MECHANICAL BEHAVIOUR OF TWO CRUSHED MATERIALS USED IN PORTUGUESE UGL

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Abstract

The crushed materials of extensive granulometry (UGM) are often used as unbound granular layers of road pavements, namely as granular sub-base and base. The behaviour of these materials on pavement layers is not sufficiently characterized, in spite of several studies already performed on this matter, due to reasons connected to the heterogeneity of the rock masses from which they come from. This has special importance for the Portuguese pavement technology. In the attempt of contributing for a better knowledge of that behaviour, a work was developed with the objective of obtaining the mechanical characterization and the establishment of behaviour models for crushed materials coming from different lithologies, namely limestone and granite, susceptible of being used as UGM. This paper describes the principal results obtained from this work and points out the main directives that can be extracted from it, in terms of the global behaviour of a road pavement.

INTRODUTION

In this paper we analyse the behaviour of two crushed materials of extended grading, limestone and granite, used as unbound granular sub-base of road pavements in Portugal. We performed the geotechnical characterization through tests such as the blue methylene or the micro-Deval as well as the characterization of the mechanical behaviour, using cyclic triaxial tests, according to the standard AASHTO TP 46 (AASHTO, 1994). The aim is, in the ambit of a PhD thesis, to contribute to the modelling of the behaviour of this type of material.

USED MATERIALS

In the project two types of materials were used: limestone and granite. The number of characterized samples for each material was: 5 samples of crushed limestone, from Pombal, in the centre of Portugal, and 3 samples of crushed granite, 2 of them from the outcrops near Celorico da Beira and the 3rd one was from Braga, in the interior centre and north of Portugal, Figure 1 and Figure 2.

Figure 1. Used materials: limestone

Figure 2. Used materials: granite

All materials examined were used in granular sub-base of pavements constructed or under construction in Portugal, namely in the motorway A23, fragment of Castelo-Branco Sul - Fratel, centre of Portugal, where the limestone has been used.

GEOTECHNICAL CHARACTERIZATION

A set of lab tests was performed on the collected samples in order to observe the evaluation of their characteristics concerning the granulometry distribution, hardness, resistance and water susceptibility.

The following tests were performed: Los Angeles (LNEC, 1970), the micro-Deval, (IPQ, 2002), the sand equivalent (LNEC, 1967b) and the methylene blue (AFNOR,1990), the California bearing ratio (CBR) (LNEC, 1967a) and compaction, which, due to the grading characteristics of the material, was executed by vibration, according to the BS 1377: part 4 standard (BSI, 1990), compacting specimens with the thickness varying between 127 mm and 133 mm in 3 layers for about 60 seconds each.

The results of the grading analysis are presented in Figure 3 (using the Portuguese road national administration specifications as reference) and the results of geotechnical characterization are presented in Table 1.

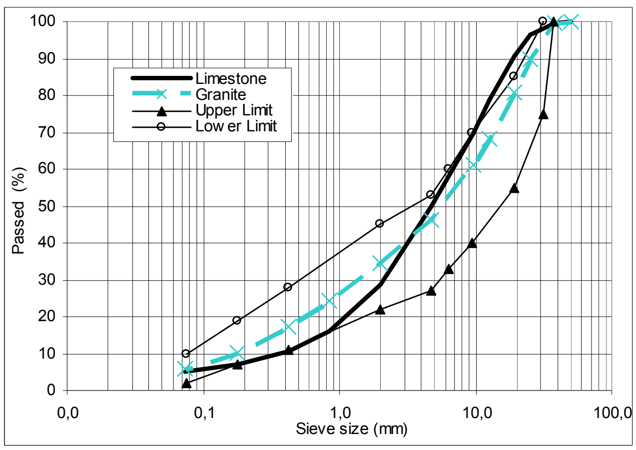

Figure 3. Gradation analysis results using as reference the upper and lower limits of the Portuguese specifications

Table 1. Results of the characterization tests

Parameter	Unit	Limestone	Granite
Optimum moisture content	%	3.6	3.5
Maximum dry density	kN/m^3	22.9	21.7
CBR	%	99	84
Swell	%	0	0
Los Angeles	%	33	37
Micro-Deval	%	14	21
Sand equivalent	%	70	61
Methylene blue (0/0.075 mm)	g/100g	0.88	1.55
Methylene blue (0/38.1 mm)	g/100g	0.05	0.07

MECHANICAL BEHAVIOUR CHARACTERIZATION

The laboratory mechanical characterization of the materials was done by cyclic triaxial tests, according to AASHTO TP 46 standard (AASHTO, 1994). The test has 16 sequences, with variation of the stresses, where the first one, with 1000 cycles, corresponds to the confinement of the sample, and the other 15, with 100 cycles each, correspond to the resilient modulus, Table 2.

Table 2. Load conditions and resilient modulus obtained from cyclic triaxial tests

Sequence	Load conditions (kPa)				n^{er} cycles
	σ_3	σ_{max}	σ_{cyclic}	$\sigma_{contact}$	
0	103.4	103.4	93.1	10.3	1000
1	20.7	20.7	18.6	2.1	100
2	20.7	41.4	37.3	4.1	100
3	20.7	62.1	55.9	6.2	100
4	34.5	34.5	31.0	3.5	100
5	34.5	68.9	62.0	6.9	100
6	34.5	103.4	93.1	10.3	100
7	68.9	68.9	62.0	6.9	100
8	68.9	137.9	124.1	13.8	100
9	68.9	206.8	186.1	20.7	100
10	103.4	68.9	62.0	6.9	100
11	103.4	103.4	93.1	10.3	100
12	103.4	206.8	186.1	20.7	100
13	137.9	103.4	93.1	10.3	100
14	137.9	137.9	124.1	13.8	100
15	137.9	275.8	248.2	27.6	100

The duration of each cycle is 1 second. The phase of load corresponds to 0.1 second and the phase of rest to 0.9 second, Figure 4.

