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Mechanistic evaluation of a monitored pavement constructed with asphalt surface treatment

Avaliação mecanicista de um trecho monitorado de pavimento asfáltico construído com tratamento superficial duplo

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ABSTRACT

This paper aims to present the preliminary results of a monitored highway track in the Brazilian state of Goiás, incorporating mechanistic analyses in order to contribute to the database of materials, technologies and performance for an asphalt pavement executed with Double Asphalt Surface Treatment (DAST) as the surface layer. In this study, laboratory tests were performed with the materials in terms of characterization, mechanical and resilient behavior for each layer, and checking the behavior of an improved lateritic gravel with cement in three different contents. The pavement test section was monitored during execution and up to 6 months after opening to traffic by determining deflections, macro and micro textures, rutting measurement, traffic evaluation and identification of apparent defects. The DAST design was also evaluated. The stresses and deformations that developed along the layers were obtained and the field modules were determined by back analysis. The results showed a mechanical performance gain with the addition of cement in the gravel. The results also show some problems during the pavement construction, the emergence of pathologies, high deformations in the lower layers, and emulsion oversizing in the DAST.

RESUMO

Este artigo objetiva apresentar os resultados preliminares de um trecho monitorados em Goiás, incorporando análises mecanicistas de modo a contribuir com o banco de dados de materiais, tecnologias e desempenho de um pavimento asfáltico executado com Tratamento Superficial Duplo (TSD). Para isso, nesta pesquisa foram realizados ensaios laboratoriais com os materiais em termos de caracterização, comportamento mecânico e resiliente dos componentes de cada camada, verificando também o comportamento do cascalho melhorado com cimento em três teores diferentes. A seção de teste do pavimento foi monitorada durante a execução e até 6 meses após liberação do tráfego, por meio da determinação das deflexões, das macro e microtexturas, da medida da trilha de roda, da avaliação do tráfego e da identificação dos defeitos aparentes. O dimensionamento do TSD também foi avaliado. As tensões e deformações desenvolvidas ao longo das camadas foram obtidas e os módulos de campo determinados por meio de retroanálise. Os resultados mostraram ganho de desempenho mecânico com a adição de cimento no cascalho, falhas na execução, surgimento precoce de patologias, elevadas deformações nas camadas inferiores à base e superdimensionamento de emulsão no TSD.

1. INTRODUCTION

The sustainable development of a country depends on a transport system that combines productivity and quality. A regular infrastructure review is necessary for the system to increase its performance and adapt to new demands. In Brazil, road transport is the most used for both cargo and people. However, in Brazil only 13% of the highways are paved, and 57% have their general status classified as reasonable, bad or very bad (CNT, 2019). The fact that more than half of the built pavements have problems is partly due to the design method that is still used for asphalt pavements, which involves a calculation process that is no longer adequate for current characteristics. In addition, other factors can contribute to the rapid degradation of Brazilian pavements such as: a lack of in-depth studies conducted for the materials used, a lack of adequate execution control, and the failure to monitor the performance of the post-construction structure.

There is also a gap in the production of research that studies the main themes involved in this process. The survey by Santiago and Soares (2015) of studies carried out in Brazil between 1990 and 2015 shows that, while the characterization of materials represents 60% of the total research, the sum of important parameters for design such as traffic, environmental factors and performance corresponds to only 30%.

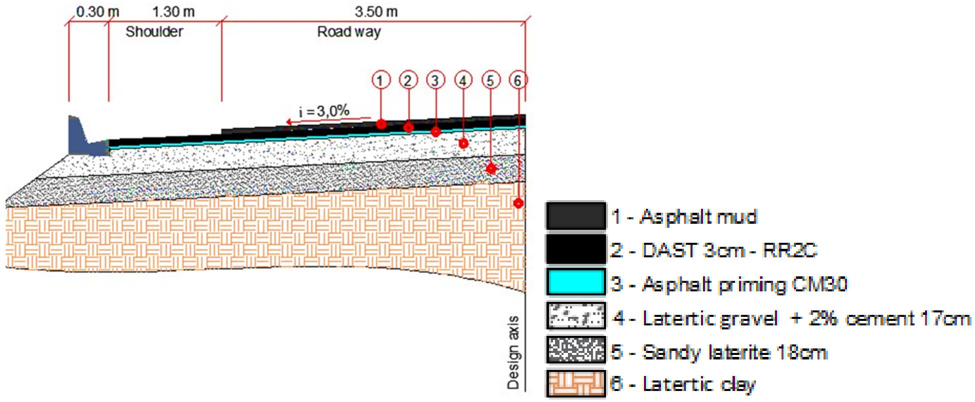
In this context, the Brazilian Asphalt Technology Network (RTA), financed by Petrobras and the National Agency of Petroleum, Natural Gas and Biofuel (ANP), has the support of universities that are conducting paving research, with the help of the National Department of Transport Infrastructure (DNIT). One of its actions is the monitoring of experimental pavement tracks, in a standardized way, with the objective of collecting systematic data on the behavior of national pavements. In 2018, the National Design Method (MeDiNa) was presented by DNIT based on data obtained by RTA. This is a Brazilian mechanistic-empirical method: mechanistic for analyzing a stress-strain relationship in the pavement, and empirical because it needs calibration for stress–strain models (Huang, 2003). The MeDiNa was developed from the SisPav software created by Franco (2007). Among the various works linked to the RTA, studies of the sections of the Fundão-RJ design (partnership between CENPES, COPPE/UFRJ and the City Hall of the UFRJ University City) of the five segments in the MG-202 road (Medrado, 2009) and the stretch on the BR-040 road (Souza Junior, 2018) were very important and fundamental to this process.

Thus, this article presents the preliminary results of one of the several monitored tracks in the state of Goiás, Brazil, implanted in the GO-230, incorporating mechanical methods in order to contribute to the database of materials, technologies and performance of an asphalt pavement constructed with Double Asphalt Surface Treatment (DAST) as the surface layer.

2. MATERIALS AND METHODS

2.1. Materials

This study followed the execution of a 340 meter long experimental track located on the GO-230 highway that connects the cities of Mimoso de Goiás and Água Fria de Goiás, Goiás State, Brazil. The design was developed by the State of Goiás Infrastructure and Transport Agency (GOINFRA), using the conventional method based on the California Bearing Ratio (CBR) for an equivalent number of standard load (N) equal to 2.58×10^6 vehicles. Figure 1 shows the built structure. Note that the choice of the Double Asphalt Surface Treatment (DAST) surface layer would not be indicated by the current Brazilian standards for the traffic level considered in the design. However, the reason for this choice was not formally clarified by the executing agency, and it is assumed that this was due to the lack of an asphalt-producing facility in the region of the work. Nine materials were studied in the laboratory, considering the materials used in the construction of the highway and the behavior of the base soil with different cement contents.



Layer	Sample	Description
DAST	BR2	Gravel type 2 – Granitic gneisses (1st layer)
	BR0	Gravel type 0 – Granitic gneisses (2nd layer)
	Emulsion	Asphalt emulsion RR-2C
Base	BA	Latertic gravel
	BA2C	Latertic gravel + 2% de cement
	BA3C	Latertic gravel + 3% de cement
	BA4C	Latertic gravel + 4% de cement
Subbase	SB	Sandy laterite
Subgrade	SL	Lateritic clay

Figure 1. Design section for GO-230 and samples description.

