Flexible pavement with mining waste proposal - execution and analysis of an experimental section

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Abstract

The extraction of iron ore has a fundamental role in the Brazilian economy. However, such activity generates considerable volumes of waste whose disposal, even if regulated and licensed, has a significant environmental impact. The worldwide concern for sustainable practices along with the urgency for measures to mitigate environmental damage, justifies research on its use in other activities. Therefore, the present study aims to analyze the test results of laboratory versus field of mixtures composed of mining waste. The field studies were made on the execution of an experimental section on the BR 040 highway, in Minas Gerais - Brazil. According to the validation of the laboratory results, the mixtures composed of 35% Tailings + 15% Waste Rock + 50% Canga of Ore and 35% Tailings + 20% Gravel 1 + 45% Gravel 0 were used in the base and sub-base layers of the experimental section, respectively. The field technological control showed that the mixtures had an anomalous behavior from the one observed in the laboratory, mainly regarding strength and deformability.

Keywords: sustainable, mining, tailings, test, and section.

1. Introduction

The Center-South region of Minas Gerais, Brazil, has one of the largest reserves of iron ore, mainly centralized in the Quadrilátero Ferrífero area (Pereira, 2005). Due to the growing demand for materials and inputs made from minerals, the mineral reserves are being overexploited, generating mineral concentrates and mining waste called tailings and waste rock.

Along with the environmental impacts caused by the extraction of minerals, there is the deforestation of areas for the disposal of mining waste and the possible contamination of watercourses when in contact with these materials. One alternative solution is to establish new uses for mining waste to enhance environmental preservation and reduce transport and storage costs (Galhardo, 2015).

The authors Fernandes *et. al*. (2004), Campanha (2011), Oliveira (2013), Pinto (2013), Friber (2015), and Galhardo (2015) carried out several laboratory tests to verify the applicability of using tailings in the granular layers of flexible pavements. However, all these studies focused only on laboratory data. Hence, to truly verify the tailings' behavior against dynamic loads, experimental sections built with mining waste are necessary.

Therefore, this article evaluates the performance of mining waste mixtures used in an experimental section, specifically from km 574+400/MG to 574+300/MG northbound on the slow traffic lane of highway BR 040.

2. Materials and methods

The materials and mining waste applied in the experimental section are shown in the sequence. Furthermore, the reference techniques and regulations used in the tests and execution of the experimental section are addressed.

2.1 Materials

• Tailings and waste rock: The mining waste under study comes from a mining company in Congonhas in Minas Gerais state. The tailings and waste rock were placed in piles, so the criterion used for their collection was sampling.

• Soil – Canga of Ore: This is the main material in the experimental section area. Thus, this soil was used to compose the sub-base material because of ATD (Average Transport Distance) and available quantity.

• Stone aggregates: Stone aggregates, gravel 0 and 1, were used to compose the base layer. They were collected in a quarry in the city of Ouro Preto, in Minas Gerais state.

Initially, the tailings and waste rock samples were tested to assess their characteristics. After checking the results, it was concluded that a composition of these mining wastes with other materials with better characteristics for paving would be necessary. Then, the compositions of tailings, waste rock, and canga of ore, shown in Table 1, were tested for the sub-base layer. However, for the base layer, stone aggregates were added to the tailings to improve the granulometric characteristics and support capacity. Although several compositions were tested, this study only shows the mixtures with the best geotechnical properties.

Table 1 – Studied materials.

DESCRIPTION	IDENTIFICATION	INTENDED DESTINATION
Tailing	T	Base/Sub-base
Waste rock	WR	Base/Sub-base
35%T + 15% WR + 50% Canga of ore	M1	Sub-base
35%T+ 20%Gravel 1 + 45%Gravel 0	M2	Base

3. Methods

3.1 Execution of the experimental section

The experimental section (Figure 1) was on BR 040, specifically from km 574+400/MG to 574+300/MG northbound traffic lane, between Congonhas/MG and Itabirito/MG. All construction phases followed the Brazilian specifications. Table 2 describes the main information regarding the execution of the experimental section. It is important to state that the number N $1.18x10^7$, was given by the company responsible for the highway.

Figure 1 - Experimental section.

