravel of basic quality | 150 | 400 | 300 |
| Materials with normal standard for subbase | 150 | 400 | 250 |

Analogously to NCHRP 1-37A (2004), the reference values would be between 150 and 400 MPa, therefore lower than that obtained for the laterite studied in the present work.

### 5.3. Permanent deformation

Figure 6 shows the result of the PD test, considering the various stress states used, shown in Table 2, in which it was possible to verify a maximum total permanent deformation of 1.7 mm (in test 5). This deformation was very low taking in consideration that the height of the tested specimens was 200 mm. Such deformations with low values indicate the good behavior of this material.

Therefore, considering a 200 mm thick layer of laterite of the BR-163 and subjected to stresses of the test, a contribution of only 1.7 mm of this layer was observed for the total track-of-wheel sinking of the pavement. The maximum value allowed can be considered in the order of 1" or 12.5 mm. Therefore, the contribution of the lateritic base is small being equivalent to 13.6% of the total.

It is important to emphasize that similar results for laterites were obtained in the works of Guimarães (2009) and Santos (1998) who investigated laterites of various origins.

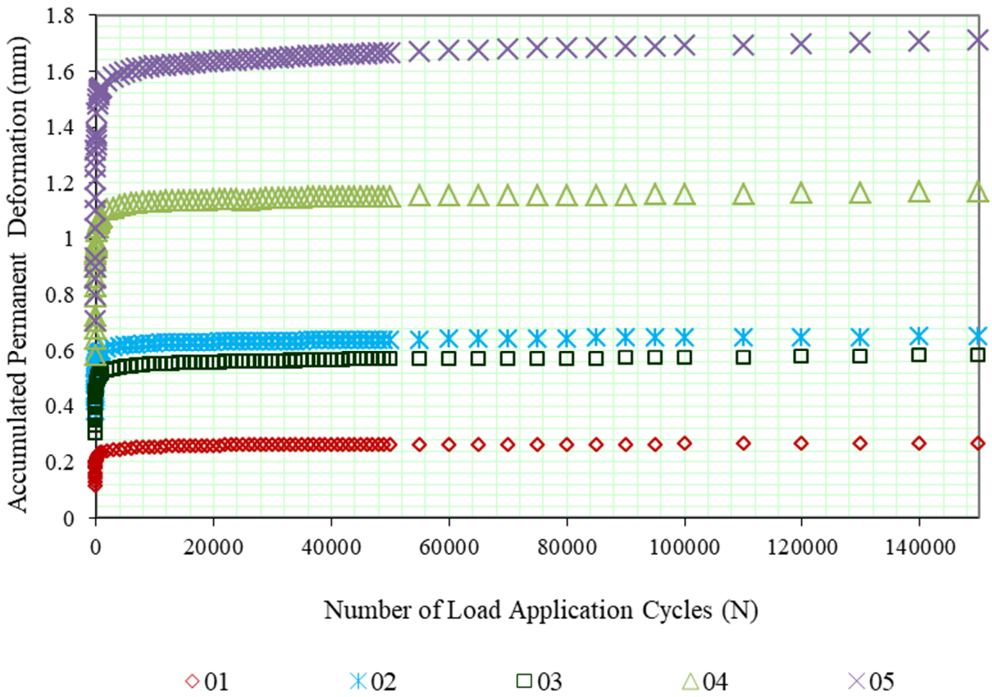


**Figure 6.** Variation of accumulated permanent deformation as a function of the number of load application cycles (N)

Another result that can be evaluated from the permanent deformation test is the analysis of the rate of increase of this variable as a function of the number N. The best analysis refers to the search for the occurrence of shakedown, or accommodation of plastic deformations, using triaxial tests of repeated loads and the graphic representation of Dawson and Wellner, presented in Figure 7, as described in Werkmeister (2003) and Werkmeister *et al.* (2001). This study is consolidated in the British standard, already mentioned, and has been internationally applied by several authors, with the aim of ranking the behavior of soils available for paving.