Table 1 – Basic properties for aggregates and soils

Properties	SL	SB	BA	BA2C	BA3C	BA4C
Grain size distribution without dispersant						
Gravel (%)	0.73	44.13	62.52	-	-	-
Sand (%)	52.94	30.60	16.52	-	-	-
Silt (%)	44.07	22.76	15.90	-	-	-
Clay (%)	2.27	2.50	5.06	-	-	-
Grain size distribution with dispersant						
Gravel (%)	0.73	44.13	62.52	-	-	-
Sand (%)	38.90	24.67	14.50	-	-	-
Silt (%)	13.12	15.97	10.22	-	-	-
Clay (%)	47.25	15.23	12.76	-	-	-
Specific gravity and Atterberg limits						
ρ (g/cm³)	2.678	2.735	2.728	-	-	-
w_L (%)	43	39	37	40	-	-
w_P (%)	28	22	19	21	-	-
PI (%)	15	17	18	19	-	-
Classification						
USCS	ML	SC	SC	-	-	-
AASHTO	A-7-6	A-2-6	A-2-6	-	-	-
FERET	Clay	Silt sand	Silt sand	-	-	-
MCT	LG'	LA'	NG'	-	-	-
MCT (c')	2.33	1.46	1.73	1.73		
MCT (e')	0.80	0.98	1.39	1.26		
MCT (d')	48.59	34.62	11.82	9.99		
MCT (Pi-%)	10.00	35.00	100.00	0.00		
Compaction and CBR						
Energy	SE	IME	ME	ME	ME	ME
w_{opt} (%)	19.80	13.00	10.47	9.73	8.70	10.43
γ_{dmax} (kN/m³)	16.45	18.43	20.50	20.50	20.55	20.60
CBR (%)	16	69	76	92	-	-
S (%)	0.08	0.05	0.05	0.00	-	-

Note: ρ = specific gravity; w_L = liquid limit; w_P = Plastic limit; PI= Plasticity index; NP = non-plastic; AASHTO = American Association of State Highway and Transportation Officials; USCS = Unified Soil Classification System; MCT = Tropical Soil Classification; LG' = Lateritic clay soil; LA' =Lateritic sand soil; NG' = Non-lateritic clay soil; c', e', d', Pi = MCT indexes; wopt = Optimum water content,; γ_{dmax} = maximum dry density; CBR = California Bearing Ratio; S = Swelling; SE= Standard energy (12 blows); IME= Intermodified energy (39 blows); ME= Modified energy (55 blows).

Table 1 presents a summary of the soil characterization and classification data and the results of the compaction, swelling and CBR tests. The subgrade, clay soil according to the Feret triangle, was classified as A-7-6 soil (fine clay soil) by AASHTO (American Association of State Highway and Transportation Officials) and identified as LG' (lateritic clay soil) by the MCT (Miniature, Compacted, Tropical) classification. Base and subbase soils, classified as silty sand by Feret, have a similar traditional classification, A-2-6, but differ in the classification for tropical soils. The subbase soil belongs to the LA' group (lateritic sandy soil) and can develop good mechanical behavior. The base soil, on the other hand, belongs to the NG' group (non-lateritic clay soil) and may have limited use for paving. The incorporation of 2% cement appeared as an alternative to improve the parameters of the base soil.

2.2. Methods

2.2.1. Laboratory studies and construction technical control

The test for determining the Resilient Modulus (MR) was performed on soil samples compacted in a universal servo-hydraulic testing machine (UTM) with capacity of 30 kN following the instructions of ME 134 (DNIT, 2018a) for soils without the addition of cement, and ME 181 (DNIT, 2018b) for cemented soils. The specimens (CP) with CPIIF-40 cement were cured in a humid chamber with relative humidity greater than 95% and a controlled temperature (25ºC) for 7 and 28 days. To predict the behavior, data regressions were performed to determine the parameters of the models presented in Table 2 in order to identify the one that best represents the behavior of each material.

For reasons of optimization, the amount of material available and the fact that the dynamic triaxial test is not a destructive method, the CPs were used in an Unconfined Compression (UC) test, performed according to NBR 12770 (ABNT, 1992) and NBR 12025 (ABNT, 2012). It is noteworthy that the specimens had their moisture measured at the moment of compaction, before the MR tests were performed, and after the rupture in the RCS test, making it possible to observe that the average variation in absolute value of the water content before and after the rupture was 0.6%, which can be considered insufficient to have a significant impact on the values of the obtained parameters.

The DAST layer was designed by the empirical method of the "red bottom tray" presented in ES-P 11 (DERT, 2000) and indicated by Silva (2018), as well as by the theoretical methods of Hanson, California, Linckenheyl and Tagle and Podestá, as cited by Pinto (2002). These data were compared with those of the DAST performed on the monitored track and required by ES 147 (DNIT, 2012). The aggregates used were also evaluated using the theoretical parameter performance-based uniformity coefficient (PUC). The coefficient is an indication of the quality of the aggregate to be used in surface treatments using granulometry. It evaluates individually, in each layer, the potential for aggregate loss and bleeding (Silva et al., 2018).

In the field, tests were carried out to determine the apparent specific mass *in situ* according to ME 092 (DNER, 1994a) and moisture using the Speedy method according to ME 052 (DNER, 1994b). Regarding the DAST layer, the execution control was executed according to ES 147 (DNIT, 2012) in order to determine the temperature of the asphalt emulsion while inside the distribution truck, immediately before application. The application rates of both the asphalt emulsion and the aggregates of the first and second layers along the track were also determined by placing and weighing trays of known mass and area.

Table 2 – Resilient modulus models from the literature

Models	Equations
Compound	$MR=K_1\sigma_3^{K_2}\sigma_d^{K_3}$
Universal AASTHO	$MR=K_1P_a\left(\frac{\theta}{P_a}\right)^{K_2}\left(\frac{\tau_{oc}}{P_a}+1\right)^{K_3}$
Universal Uzan-Witczak	$MR=K_1\theta^{K_2}\tau_{oc}^{K_3}$
Confining stress	$MR=K_1\sigma_3^{K_2}$
Deviator stress	$MR=K_1\sigma_d^{K_2}$
Semi-log	$MR=10^{(K_1\text{-}K_2\sigma_d)}$
Hyperbolic	$MR=\frac{K_1+K_2\sigma_d}{\sigma_d}$

Note: MR = Resilient modulus; σ_3 = confining stress; σ_d = deviator stress (σ_1-σ_3); θ = bulk stress ($\theta = \sigma_1 + \sigma_2 + \sigma_3$); τ_{oc} = octahedral shear stress, (τ_{oc} = √2/3. σ_d); Pa = atmospheric pressure; K_1, K_2 e K_3 = tests parameters

2.2.2. Monitoring

The monitoring evaluated the initial conditions of the pavement through the monitoring and analysis of degradations related to traffic. For that, a variety of field tests were done, as shown in Table 3.

Table 3 – Tests for pavement monitoring.: Field

Tests	Standard	7 days	3 months	6 months
Deflection	ME-133 (DNIT, 2010)	X	X	X
British pendulum	E 303 (ASTM, 2018)	X	X	X
Sand patch	E 965 (ASTM, 2015)	X	X	X
Visual inspection	REDE (2010)	X	X	X
Permanent deformation (ATR)	PRO 007 (DNIT, 2003)	X	X	X
Traffic	MANUAL (DNIT, 2006)		X	

Deflections were determined with the Benkelman beam test with measures every 25 cm until completing 2 m and a final measurement 10 m from the standard truck to the starting point. The visual survey of defects was carried out according to the RTA procedure with a specific methodology based on the procedures TER-005 (DNIT, 2003) and SHRP (1993). Finally, the traffic survey was carried out using a manual (DNIT, 2006) counting methodology without weighing the vehicles. The counting was done on seven consecutive days (March 13–19 2019) from 6:00 am to 7:00 pm, thus adopting the precision level C (when there is a 90% probability that the value of the error is between 10% and 25%).

2.2.3. Structural analysis

The MeDiNa software does not allow the use of DAST when the traffic is greater than N = 9.99 × 10^5. Therefore, for this analysis, the measured traffic was used. It is noteworthy that the permanent deformation test was not performed on the studied soils, so it is necessary to adopt already existing models from the software's database for each one of them.

With the Elastic Multiple Lcomayer Analysis (AEMC) software, the stresses and deformations developed along the layers were calculated considering the load applied by the standard road axis of 8.2 tf. In addition to the structure adopted in the project, the analysis of two more structures was also carried out, changing the cement content used in the base (3% and 4%). Considering the depth (z), points were determined on the surface, center and interface of the layers, and on the subgrade at 25 and 65 cm deep after the previous layer. According to the guidance for the AEMC software, to determine the points on the interface, pairs of points were defined at 0.001 cm below and above the interface. As for the distance perpendicular to the bearing (x), the determination of the points is a function of the type of road axis, being in the center of loading (x = 0), in the middle between the center and the inner rim of the wheel, in the center and on the rim of the outer wheel, and five points every 20 cm from the outer edge of the wheel.

2.2.4. Back analysis

In order to obtain the MR values of the pavement layers in the field after its construction, a back analysis of the Benkelman beam tests was executed using two software packages: BackMeDiNa, based on SisPav (Franco, 2007) and BAKFAA, the Federal Aviation Administration (FAA) software.