Table 2 – Execution of the experimental section.

Activity	Thickness	Description
Size of the pavement cut	62 cm	100 m long x 3.5 m wide x 0.62 m deep
Subgrade regularization	-	-
Sub-base execution	20 cm	Mixture composed of 35% Tailings + 15% Waste Rock + 50% Ore Canga carried out in a Soil Plant
Base execution	30 cm	Mixture composed of 35% Tailings + 20% Gravel 1 + 45% Gravel 0 carried out in a Soil Plant
Pavement priming	-	Emulsion used: CM-30
Tack coat	-	Carried out after priming and between the two coating layers. Input: RR1C
Surface layer	12 cm	CBUQ used: CAP 30/45 Range B
Geogrid	-	Model: Haltelit C40/17 - Huesker

Note that the geogrid was installed along the entire experimental section and between the two layers of the asphalt coating (6 cm +6 cm), as shown in Figure 2.

Figure 2 – Cross-section using the geogrid. Source: Huesker, 2022.

Hatelitc geogrid, made from high modulus polyester filaments combined with an ultra-lightweight non-woven fabric, is used for reinforcements. Among its main advantages is the reflection of the cracks control, as specified by the Manufacturer Huesker.

3.2 Technological control

Technological control of all layers of the experimental section was carried out to ensure the characteristics obtained in the laboratory, including asphalt coating. However, the coating results will not be addressed as it is not the focus of this study. Table 3 shows the tests carried out and the standards used.

Table 3 - Field Experiments.

Activity	Norm	Observation
Determination of in situ moisture	DNER-ME 052/1994	Method: Speedy Equipment
Determination of in situ specific mass	DNER-ME 092/1994	Method: Sand Bottle. Obtain the Degree of Compaction of each layer.
DCP (CBR in situ)	ASTM 6951M-18.	Use of data correlations from the DCP (Dynamic Penetration Cone) equipment.
Recoverable Deflection	DNER-ME-24/1994	Method: Benkelman Beam and LWD (Light Weight Deflectometer)

4. Results and discussions

4.1 Physical and mechanical description

To ensure the normative requirements in the base and sub-base layers, several laboratory mixtures were made until mixtures suitable for paving were found. Therefore, Table 4 shows the physical and mechanical test results from the tailings, residues, and mixtures used in the experimental section.

Table 4 – Physical and mechanical test results.

				MATERIALS				
				Tailings	Waste Rock	M4	M7	Subgrade
PHYSICAL	GRANULOMETRY	2"	% Passing	100	100	100	100	-
		1"		100	100	94.06	100	-
		3/8"		100	98	74.63	68	-
		N°4		100	95	59.34	39	-
		N° 10		100	91	45.78	35	-
		N° 40		99	585	40.63	35	-
		N° 200		77	74	32.28	27	-
	LL		%	NL	58	NL	NL	NL
	IP		%	NP	19	NP	NP	NP
	EA		%	8,18	12.76	-	-	-
	IG		-	8	15	0	0	-
	HRB			A-4	A-7	A-2-4	A-1-B/A-2-4	
	Grains Ps		**g/cm³**	2,783	2.978	-	-	-
	MCT		-	-	LG'	-	-	-
MECHANICAL	ρs max		**g/cm³**	1.833	1.658	2.379	2.173	2.812
	w opti		%	13.9	25.20	9.55	6.7	8.8
	C.B.R		%	17.5	29.30	34	219.8	43
	EXP.		%	0.38	0.00	0.40	0.00	0.00
	MR*		**MPa**	-	-	-	312	392

LEGEND:

LL: Liquid Limit	MCT: Miniature, Compacted, Tropical
IP: Plasticity Index	W opti: Optimal moisture
EA: Sand Equivalent	CBR: California Bearing Ratio
IG: Grading Index	EXP: Expansion
HRB: Higway Research Board	MR: Resilience Module

For the M4 material, which consisted of tailings, waste rock, and canga of ore, it was found that by adding canga of ore to the mixture the percentage of fines passing through sieves number 40 and 200 are relatively lower than the mining waste alone. Furthermore, this mixture is non-plastic, with CBR, expansion, and MR suitable for the sub-base layer. Also, the IG is zero, thus meeting the Norm specifications for the sub-base layer. The material is classified as A-2-4 and forms a mixture of silty sand that has an overall behavior from excellent to good for flexible pavements. Therefore, the material has, according to the laboratory results, suitable properties for its use in the sub-base layer.