According to Guimarães (2009), when the rate of increase of permanent deformation in function of the number of cycles (N) reaches a value of approximately $10^{-7}$ meters per load cycle, the material is said to have gone into shakedown, which means that there are no more permanent deformations. This situation minimizes wheel track sinking which is an extremely favorable characteristic for a paving material. It can be seen in Figure 7, that the laterite of BR-163 highway went into shakedown in all tests carried out involving stresses even higher than the maximum observed for standard wheel load (8.2 tf) on this type of pavement.

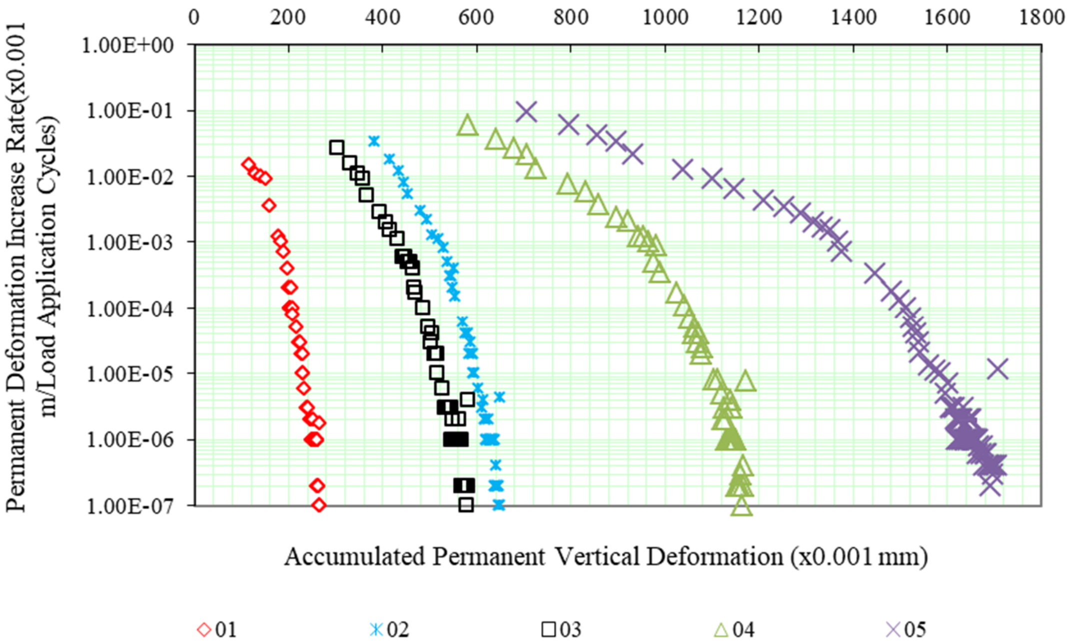


**Figure 7.** Research on the occurrence of shakedown for the laterite of the BR-163 highway

## 6. FINAL CONSIDERATIONS

The pavement laterite of a BR-163 highway track in the state of Pará studied in the present work presented a high RM value and a low PD value. This fact combined with the almost null expansion observed in the CBR test confers excellent condition to constitute pavement base material.

Regarding the occurrence of shakedown, it was possible to observe that the rate of increase of permanent deformation was close to zero in all tests, even for higher stresses due to the clear tendency of the accumulated deformation curve to become parallel to the horizontal axis. Therefore, the material has a high shakedown limit that confers good behavior in terms of PD.

Considering the results of the RM and PD tests, it is observed that the use of granulometric stabilization with graded gravel to increase the CBR value would result in an increase of up to 30% of gravel in the project, which would lead to an increase in the unit cost of the basic service of an equal proportion. The cost savings by using lateritic soil as pavement base considering the entire segment of the highway to be restored were of approximately R$ 2,000,000.00 (two million reais according to the Brazilian “Sistema de Custos Referenciais de Obras (SICRO)” of 2016 - Reference Costs System for Works.