BackMeDiNa and BAKFAA were developed to work with data from Falling Weight Deflectometer (FWD) trials. Thus, for the calculation of MR, adaptations were necessary in the load value of the shaft and its application radius, as indicated by Theisen et al. (2009). The application load of 20.1 kN, a quarter of 8.2 tf, was defined, applied in an area of 0.035 m^2, due to the pressure of 560 kPa, corresponding to a circular plate with a radius of 10.69 cm. The stopping criterion for the calculation process was based on the Root Mean Square (RMS) error values, noting that BackMeDiNa considers values between 10 μm and 5 μm as a reasonable correlation and less than 5 μm as a good correlation.

3. RESULTS AND DISCUSSION

3.1. Laboratory tests

Altogether 33 CPs were tested with an average of a 99% degree of compaction and water content variation, equal to 0.7% around the optimum. The first CP of the subgrade broke during the stress sequence of the conditioning phase. Therefore, the norm suggestion was adopted and the conditioning phase and the number of voltage pairs for the subgrade were reduced. In the analyses, the mean values of MR and UC were considered.

The subgrade samples compacted in standard Proctor energy showed an average MR of 190 MPa. The subbase and base samples compacted in the intermodified (5 layers, 39 drops per layer and 4.5 kg impact hammer weight) and modified energy, respectively, presented average modules of 329 and 348 MPa, values commonly found for subbase and base materials (Bernucci et al., 2006). For chemically stabilized soil, it is noteworthy that the addition of 2% cement did not generate a significant change in the first 7 days, but at 28 days of curing it caused an increase of 85.8% (647 MPa) in the average value of the MR. The incorporation of 3% cement resulted, at 28 days, in an average MR value equal to 1275 MPa. For the soil with 4% cement after 28 days of curing, the average RM reached 4299 MPa. It appears that, even for low levels of cement, higher RM values could be expected. However, it should be noted that the stabilized soil is saprolitic; if the soil used was lateritic, perhaps better mechanical results could have been obtained. Rocha and Rezende (2017) obtained MR values in the order of 4000 MPa already with 7 days of curing for a mixture of 4% cement and lateritic gravel.

Table 4 presents the models that obtained the best behavior for each sample. Good results stand out for the model proposed by AASTHO. Bastos (2013) and Santos et al. (2019) studied similar soils and noticed the same behavior, highlighting that, despite the Brazilian tendency to use the composite model, further research is necessary. It was found that, for the samples with cement addition, the results obtained in the tests with 7 days of curing did not generate good predictions for any of the models used. This is because the chemical reactions are still very active in the first days, meaning that the mixture of the saprolitic soil with cement is still not stable.

For UC tests performed with the BA4C sample, the CPs were tested after 31 days due to the availability of the press. The results of UC at 28 days are presented together with those of MR in Figure 2a. The studied mixtures are treated, in principle, as soil improved with cement, since the incorporated contents are below 5% and no sample reached the resistance of 2.1 MPa, the minimum required by the ES 143 standard (DNIT, 2010).

Table 4 – Samples' resilient modulus behavior

Sample	Best model	R^2	Equation
SL	Universal AASTHO	0.74	$MR=2962.387P_a\left(\frac{\theta}{P_a}\right)^{0.15919}\left(\frac{\tau_{oc}}{P_a}+1\right)^{0.18014}$
SB	Compound	0.42	$MR=347.31902\sigma_3^{0.19867}\sigma_d^{-0.23839}$
BA	Universal Uzan-Witczak	0.84	$MR=359.01772\theta^{0.63465}\tau_{oc}^{-0.14182}$
BA2C-28D	Universal AASTHO	0.93	$MR=2255.41703P_a\left(\frac{\theta}{P_a}\right)^{-1.88363}\left(\frac{\tau_{oc}}{P_a}+1\right)^{2.39019}$
BA3C-28D	Universal AASTHO	0.97	$MR=28090.5077P_a\left(\frac{\theta}{P_a}\right)^{0.06645}\left(\frac{\tau_{oc}}{P_a}+1\right)^{-0.74728}$
BA4C-28D	Universal AASTHO	0.93	$MR=139156.445P_a\left(\frac{\theta}{P_a}\right)^{0.19956}\left(\frac{\tau_{oc}}{P_a}+1\right)^{-1.22881}$

Note: 28D= After 28 days curing; R^2 = coefficient of determination.

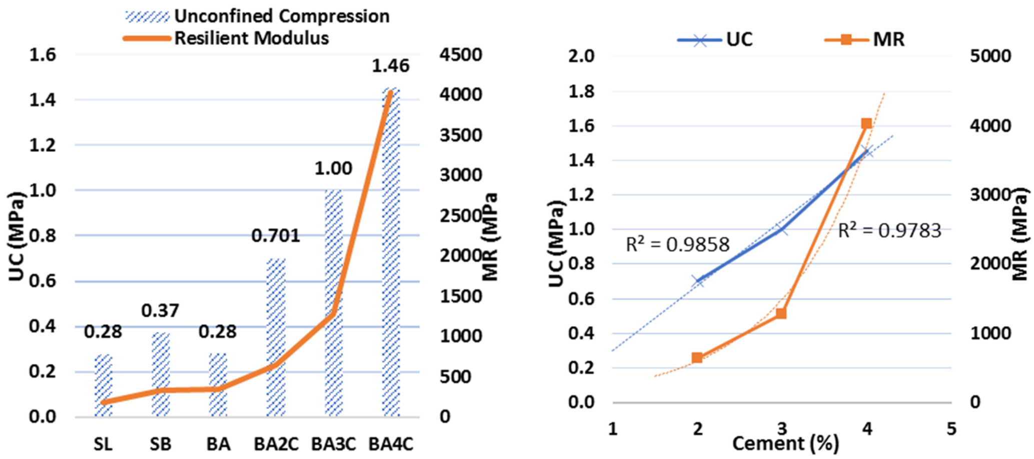


Figure 2. MR and UC results after 28 days of curing: (a) for each soil and cement mixture sample; (b) for different cement content

Figure 2b shows the growth of MR and UC generated by the addition of cement, where it is observed that the increase in MR is more significant than that of UC for the content of 4% cement. This increase in strength is explained by the pozzolanic reactions that develop over time in the chemical stabilization of soils, and the dynamic triaxial test seems to be more sensitive in obtaining the impact of altering the cement content. Thus, the addition of 4% cement caused a greater increase in stiffness, which may indicate that the soil is leaving the condition of being improved with cement and reaching that of soil-cement. Rocha and Rezende (2017) demonstrated, through microscopic analysis, that there are structural differences depending on the content of cement incorporated into the soil and this will have an impact on the structural behavior of the mixtures: while for 2% of cement, no changes were observed in the microstructure, with 4% hydration products were observed, which crystallized in the form of needles, generated a reticulated mesh in the pore voids, and improved the mechanical behavior.

3.2. Construction technical control

Table 5 presents the results of the sand replacement method and Speedy tests. It is observed that none of the layers reached a degree of compaction of 100%, so they were less rigid and more susceptible to deformations. All layers had water content lower than the optimum water content obtained in the laboratory (variation of 2.3%, 5.1% and 4.4% in the subgrade, subbase and base layers, respectively). The lack of a strict compaction control during construction, in terms of both water content and density, should impair the performance of the structure. It is assumed that the inspection did not notice such discrepancies due to the differences in the laboratory compaction results performed by the executing company. Generally, when the soils are compacted in the dry branch, they may present higher values of suction and strength, but they are also more prone to the development of shrinkage cracks due to moisture loss. On the other hand, there may be a sudden reduction in resistance if there is saturation.

Table 5 – Field compaction control for geotechnical layers

Layer	Stake	Position	w_{field} (%)	$w_{average}$ (%)	$w_{optimum}$ (%)	Δ (%)	$\rho_{d\ field}$ (kg/m³)	$\rho_{d\ average}$ (kg/m³)	$\rho_{d\ maximum}$ (kg/m³)	CG (%)
Subgrade	1602	LS	18.4	17.5	19.8	2.3	1493	1502	1645	91.3
	1602	RS	18.4				1501			
	1597	LS	16.6				1502			
	1597	RS	16.5				1513			
Subbase	1593	LS	6.0	7.9	13.0	5.1	1662	1584	1843	86.0
	1606	RS	7.5				1561			
	1599	C	10.1				1531			
Base with 2% of cement	1589	LS	6.0	6.1	9.7	4.4	1880	1909	2050	93.1
	1597	C	6.2				1938			

Note: LS = left side; RS = right side; C = center; w = water contend; Δ = water contend variation; ρ_d = dry density; CG = compaction grade.