M7 material, which consisted of tailings, gravel 0, and 1, was designed for the base layer with expected traffic of N > $5X10^6$. Thus, the granulometry is between zones B and C, not meeting only the 200 sieve percentage criteria. Since this is an experimental section, the M7 mixture was used even though the granulometric requirements have not been fully met. The reason is that M7 easily meets the strength and stability parameters, has a high CBR, low expansion, is non-plastic, and the material is classified between A-2-4 and A-1-B, with its behavior from excellent to good for pavements. The MR value was very low for a mixture that is composed of 65% of stone aggregates, which are materials from which good mechanical responses are expected. However, the addition of tailings to the mixture, which is a silty-sandy, non-cohesive material with a uniform granular distribution, tends to reduce friction among aggregate particles, leading the mixture to experience greater deformations and explaining the low MR value.

4.2 *In situ* Moisture

Table 5 shows the average in situ moisture of each layer in comparison with the optimal moisture obtained in the laboratory. The *in situ* moisture is an average from 4 distinct points in the experimental section.

Table 5 - Field vs laboratory moisture results.

Layer	Average *in situ* moisture (%)	Optimal moisture (%)	Range (%)	Allowable range (%)
Subgrade	6.63	8.00	-1.37	± 2
Sub-base	9.35	9.55	-0.20	± 2

Note that, for the subgrade, there was a moisture deviation of 1.37% below the optimal moisture. According to the specifications of the DNIT-ES 137 Norm (DNIT, 2010), the allowable moisture deviation is ± 2%, validating the results obtained. For the sub-base, the range obtained was 0.20% below the optimal moisture, meeting the norm specification of an allowable range of ± 2%. The same case happens for the base, 0.08% below the optimal moisture. Therefore, the results from these two layers are adequate, since they had small ranges that met the norm specifications.

4.3 *In situ* specific mass measured by the sand bottle method

The test of in situ specific mass measured by the sand bottle method was carried out to verify the Degree of Compaction (DC) of each granular layer in the experimental section. Thus, Table 6 shows the Degree of Compaction (DC) obtained for each layer as a function of the average in situ specific mass and the laboratory specific mass. The average was obtained by executing the test at 5 different points in the experimental section.

Table 6 – DC results.

Layer	*In situ* bulk density (g/cm³)	Dry-bulk density (g/cm³)	Degree of Compaction (%)
Subgrade	2.835	2.812	101
Sub-base	2.443	2.379	103
Base	2.296	2.220	103

The DC from each layer was above 100%, hence, meeting the current normative requirements. DC values above 100% can be linked to moisture, since, on average, the moisture was slightly below the optimal compaction moisture but within the tolerable limit, which tends to increase the apparent dry specific mass of the compacted material in comparison with the laboratory.

This observation was also verified by Trindade *et al*. (2003) who stated that when the soil has a moisture content below the optimal, the application of greater compaction energy increases the apparent dry specific weight, but when the moisture is greater than the optimal, greater compaction effort causes little to no increase in the apparent dry specific weight, as it is not possible to expel the air from the voids.

4.4 *DCP (Dynamic Penetration Cone)*

The DCP test was carried out to evaluate the CBR in situ, that is, the stability of the base and sub-base layers under field conditions (Figure 3). Table 7 shows the results of the sub-base and base layers by applying the correlation shown in the ASTM 6951M-18 norm. The ranges of the CBR values obtained at each point of the highway are shown in Figure 4 and Figure 5.

Figure 3 - Execution of CBR *in situ*.

Table 7 - CBR *in situ* results.