### REFERENCES

AUSTROADS (2012) *A guide to the structural design of road pavements.*

BS EN 13286-7 (2004) *Unbound and Hydraulically Bound Mixtures - Part 7: Cyclic Load Triaxial Test for Unbound Mixtures*. British Standards Instituition, London.

Buchanan, F. (1807) *A journey from Madras through the countries of Mysore, Canara and Malabar*. Book. Vol. 2 East India Company, London.

Charman, J. H. (1988) Laterite in road pavements*. London Construction Industry Research and Information Association Special Publication.* v. 47, p. 71.

COMMITTEE ON TROPICAL SOILS OF THE ISSMFE (1985). Peculiarities of Tropical Soils Used as Construction Materials. *First International Conference on Geomechanics in Tropical Lateritic and Saprolitic Soils*.

Costa, M. L. da (1991) Geological Aspects of Amazon Laterites. *Brazilian Journal of Geosciences.* v. 21, n. 2, p.146 – 160.

Costa, M. L. da (1997) Lateritization as a major process of ore deposit formation in the amazon region. *Journal of Explor. Mining Geol.* v. 6, n. 1, p. 79-104.

DNER-ES 281/97 (2006) *Manual for environmental road activities*. Rio de Janeiro.

DNIT-ES 098 (2007) Pavimentation – Granulometrically Stabilized Base Using Lateritic Soil. *National Department of Transport Infrastructure*, Rio de Janeiro.

DNIT-IE 179 (2018) Pavimentation – Soils – Determination of Permanent deformation – Test instructions. *National Department of Transport Infrastructure*, Rio de Janeiro.

Dunlap W. S. (1963) *A report on a mathematical model describing the deformation characteristics of granular materials.* Technical report 1, project 2-8-62-27, Texas A&M University.

Guimarães, A. C. R (2001) *A Study About Permanent Deformation on Soils and the Shakedown Theory Applied to Flexible Pavements*. Master thesis. Civil Engineering Program, COPPE/UFRJ (Institute Alberto Luiz Coimbra of Post-Graduation and Research in Engineering / Federal University of Rio de Janeiro). Rio de Janeiro.

Guimarães, A. C. R. e L. M. G. da Motta (2008a) A Study on Permanent Deformation in Concreted Lateritic Soils in Southwestern Amazonia through Triaxial Repeated Load Tests. Annals*. IV Portuguese-Brazilian Geotechnical Congress,* Coimbra, Portugal.

Guimarães, A. C. R. e L. M. G. da Motta (2008b) A Study on Deformation of a Laterite in the State of Acre through Triaxial Repeated Load Tests. Annals. *15ª Urban Paving Meeting*, ABPv, Salvador.

Guimarães, A. C. R. (2009) *U A Mechanistic Empirical Method to Predict Permanent Deformation on Tropical Soils on Pavements*. PhD. Thesis. Civil Engineering Program. COPPE/UFRJ (Institute Alberto Luiz Coimbra of Post-Graduation and Research in Engineering / Federal University of Rio de Janeiro). Rio de Janeiro.

Hammond, A. A. (1970) A study of some lateritic gravels from Kumasi district. Build. *Road Research*, v. 5, p. 17.

Hopkins T. C., T. L. Beckham, L. Sun e B. Ni. (2001) *Resilient modulus of Kentucky soils*. Research report, University of Kentucky Transportation Center, College of Engineering, Lexington, Kentucky.

Indraratna, B. e P. Nutalaya. (1991) Some Engineering Characteristics of a Compacted Lateritic Residual Soils. *Journal of Geotechnical and Geological Engineering*, v.9, p. 125 – 137.

Johnson T. C., R. L. Berg e A. Dimillio. (1986) Frostaction predictive techniques: a overview of research results*. Transp Res Rec*. v.1089, p. 147–6.1

Lima, C. D. A. (2020) *Evaluation of Permanent Deformation on Paving Materials from Triaxial Repeated Load Tests.* PhD. Thesis. Civil Engineering Program. COPPE/UFRJ (Institute Alberto Luiz Coimbra of Post-Graduation and Research in Engineering Federal University of Rio de Janeiro). Rio de Janeiro.