Table 6 – DAST field control

Stake initial	Stake final	Layer	Position	Asphalt Emulsion Temperature °C	Asphalt Emulsion Ratio kg/m²	Aggregates Ratio kg/m²
1602	1630	1st layer	For M	70.5	1.22	20.07
1602	1630		For AF	70.5	1.26	20.20
1602	1630		C	71.0	0.94	20.26
1561	1602		For M	71.5	1.26	20.16
1561	1602		For AF	71.0	1.19	19.70
1561	1602		C	71.0	1.07	20.26
			Average	71.0	1.16	20.11
			SD	0.35	0.13	0.21
			CV (%)	0.5	11.0	1.1
1602	1630	2nd layer	For M	70.0	1.26	10.20
1602	1630		For AF	70.5	1.22	10.26
1602	1630		C	70.0	1.22	10.13
1561	1602		For M	70.0	1.22	10.46
1561	1602		For AF	70.0	1.22	10.20
1561	1602		C	70.0	1.19	10.23
			Average	70.0	1.22	10.25
			SD	0.20	0.02	0.11
			CV (%)	0.3	1.8	1.1

Note: M= Mimoso de Goiás; AF= Água Fria de Goiás; C = center, SD = standard variation, CV = coefficient of variation.

3.3. DAST design

As shown in Figure 3, it is observed that when comparing the aggregate rate proposed by each method with the DNIT acceptance ranges: (i) the execution met the standard; (ii) the red bottom tray method obtained a result very similar to the one executed, also being at the lower threshold of the DNIT standard; (iii) indirect methods varied from 15 to 20 L/m^2 in the rates of the constituent aggregate of the first layer, presenting higher rates than the one performed, with the Hanson and Linckenheyl method being above the DNIT range; (iv) in the second layer only Hanson's method showed a significant difference. The average of the aggregate rates obtained by indirect methods for the first layer was 17.29 L/m^2, a value close to the maximum allowed by the rule (17.75 L/m^2) and higher than the empirical method (14.34 L/m^2) and the one executed (14.27 L/m^2). For the second layer, the average rate by indirect methods was 5.72 L/m^2, which is less than the minimum required by DNIT (6.76 L/m^2) and lower than the rate performed (6.93 L/m^2).

Figure 4 shows the comparison between the calculated, executed and standard application rates. DNIT requires that the sum of the emulsion applied in the two layers respects the range of 2 to 3 L/m^2. It is noticed that the executed emulsion application rate of 3 L/m^2, which corresponds to the three applications, is higher than that calculated by the tray method (2.39 L/m^2), a method that also considers the application of the third layer of emulsion. It should be noted that all the theoretical methods determined considerably lower rates, with an average of 1.95 L/m^2, even with the determination of higher aggregate rates. Thus, it is concluded that the pavement was made with excess emulsion. This may explain the appearance of pathologies such as bleeding.

The granulometry of gravel type 0 and type 2 is shown in Figure 5, where the framing in the strips can be seen. The results of the analyses made by the PUC coefficient are shown in Table 9. It can be seen that the BR2 sample has a value very close to zero, indicating high uniformity and a low risk of developing defects, since there is only 8% of aggregates susceptible to excess binder and 4% to pullout. The BR0 sample, the aggregate that makes up the top layer of the DAST, has a higher PUC value, showing that it is not very uniform and with greater susceptibility to the main issues in DAST, with 24% of aggregates susceptible to excess binder and 28% to pullout. Table 7 also presents the same calculations for the maximum and minimum ranges, A and C, proposed by DNIT in order to compare the limits of acceptance of the standard with respect to excess binder and the loss of aggregates. It is interesting to note that the bands are permissive when compared to the PUC: Band C, for example, allows 36% of excess binder and 28% of aggregate loss, that is, 64% of the layer on the verge of failure, these being the two most critical defects in TSPs which, when not treated, are the source of several other problems, such as combing, pans, and cracking, among others (Lee; Kim, 2008). It is up to each organization to define its acceptance limits; the *Centre de Recherches Routières* (CRR, 1981), for example, considers an aggregate loss of 5–10% acceptable. Therefore, with the development of other research and improvement in the use of PUC, it is expected that the regulations can be updated in order to minimize the occurrence of these pathologies.

It is worth mentioning that the PUC was proposed for the analysis of simple surface treatments, but it is also interesting to study its application for the analysis of DAST, especially with regard to the second layer of the DAST.