Layer	Average CBR in situ (%)	Standard Deviation (%)	DNIT CBR norm requirement (%)
Sub-base	21.8	1.53	20
Base	37.0	2.18	80

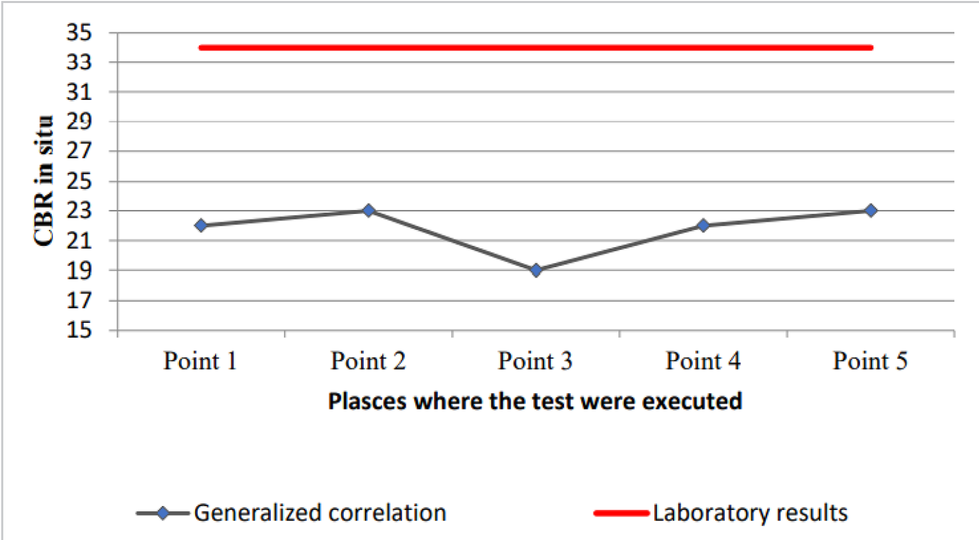

Figure 4 - Range of CBR *in situ* values - Sub-base.

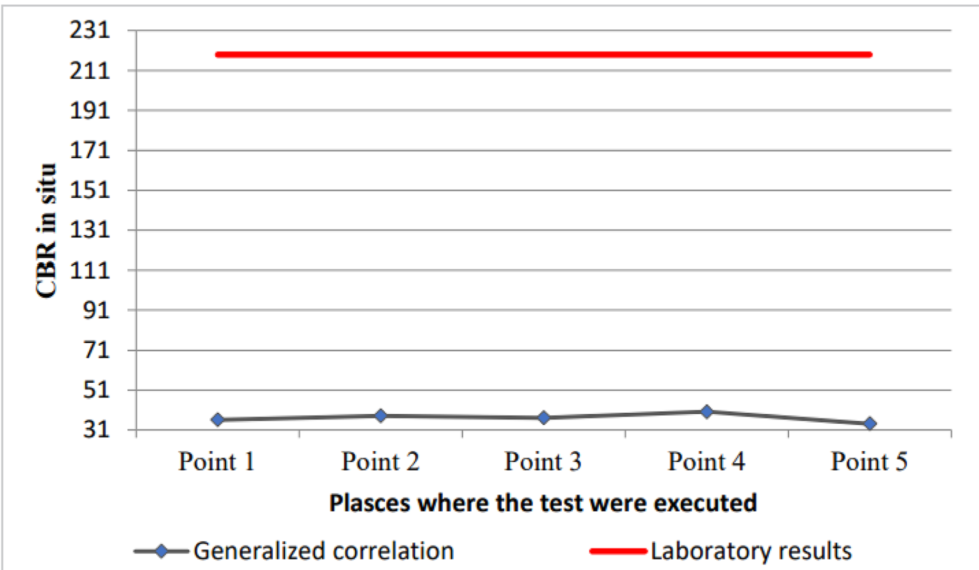

Figure 5 - Range of CBR *in situ* – Base.

The average CBR in situ of the sub-base layer was 21.8% with a standard deviation of 1.53%, indicating homogeneity of the results. Furthermore, the CBR in situ was above 20% meeting the normative requirements. However, it should be noted that the results achieved were much lower than the laboratory ones, a difference of 12.2%.

For the base layer, the average CBR in situ was 37% while the laboratory one was 219.8%, a difference of 182.8%, indicating that the M7 mixture behaved differently from the laboratory. Furthermore, this result does not meet the DNIT normative for the base layer's CBR. However, the base layer field results are homogeneous with a standard deviation of 2.18%. For both layers, the CBR in situ does not match the laboratory CBR, which means that the materials behave differently in the field, enhancing the need to determine a field-laboratory factor. It is noteworthy, however, that CBR *in situ* is obtained through a correlation, and correlations are obtained considering relatively conservative safety factors. This can also be associated with the difference in field and laboratory results.