Macêdo, J. A. G. (1986) *Interpretation of deflectometric tests for structural evaluation of flexible pavements.* PhD. Thesis. Civil Engineering Program. COPPE/UFRJ (Institute Alberto Luiz Coimbra of Post-Graduation and Research in Engineering Federal University of Rio de Janeiro). Rio de Janeiro.

Mahalinga-Iyer, U. e D. J. Williams. (1996) Properties and Performance of Lateritic Soil in Road Pavements. *Journal of Engineering Geology.* v. 46, p. 71-80.

Medina, J., L. M. G. Motta e J. G. D. dos Santos. (2006) Deformability Characteristics of Brazilian Laterites. *Geotechnical and Geological Engineering*. v. 24, p. 949-971. DOI 10.1007/s10706-005-8507-z.

Netterberg, F. (2014) Review of Specifications for the use of Laterites in Road Pavement*. AFCAP/GEN/124*. Crown agents, AFCAP and UKaid.

NHCRPP 1-28A. (2004) Laboratory determination of resilient modulus for flexible pavement design. *National Cooperative Highway Research Program Research Results Digest No. 285*.

NHCRPP 1-37A. (2004) Summary of the 2000, 2001, and 2002 AASHTO guide for the design of new and rehabilitated pavement structures. *National Cooperative Highway Research Program.*

Ni B., T. C. Hopkins, L. Sun e T. L. Beckham. (2002) Modelling the resilient modulus of soils. Anais*. 6th international conference on the bearing capacity of roads, Railways and air field*. v. 2, p. 1131–42.

Ooi P.S.K, A. R. Archilla e K.G. Sandefur. (2004) Resilient modulus for compacted cohesive soils. *Transp Res Rec*, v. 1874, p. 115–24.

Pezo R. (1993) A general method of reporting resilient modulus tests of soils, a pavement engineer's point of view. Anais. *72nd annual meeting of transportation research board.*

Queiroz, C. A. V. (1981) *Performance prediction models for pavement management in Brazil*. PhD thesis. University of Texas, Austin, USA.

Ramos, C. R e L. M. G. Motta. (2004) Study of the resilient behavior of various soils and gravel in the city of Rio de Janeiro. Annals. *35th RAPV of ABPv*. Rio de Janeiro, October 2004.

Santos, J. D. G. (1998). *Contribution for Study of Granular Lateric Soils in Pavement.* PhD. Thesis. Civil Engineering Program. COPPE/UFRJ (Institute Alberto Luiz Coimbra of Post-Graduation and Research in Engineering / Federal University of Rio de Janeiro). Rio de Janeiro.

Seed, H. B., F. Mitry, C. L. Monosmith e C. K. Chan. (1967) Prediction of pavement deflection from laboratory repeated load tests. *National Highway Cooperative Research Program Report*. v. 35.

Tam W. A. e S. F. Brown. (1985) *Use of the falling weight deflectometer for in situ evaluation of granular materials in pavements.* Anais. 14th ARRB conference. p. 155–63.

Uzan J. (1985) Characterization of granular materials. *Transp Res Rec*, v. 1022 p. 52–9.

Vertamatti, E. (1988). *Contribution to Geotechnical Knowledge of Amazonian Soils Based on Airport Research and MCT and Resilient Methodology*. PhD thesis. Air Transport Program. Technological Institute of Aeronautics. São José dos Campos, Brazil.

Werkmeister, S., A.R. Dawson e F. Wellner. (2001). Permanent Deformation Behavior of Granular Materials and the Shakedown Concept. *Transportation Research Record,* nº 01-0152, Washington, DC.

Werkmeister, S. (2003). *Permanent Deformation Behavior of Unbound Granular Materials in Pavement Constructions.* PhD thesis. Civil Engineering Program. Technische Universität Dresden. Alemanha.

Witczak M. W. e J. Uzan. (1988). *The universal airport pavement design system, Report I of V: Granular material characterization.* Master thesis Civil Engineering Program. University of Maryland. Maryland.