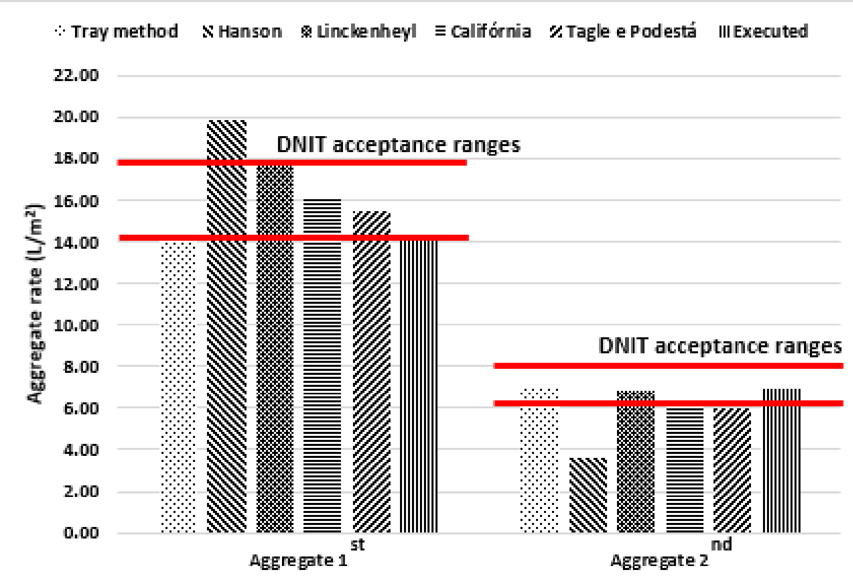


Figure 3. DAST aggregates rate evaluation

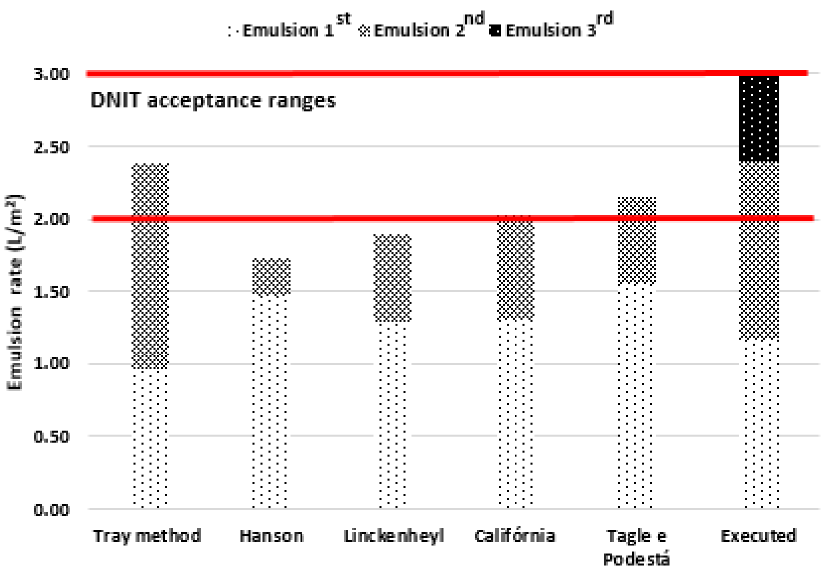


Figure 4. DAST emulsion rate evaluation

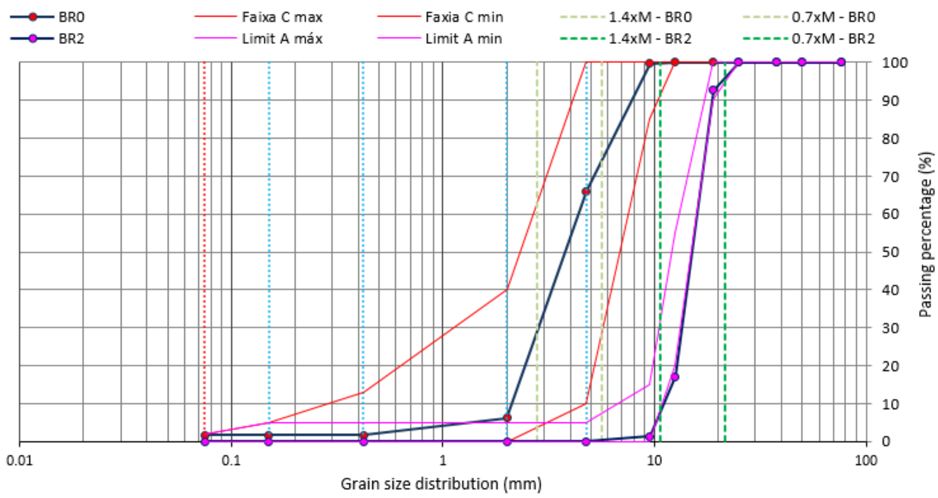


Figure 4. DAST emulsion rate evaluation

Table 7 – PUC results

	BR0	DNIT C maximum limit	DNIT C minimum limit	BR2	DNIT A maximum limit	DNIT A minimum limit
M	4	2.5	7.3	15.3	12.1	15.3
0.7M	2.8	1.8	5.1	10.7	8.5	10.7
1.4M	5.6	3.5	10.2	21.4	16.9	21.4
P_{EM}	24%	36%	18%	8%	13%	8%
P_{2EM}	72%	72%	89%	96%	86%	94%
PUC	0.33	0.50	0.20	0.08	0.15	0.09
Bleeding	24%	36%	18%	8%	13%	8%
Aggregate loss	28%	28%	11%	4%	14%	6%

Note: M = median size; P_{EM} = percentage passing at a given embedment depth of 0.7M; P_{2EM} = percentage passing at twice the embedment depth of 1.4M; PUC = performase-based uniformity coefficient.

3.4. Monitoring

As the traffic survey was carried out for 13h over 7 days, it was necessary to use an hourly expansion coefficient to represent 24h. Based on the study by Freitas (2019), the daily average traffic (DAT) measured from 151 vehicles for 13h of monitoring was transformed into 227 vehicles for 24h. By making the traffic forecast for 10 years, with 3% annual growth, the number N was equal to 6.67 × 105, a value lower than that considered in the highway project (N = 2.58 × 106). It is noteworthy that at the time of the survey, the highway was not fully completed, which may explain the low volume of heavy vehicles traffic. The average speed of the vehicles was calculated divided into three categories: light vehicles had an average speed of 92 km/h, higher than the limit for the highway, which is 80 km/h; heavy vehicles and buses with 76 km/h and 77 km/h, respectively, both below the speed limit.

Deflection measurement and structural evaluation were performed using the Benkelman beam test. The average of the deflection values, radius of curvature, coefficient of variation and characteristic deflection corrected by the seasonal factor are shown in Table 8. It appears that up to 3 months after opening to traffic, the deflections did not vary significantly. As for the 6-month monitoring, there was an increase in deflections. The values obtained can be considered acceptable, but the fact that the deflections increase rapidly may indicate early compromise of the structure, probably associated with the materials chosen for the surface and the base, as well as the construction flaws observed during the executive control.

The maximum recoverable deflection (D0) is relevant to understand the behavior of the structure, because this value must be less than an allowable value. The PRO 11 standard (DNER, 1979) determines the permissible deflections for the pavement according to the number "N". As the highway was designed for traffic with the value of N = 2.58 × 106, three N scenarios were defined for calculating the allowable deflection: medium traffic (N = 105), heavy traffic (N = 106) and very heavy traffic (N = 107). The results obtained were 134.90, 89.95 and 59.98 ×10-2 mm, respectively.

The structural evaluation is shown in Table 9 and is based on the allowable deflections, characteristic deflections and determined radius of curvature. For the initial and 3-month monitoring, all points showed good structural quality. For 6 months, the quality remained good for traffic levels of 10^5 and 10^6, while for 10^7 it decreased to regular. According to PRO 11 (DNER, 1979), when the characteristic deflection is less than the allowable level and the radius of curvature greater than or equal to 100 m, the pavement has not yet reached its fatigue phase.

Table 8 – Average of deflections

Position	Initial stake	Final stake	Length (m)	R average (m)	$D_{0\ average}$ (10^{-2} mm)	SD	CV (%)	$D_{0\ characteristic}$ (10^{-2} mm)
				7 days				
Total	1589	1606	340	434.08	36.22	11.39	31	47.61
For Água Fria	1589	1606	340	449.67	34.19	14.66	43	48.85
For Mimoso	1589	1606	340	418.50	38.26	6.59	17	44.85
				3 months				
Total	1589	1606	340	731.5	33.26	11.27	34	53.44
For Água Fria	1589	1606	340	904.49	35.32	13.42	38	58.48
For Mimoso	1589	1606	340	558.51	31.20	8.53	27	47.67
				6 months				
Total	1589	1606	340	314.43	50.21	17.76	35	81.55
For Água Fria	1589	1606	340	348.93	42.68	19.52	46	74.64
For Mimoso	1589	1606	340	279.93	57.73	12.16	21	83.87

Note: R = bend radius; D0 = maximum deflection; SD= standard deviation; CV= coefficient of variation.

Table 9 – Structural analyses

Position	N	D_{adm} (10^{-2} mm)	D_p (10^{-2} mm)	R (m)	D_p/D_{adm}	Structural quality
			7 days			
Total	10^5	134.9	47.61	434.08	0.35	GOD
	10^6	89.95	47.61	434.08	0.53	GOD
	10^7	59.98	47.61	434.08	0.79	GOD
For Água Fria	10^5	134.9	48.85	449.67	0.36	GOD
	10^6	89.95	48.85	449.67	0.54	GOD
	10^7	59.98	48.85	449.67	0.81	GOD
For Mimoso	10^5	134.9	44.85	418.5	0.33	GOD
	10^6	89.95	44.85	418.5	0.5	GOD
	10^7	59.98	44.85	418.5	0.75	GOD
			3 months			
Total	10^5	134.9	53.44	731.5	0.4	GOD
	10^6	89.95	53.44	731.5	0.59	GOD
	10^7	59.98	53.44	731.5	0.89	GOD
For Água Fria	10^5	134.9	58.48	904.49	0.43	GOD
	10^6	89.95	58.48	904.49	0.65	GOD
	10^7	59.98	58.48	904.49	0.97	GOD
For Mimoso	10^5	134.9	47.67	558.51	0.35	GOD
	10^6	89.95	47.67	558.51	0.53	GOD
	10^7	59.98	47.67	558.51	0.79	GOD
			6 moths			
Total	10^5	134.9	81.55	314.43	0.6	GOD
	10^6	89.95	81.55	314.43	0.91	GOD
	10^7	59.98	81.55	314.43	1.36	REGULAR
For Água Fria	10^5	134.9	74.64	348.93	0.55	GOD
	10^6	89.95	74.64	348.93	0.83	GOD
	10^7	59.98	74.64	348.93	1.24	REGULAR
For Mimoso	10^5	134.9	83.87	279.93	0.62	GOD
	10^6	89.95	83.87	279.93	0.93	GOD
	10^7	59.98	83.87	279.93	1.4	REGULAR

Note: N = traffic; Dadm= maximum deflection admissible; Dp= characteristic deflection; R= bend radius.

Table 10 shows the test results carried out for the macrotexture evaluation with the sand patch test and its classification according to Aps (2006). The main characteristics that influence the texture are the grain size distribution, shape and texture of the aggregates (Aps, 2006). The first determination of the macrotexture, at time 0, was performed in the DAST, while the other two determinations, at 3 and 6 months, were performed after the application of the asphalt coating. The DAST is by nature a thick macrotexture and, according to Aps (2006), it tends to easily exceed the value of 1 mm of roughness height. In the field, an average roughness height of 1.98 mm (very coarse or very open classification) was found, providing greater tire–pavement friction, and greater noise. The other two tests performed after the execution of the asphalt coating presented similar macrotextures, with values of 0.52 mm and 0.56 mm, but smaller than that determined for the DAST, which was to be expected since the asphalt coating uses small size aggregates.