4.5 Recoverable deflections

Benkelman Beam tests were carried out on the Inner Wheel Rail (IWR) and Outer Wheel Rail (OWR) of the experimental section, following the guidelines from the DNER-ME 024/94 and DNER-PRO 011/79 norms (Figure 6). The values of deflection and radius of curvature measured by the Benkelman Beam test are shown in Table 8.

For the subgrade layer, the largest recoverable deflection was 22x10^{-2} mm in the Outer Wheel Rail (OWR). The radius of curvature results ranged from 500 to 1000 m. According to Nunes (2015), a radius of curvature below 100 m indicates an intermediate or poor condition of the pavement. Thus, all points tested, both in the IWR and in the OWR, have a radius greater than the one mentioned, configuring a satisfactory condition of the subgrade layer.

For the sub-base layer, high deflections are found at the initial point (D0) ranging from 90x10^{-2} mm to 168x10^{-2} mm for the IWR, and from 102 x10^{-2} mm to 202x10^{-2} mm for the OWR. Regarding the radius of curvature, from three points tested on each wheel rail, only one had a radius of curvature above 100 m, configuring a poor pavement condition.

Figure 6 - Test execution - recoverable deflections.

Table 8 - Results of recoverable deflections and radius of curvature obtained in the experimental section.

Wheel rail	Layer	Survey Stake	Recoverable Deflections (0.01 mm)							Radius of Curvature (m)
			D_0	D_{20}	D_{30}	D_{45}	D_{60}	D_{90}	D_{120}	
Inner	Subgrade	574+400	4.0	0.0	0.0	0.0	0.0	0.0	0.0	500.0
		574+375	6.0	4.0	4.0	4.0	2.0	2.0	0.0	1000.0
		574+350	4.0	2.0	0.0	0.0	0.0	0.0	0.0	1000.0
		574+325	8.0	6.0	6.0	4.0	4.0	4.0	0.0	1000.0
	Sub-base	574+375	104.0	84.0	80.0	4.0	2.0	2.0	0.0	100.0
		574+350	90.0	66.0	32.0	18.0	6.0	2.0	0.0	83.3
		574+325	168.0	108.0	64.0	26.0	16.0	2.0	0.0	33.3
	Base	574+400	86.0	78.0	40.0	20.0	6.0	4.0	0.0	250.0
		574+375	100.0	98.0	64.0	26.0	8.0	4.0	0.0	1000.0
		574+350	14.0	6.0	2.0	2.0	0.0	0.0	0.0	250.0
		574+325	58.0	24.0	14.0	8.0	6.0	4.0	0.0	58.8
Outer	Subgrade	574+400	4.0	0.0	0.0	0.0	0.0	0.0	0.0	500.0
		574+375	6.0	2.0	2.0	0.0	0.0	0.0	0.0	500.0
		574+350	12.0	10.0	8.0	6.0	4.0	2.0	0.0	1000.0
		574+325	22.0	20.0	0.0	0.0	0.0	0.0	0.0	1000.0
	Sub-base	574+375	202.0	62.0	18.0	6.0	6.0	4.0	0.0	14.3
		574+350	102.0	84.0	62.0	36.0	14.0	4.0	0.0	111.1
		574+325	108.0	68.0	48.0	22.0	14.0	10.0	0.0	50.0
	Base	574+400	12.0	10.0	8.0	4.0	2.0	0.0	0.0	1000.0
		574+375	6.0	2.0	2.0	2.0	2.0	2.0	0.0	500.0
		574+350	14.0	12.0	-4.0	2.0	0.0	0.0	0.0	1000.0
		574+325	80.0	44.0	36.0	12.0	8.0	2.0	0.0	55.6

In summary, the base layer recoverable deflections are heterogeneous. The behavior of the base layer corresponds to the behavior of the sub-base and subgrade layers. Regarding the radius of curvature, 66% of the values of the sub-base are below 100, while, for the base layer 75% of the results were above the threshold recommended in literature as a quality reference. All values obtained for the subgrade are above this threshold, which can be concluded that the sub-base layer is of poor quality and impairs the behavior of the base layer.