Table 10 – Macro and microtexture data

Macrotexture				
7 days				
Position	Stake	Diameter average (mm)	HS (mm)	Classification
For Mimoso	1591	115	2.31	Very coarse or Very open
	1595	126.25	1.92	Very coarse or Very open
	1599	136.25	1.65	Very coarse or Very open
	1603	133.75	1.71	Very coarse or Very open
For Água Fria	1593	105	2.77	Very coarse or Very open
	1597	135	1.68	Very coarse or Very open
	1601	126.25	1.92	Very coarse or Very open
	1605	127.5	1.88	Very coarse or Very open
Average		125.63	1.98	Very coarse or Very open
SD		10.75	0.38	
CV (%)		8.6	19.4	
3 months				
For Mimoso	1591	245.42	0.51	Fairly thin
	1595	260.83	0.45	Fairly thin
	1599	258.33	0.46	Fairly thin
	1603	239.58	0.53	Fairly thin
For Água Fria	1593	233.75	0.56	Fairly thin
	1597	236.25	0.55	Fairly thin
	1601	242.08	0.52	Fairly thin
	1605	229.17	0.58	Fairly thin
Average		243.18	0.52	Fairly thin
SD		11.3	0.05	
CV (%)		4.6	9	
6 months				
For Mimoso	1591	253.33	0.48	Fairly thin
	1595	253.33	0.48	Fairly thin
	1599	267.5	0.43	Fairly thin
	1603	248.33	0.5	Fairly thin
For Água Fria	1593	211.25	0.68	Average
	1597	229.17	0.58	Fairly thin
	1601	213.33	0.67	Average
	1605	213.33	0.67	Average
Average		236.2	0.56	Fairly thin
SD		22.14	0.1	
CV (%)		9.4	18.7	

For Água Fria	**1593**	**71.40**	**Rough**	**84.00**	**Very rough**	**73.80**	**Rough**
	1597	65.60	Rough	88.20	Very rough	64.80	Rough
	1601	63.20	Rough	87.40	Very rough	69.00	Rough
	1605	74.80	Rough	70.00	Rough	75.40	Very rough
For Mimoso	1591	55.00	Rough	83.00	Rough	73.60	Rough
	1595	64.60	Rough	77.20	Very rough	81.40	Very rough
	1599	74.60	Rough	79.40	Very rough	57.80	Rough
	1603	77.80	Very rough	83.20	Very rough	74.00	Rough
Average		68.38	Rough	81.55	Very rough	71.23	Rough
SD		7.61		5.93		7.24	
CV (%)		11.1		7.3		10.2	

Note: HS = average high; SD= standard deviation; CV= coefficient of variation.

Table 10 also presents the tests executed for the microtexture with the British Pendulum test and its classification according to Aps (2006). The initial average friction resistance values were 68.38 BPN; the tests done 3 and 6 months after the execution of the asphalt mud showed values of 81.55 and 71.23 BPN, respectively. It is noticed that the application of a new layer increased the friction resistance, a value that decreased again after 6 months, most likely due to the polishing of the aggregates. The asphalt pavement restoration manual (DNIT, 2006) suggests that the BPN value should be greater than 55, and the average roughness height between 0.6 mm and 1.2 mm. According to Aps (2006), the ideal to ensure good tire–pavement adhesion is to obtain an open texture related to the macrotexture, to provide good drainage of the water, and roughness related to the microtexture, to break with the water layer. However, in the determination done after 6 months of release to traffic, a moderately fine and rough classification was obtained. Therefore, it is necessary to evaluate the friction resistance of these aggregates, since with the action of traffic and inclement weather the tendency is for the microtexture to decrease.

After evaluating the micro and macrotextures, the International Friction Index (IFI) was determined. The speed constant (Sp) is expressed in km/h and was determined with the macrotexture measurements. The adjusted friction factor for the speed of 60 km/h (F60) is dimensionless and was calculated using British Pendulum data (Table 11). A friction homogeneity is perceived throughout the track, being classified as optimal at time 0. After 3 and 6 months of release to traffic and execution of asphalt mud, the classification changed to very good. If calculated for the speed of 80 km/h, the IFI decreases, so that at 6 months the classification is reduced to good.

Table 11 – International Friction Index (IFI) results

IFI - 60 km/h									
Stake	7 days			3 months			6 months		
	Sp	F60	Classification	Sp	F60	Classification	Sp	F60	Classification
1591	250.9	0.42	Great	46.0	0.28	Very good	42.5	0.24	Very good
1593	303.3	0.54	Great	51.9	0.31	Very good	66.2	0.33	Very good
1595	206.2	0.46	Great	39.4	0.23	Very good	42.5	0.26	Very good
1597	178.9	0.45	Great	50.6	0.32	Very good	54.5	0.26	Very good
1599	175.4	0.50	Great	40.4	0.24	Very good	36.9	0.18	Good
1601	206.2	0.45	Great	47.6	0.30	Very good	64.7	0.31	Very good
1603	182.4	0.53	Great	48.9	0.30	Very good	44.7	0.25	Very good
1605	201.9	0.52	Great	54.5	0.28	Very good	64.7	0.33	Very good
Average	213.1	0.49	Great	47.4	0.28	Very good	52.1	0.27	Very good
SD	43.6	0.05		5.3	0.03		11.9	0.05	
CV (%)	20.5	9.3		11.2	11.4		22.8	20.0	

IFI - 80 km/h									
Stake	7 days			3 months			6 months		
	Sp	F60	Classification	Sp	F60	Classification	Sp	F60	Classification
1591	250.9	0.39	Great	46.0	0.20	Good	42.5	0.17	Good
1593	303.3	0.51	Great	51.9	0.23	Very good	66.2	0.26	Very good
1595	206.2	0.42	Great	39.4	0.16	Good	42.5	0.18	Good
1597	178.9	0.41	Great	50.6	0.23	Very good	54.5	0.20	Good
1599	175.4	0.46	Great	40.4	0.17	Good	36.9	0.13	Regular
1601	206.2	0.42	Great	47.6	0.22	Good	64.7	0.24	Very good
1603	182.4	0.48	Great	48.9	0.21	Good	44.7	0.18	Good
1605	201.9	0.48	Great	54.5	0.21	Good	64.7	0.26	Very good
Average	213.1	0.45	Great	47.4	0.20	Good	52.1	0.20	Good
SD	43.6	0.04		5.3	0.03		11.9	0.05	
CV (%)	20.5	9.4		11.2	13.1		22.8	23.9	

Note: Sp = speed constant; F60 = friction number; SD= standard deviation; CV= coefficient of variation.

The visual measurement performed is shown by the type of defect and the total sum of the affected area (Table 12). It is noticed that the defects have increased over time. The exception was the bleeding, which, on the Mimoso de Goiás side, at 6 months showed a smaller affected area than at 3 months, probably due to the fact that the previous bleeding print would have already "dried up", becoming imperceptible. It should be noted that, after only 6 months, isolated transverse cracks in the coating appeared, a fact that is not expected in the DAST and may be related to the retraction cracks in the base due to the low compaction humidity observed during the executive control. This fact should be evaluated better in the next monitoring. Table

12 also shows the rutting or permanent deformation (ATR) results obtained after 6 months. The values found for permanent deformation are within those expected for a pavement with less than one year of release to traffic (average deformations around 2 and 3 mm) and similar to the values found by Pérez (2016).

Table 12 – Visual inspection results and permanent deformation (ATR) 6 months after the pavement construction

Visual inspection				
Position	Type of defects	7 days	3 months	6 months
		Area (m²)	Area (m²)	Area (m²)
For Água Fria	Polished aggregate	-	0.07	1.16
	Surface layer degradation	0,01	1,10	6,99
	Exudation	0.04	1.32	1.64
	Corrugation	0.02	0.02	0.28
	Foot hole	0.02	0.03	0.12
	Pavement patch	-	-	0.01
	Crack isolated	-	-	-
For Mimoso	Polished aggregate	-	0.10	0.25
	Surface layer degradation	-	0.69	0.91
	Exudation	0.06	4.26	0.11
	Corrugation	-	0.06	0.04
	Foot hole	-	0.06	0.57
	Pavement patch	-	-	-
	Crack isolated	-	-	0.04

ATR						
Stake	For Água Fria city			For Mimoso de Goiás city		
	Left side (cm)	Right side (cm)	Average (cm)	Left side (cm)	Right side (cm)	Average (cm)
1589	0.00	0.00	0.00	0.20	0.20	0.20
1590	0.10	0.20	0.15	0.10	0.30	0.20
1591	0.20	0.20	0.20	0.00	0.30	0.15
1592	0.40	0.30	0.35	0.00	0.20	0.10
1593	0.20	0.30	0.25	0.00	0.30	0.15
1594	0.20	0.30	0.25	0.30	0.20	0.25
1595	0.20	0.40	0.30	0.20	0.40	0.30
1596	0.10	0.30	0.20	0.20	0.30	0.25
1597	0.20	0.40	0.30	0.20	0.30	0.25
1598	0.20	0.30	0.25	0.20	0.30	0.25
1599	0.40	0.40	0.40	0.20	0.30	0.25
1600	0.30	0.30	0.30	0.20	0.30	0.25
1601	0.50	0.60	0.55	0.20	0.30	0.25
1602	0.30	0.20	0.25	0.10	0.30	0.20
1603	0.30	0.20	0.25	0.00	0.50	0.25
1604	0.50	0.80	0.65	0.30	0.50	0.40
1605	0.40	0.80	0.60	0.10	0.10	0.10
1606	0.50	0.40	0.45	0.20	0.30	0.25
Average	0.28	0.36	0.32	0.15	0.30	0.23
SD	0.15	0.20	0.16	0.10	0.10	0.07
CV (%)	53	57	51	66	32	32

Note: SD= standard deviation; CV= coefficient of variation

However, in some places, RTA values greater than 8 mm were observed. For permanent deformation, the value of 7 mm is considered alarming and 10 mm as the maximum allowed during the life of a pavement considered as part of the Main Arterial System, according to the MeDiNa method. The Global Gravity Index (GGI) was also calcx'ulated, according to the standard PRO 006 (DNIT, 2003), which verifies that this parameter is indicated for the evaluation of other types of coating. In this study, the GGI was used for comparison only during monitoring. For the period of 6 months, the pavement of the side towards Água Fria was classified as Bad (GGI = 81) and Mimoso as Regular (GGI = 55).

3.5. Structural analysis

The MeDiNa program does not consider the damage due to fatigue when it comes to DAST, and the performance analysis is related to the ATR. At the end of the 10 years of the project, a total ATR forecast equal to 6.90 mm was obtained. Comparing what was predicted by the program for 6 months of use (5.79 mm) and the value measured by monitoring (average of 1.50 mm to 3.60 mm), it is clear that the forecast is higher. This difference can be explained by the fact that, in this work, still no real permanent deformation data were obtained for the soils used in the pavement. Anyway, it is observed that in the 6 months of monitoring, on average between 30% and 46% of the total predicted ATR had already occurred.

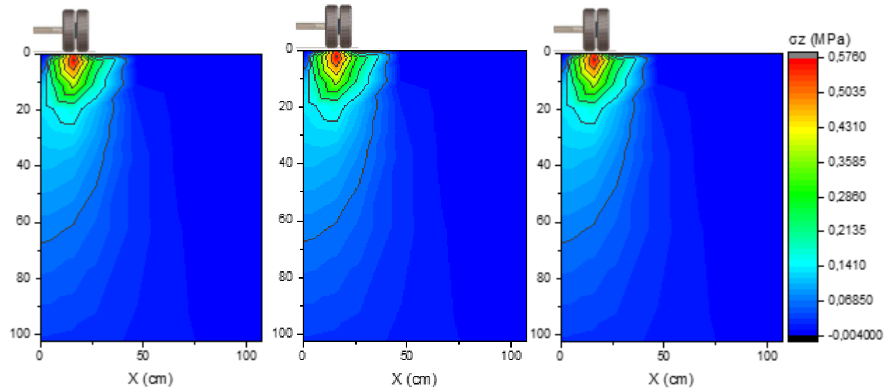
Figure 6 presents the profile of the section studied for the analysis of stresses and strains. Positive values indicate that the point is in tension, while negative values indicate compression. Regarding the stresses along the depth (σz), they dissipate in a similar way even with the change of the base material. The displacements generated in the depth (Uz) were high in the subbase and in the subgrade (6.4 mm and 14.4 mm, respectively), while in the base it was 1.4 mm. Thus, the subgrade was responsible for 65.1% of the displacement, which agrees with its less resilient performance. It is interesting to note that, with the increase in the cement content at the base, and consequently the stiffness, the total deformations decreased.

Figure 6c shows the profile of specific displacements in depth (Ez). The subgrade is the layer with the worst performance. Noteworthy is the reduction in deformations due to the change in the base material. By analyzing the stresses along the distance perpendicular to the bearing (σx) (Figure 6b), the highest stresses occur at the interface between the base and the subbase and these stresses increase as the base stiffness also increases. The increase of stresses in the lower fiber of the base can result in the appearance of cracks, which may be reflected on the surface.

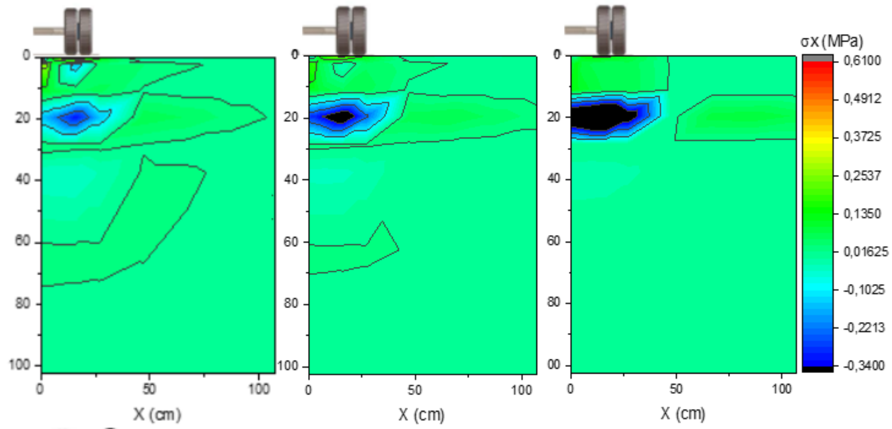
However, the increase in stresses at (x) does not result in greater deformation on that axis. On the contrary, it can be seen that the deformations in (x) as well as in (z) decreased with increasing layer stiffness (Figure 6d). Ranadive and Tapase (2016) found that increasing the modulus of the base layer reduces ATR damage, a fact also observed by Das and Pandey (1999), Hadi and Bodhinayake (2003) and Behiry (2012). This analysis showed that the built structure presents high displacements in the lower layers, and this finding, combined with the other executive problems already mentioned in the text, will contribute to the appearance of pathologies.

3.6. Back analysis

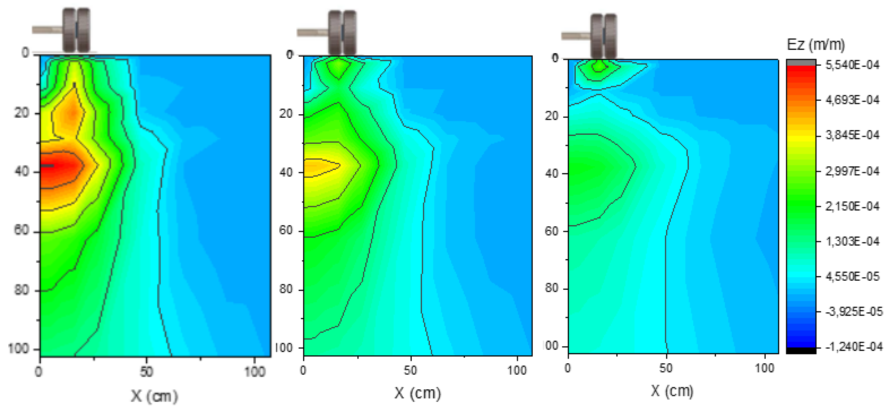
Figure 7 shows the average deflection basins for each range in the three monitoring times compared to the basins calculated by the back analysis software. It is observed that the values of the back analyzed modules (1928, 586 and 271 MPa for base, subbase and subgrade, respectively) were higher than those found in the laboratory. One of the hypotheses for this difference is that the humidity below the optimum used in the field may have resulted in increased soil suction, which, according to Santos (2019) and Freitas, Rezende and Gitirana Jr. (2020), would generate an increase in RM. The error values obtained by BackMeDiNa were lower than those of BAKFAA, and BackMeDiNa was able to better adjust the measured curve regardless of its shape. With only 6 months of monitoring and with the high dispersion presented by the results obtained with the back analysis of data from the Benkelman beam, it was not possible to determine an increasing or decreasing behavior of the MR values for that short period of time.