For comparison, Nunes (2015) performed a Benkelman beam test in the construction of a sub-base layer of a highway, previously released by laboratory tests based on in situ density. It was observed that, from the five points tested, two had recoverable deflections above the permissible, ranging from 52×10^{-2} mm to 98×10^{-2} mm.

4.6 Post-construction pavement assessment

After building the experimental section, a new step of data gathering started to evaluate the pavement's behavior based on the adopted solutions. The data gathering was characterized by structurally evaluating the pavement with the application of the LWD test.

Regarding structural pavement data gathering, the average deformation data (S_{med}) and the Elasticity Modules (E_{wd}) obtained by the LWD are shown in Table 9. The values in this table are an average of 10 points surveyed along the pavement.

Table 9 - Average values of elasticity modules and deformation via LWD.

Day	E_{wd} (MPa)			S_{med} (10^{-2}mm)		
	Average (Mpa)	DP (%)	CV (%)	Average (10^{-2}mm)	DP (%)	CV (%)
04/November/2019	76.94	13.72	17.83	30.1	0.073	24.09
21/January/2020	69.97	18.16	25.95	35.3	0.239	67.70
25/January/2020	63.16	14.05	22.24	37.5	0.193	51.64
27/January/2020	76.00	21.42	28.18	31.7	0.153	48.37
31/January/2020	64.17	21.55	33.59	38.8	0.238	61.43
03/February/2020	78.89	30.35	38.47	31.3	0.104	33.22
07/February/2020	75.31	16.42	21.80	31.2	0.080	25.59
10/February/2020	55.83	25.65	45.94	51.9	0.579	111.64
14/February/2020	59.62	20.34	34.12	43.6	0.367	84.21
17/February/2020	60.11	16.51	27.46	42.1	0.370	87.92
20/February/2020	62.18	7.96	12.81	36.7	0.062	16.92
29/February/2020	59.76	15.18	25.40	40.8	0.263	64.38
03/March/2020	68.11	12.37	18.16	33.9	0.077	22.64
09/March/2020	65.85	21.64	32.86	36.9	0.139	37.61
18/May/2020	89.41	27.17	30.38	27.1	0.108	39.96
29/June/2020	102.89	20.30	19.73	22.7	0.049	21.54

The results show that the Elasticity Modules have low values. The lowest value is 55.83 MPa with a coefficient of variation of 45.94%, for this evaluation day. It was found that the surface, in the 100 meters evaluated, has a heterogeneous behavior. During the entire evaluation period, the highest elasticity module average for the experimental section was around 102.89 MPa during the dry period, which is a period without the influence of infiltration and/or elevation of the groundwater level due to rainwater. Therefore, the observed trend is that the layers constituted of tailings are strongly influenced by the variation of the internal moisture of the pavement layers.

Taking as a reference the allowable project deformation, which for the experimental section was 60×10^{-2} mm calculated according to the DNER-PRO 011/79 norm, it can be noted that the deformations are below this reference. However, within a short lifetime (less than one year) the experimental section presented the maximum measured value (51.90×10^{-2} mm), a consumption of 83.71% of its service life during the first rainy season. This can impact fatigue consumption over the next year and will cause these deformations to reach values above the allowable limit in the next rainy season.

5. Conclusions

In summary, it can be concluded that the mixtures applied in the base and sub-base behaved differently from some characteristics observed in the laboratory, which culminated in the appearance of permanent cracks and deformations as a result of high deflections in less than a year after the construction of the section. This behavior was verified in later controls, especially associated with LWD tests.

It was found that the material applied in the sub-base layer is the main responsible for the poor quality of the pavement since this layer is very deformable and causes the layers above to experience greater deformations than those if this layer were competent.

The studies enabled the conclusion of the possibility of applying mining waste in pavement works in more conservative proportions due to the possibility of dispersing the characteristics of the materials from the beneficiation process. However, the results showed that the composition applied in the base layer obtained consistent field results for possible use in the sub-base layer.

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