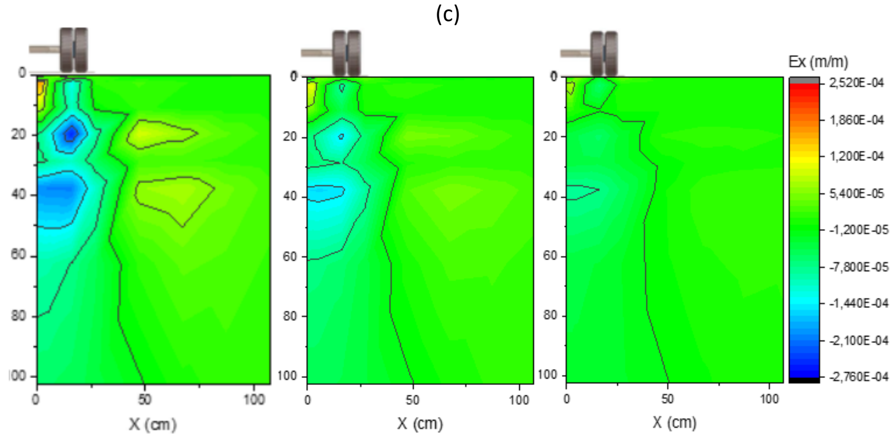
(a)



(b)



(c)



(d)

Figure 6. Structural analyses varying cement content (2%, 3% and 4%): (a) Stress on axis z (MPa); (b) Stress on axis x (MPa); (c) Specific deformation on Z (m/m); (d) Specific deformation on X (m/m)

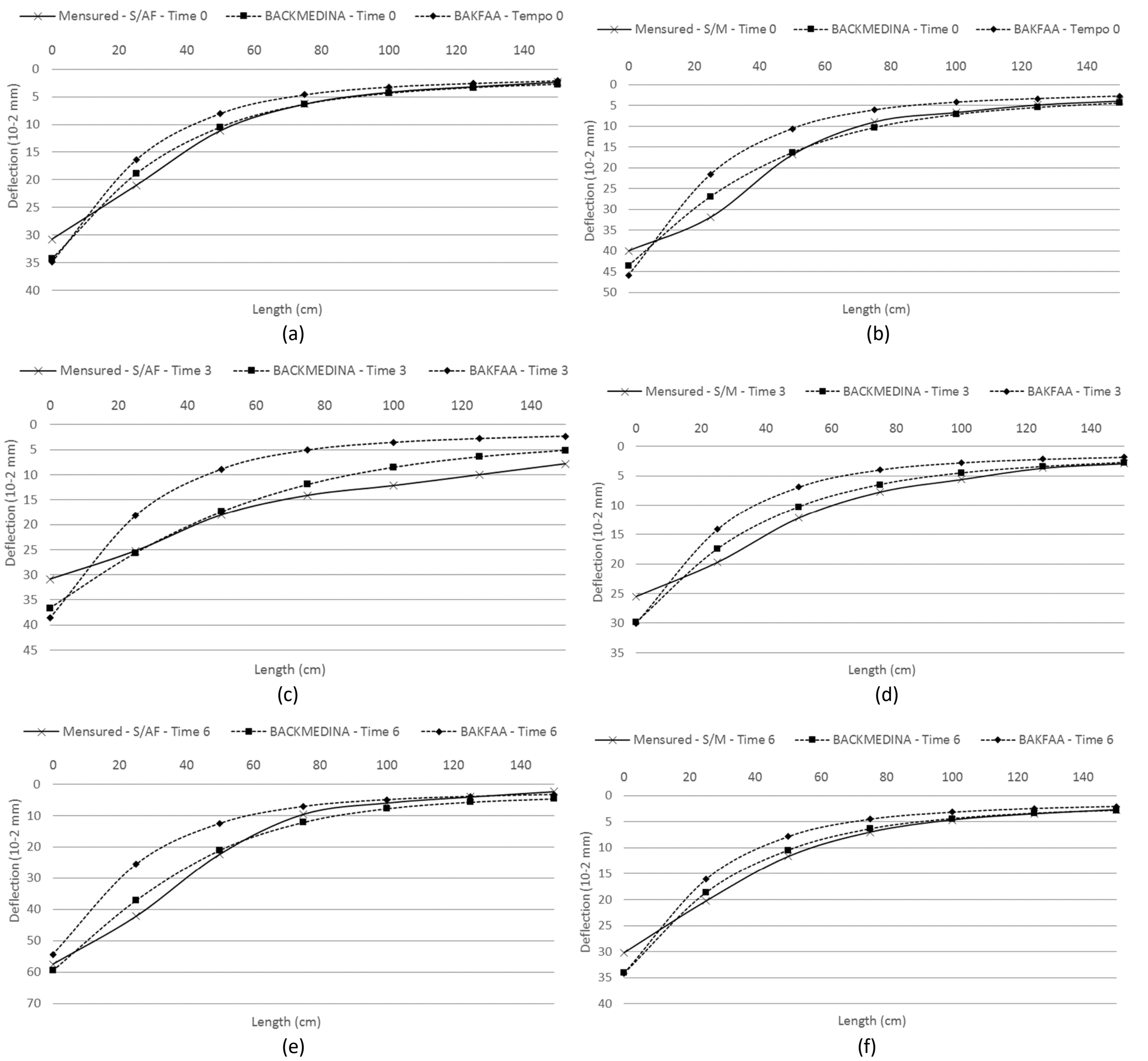


Figura 7. Field deflection basins versus calculated deflection basins: (a) 7 days – for Água Fria; (b) 7 days - for Mimoso de Goiás; (c) 3 months - for Água Fria; (d) 3 months- for Mimoso de Goiás; (e) 6 months - for Água Fria; (f) 6 months - for Mimoso de Goiás.

6. CONCLUSIONS

After a laboratory study and initial field monitoring of the track implanted at GO-230, it was concluded that:

- the international classification systems are flawed when dealing with tropical soils, since the subbase and base samples have the same AASHTO, SUCS and Feret classification, but differ when compared to the MCT methodology;
- the resistance parameters obtained in the laboratory show that the materials used in the base can present good behavior since they are compacted in the ideal water content and density, a fact that was not fulfilled in the pavement construction, as demonstrated in the compaction control;
- the incorporation of 2% cement into the base soil improved its properties. The addition of higher levels of cement raised the resistance parameters to another level and made

the base more rigid. The point is that, as the base improved with 2% cement was compacted in a water content below the optimum, this procedure may have generated the formation of shrinkage cracks in the layer with reflection for the surface;

- when the design of the DAST was evaluated, it was noticed that the acceptance ranges of the DNIT for the application of aggregates and emulsion are broad and generic. The aggregate that makes up the top layer of the DAST showed a high PUC value, with more than 50% of the layer prone to pullout or exudation, corroborating the issues found. It is recommended to use the PUC coefficient to aid in the choice of aggregates to be used for DAST, given its easy implementation and its importance for the prevention of pathologies;
- the macrotexture measured in the TSD showed a significant change due to the application of the asphalt mud, while the microtexture did not, since the value obtained after 6 months of release to traffic was similar to that found in the initial monitoring;
- the structural analysis showed high deformations in the subbase and subgrade, probably due to the lower resilience performance of the selected soils. The analysis of the structure for higher cement contents showed that the increase in stiffness generated an increase in horizontal stresses at the base/subbase interface, but decreased the deformations generated and the consequent displacements that the layers presented;
- in the back analysis process, higher modulus values were obtained than those determined in the laboratory, probably due to the lower water content values used in the field;
- the main question to be assessed is whether a pavement structure composed of subgrade, subbase and base compacted in water content and with density lower than the ideal values, for an improved base with cement that will still present an increase in its stiffness over time covered by a thin layer of DAST, will perform well.

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