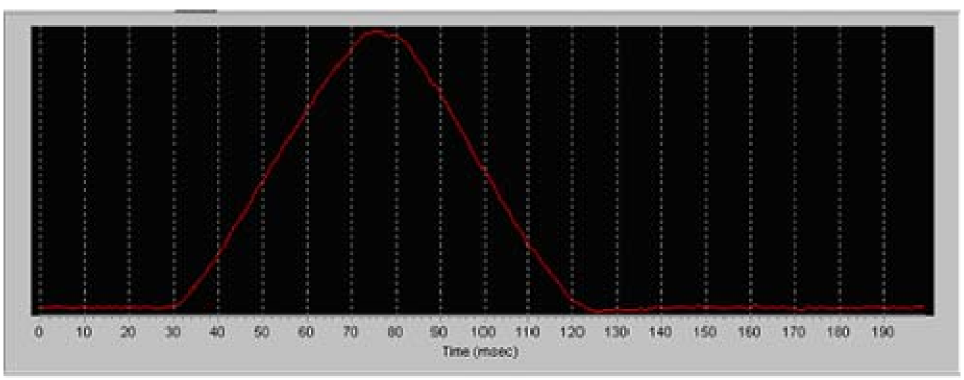

Figure 4. Part of the sinusoidal curve correspondent to a cycle in the cycle triaxial test

From the test is obtained the resilient modulus, Mr in Equation 1, corresponding to each one of the 15 sequences. This value is the average found in the last 5 cycles of each sequence.

$$M_r = \frac{\sigma_{cyclic}}{\varepsilon_r} = \frac{\sigma_1 - \sigma_3}{\varepsilon_r} \quad \text{MPa} \tag{1}$$

where: σ_{cyclic} - resilient stress;
ε_r - resilient axial strain;
σ_1-σ_3 - differential stress.

The cyclic triaxial equipment, that exists in the Lab of Road Pavement Mechanics of the Department of Civil Engineering of the University of Coimbra, Figure 5, consists of a triaxial load frame of 100 kN of capacity, with a triaxial cell for 160mm x 300 mm specimens, 8 channels for control and data acquisition and a 25 kN load cell and compressor.

Figure 5. Triaxial equipment of Lab of Road Pavement Mechanics of the Department of Civil Engineering of the University of Coimbra

The compaction of the specimens, with 150 mm diameter and 300 mm high was executed with a vibrating hammer with the following characteristics: frequency of percussion = 2750 impacts by minute, absorbed power = 750 W and diameter of compactor head = 147 mm.

The specimens tested, in Figure 6, were compacted for two conditions of compaction: the density and moisture content obtained in the lab conditions, that is, 95% of the maximum dry density and optimum moisture content, and the conditions of in situ compaction the material. The average values of these quantities for limestone and laboratory conditions are 21.7 kN/m^3 and 3.6% and

22.7 kN/m^3 and 3.5%, respectively. For the granite the average values are 21.1 kN/m^3 and 4.3 % and 22.1 kN/m^3 and 4.2 %, respectively for laboratory and in situ conditions.

Figure 6. Limestone specimen: after compaction, with the membrane, in the camera during the test and after the test

All the cyclic triaxial tests were performed using the conditions of the load presented in Table 2. The resilient modulus obtained for each material in the aforementioned conditions are presented in Table 3.

Table 3. Resilient modulus obtained from cyclic triaxial tests

Sequence	Average Mr. (MPa)			
	Limestone		Granite	
	L. C.	In situ C.	L. C.	In situ C.
0	-	-	-	-
1	163	164	88	80
2	201	196	102	91
3	214	222	112	102
4	207	221	116	103
5	240	273	136	122
6	259	301	153	138
7	293	339	187	164
8	331	414	212	194
9	352	450	228	212
10	318	381	217	186
11	341	425	231	210
12	392	514	269	245
13	376	479	265	236
14	394	498	284	250
15	453	612	317	294

L.C. Laboratory conditions; In situ C. In situ conditions

Analysing the results it can be said that the resilient modulus presents an expected variation, which means higher for higher confining pressures and increasing values for increasing differential stresses (σ_{cyclic}).

The permanent deformation during the test, varied between 0.4 % and 1.4 % for limestone and between 1.2 % and 2.4 % for the granite

Approaching the resilient modulus modelling, some behaviour models (Lekarp, 2000; NCHRP, 1998), generally used in granular materials mechanical behaviour modelling were adjusted to the tests results, namely the models Dunlap, k-θ, differential stress, Pezo and Uzan, represented in Equations 2 to 6. The results of this adjustment are presented in Tables 4 and 5 (Luzia, 2005).

$$M_r = k_1\sigma_3^{k2} \quad (2)$$

$$M_r = k_3\theta^{k4} \quad (3)$$

$$M_r = k_5\sigma_d^{k6} \quad (4)$$

$$M_r = k_7 q^{k8}\sigma_3^{k9} \quad (5)$$

$$M_r = k_{10}\theta^{k11} q^{k12} \quad (6)$$

where: M_r - resilient modulus
σ_3 - confining stress
θ - first invariant of stress ($\theta = \sigma_1 + \sigma_2 + \sigma_3$)
σ_d - differential stress ($\sigma_d = q = \sigma_1 - \sigma_3$)
k_1 to k_{12} - material constants

Table 4. Model results for limestone

Laboratory conditions	r^2	in situ conditions	r^2
$Mr = 880.91\sigma_3^{0.3916}$	0.8914	$Mr = 1488.00\sigma_3^{0.5195}$	0.8898
$Mr = 522.13\theta^{0.4388}$	0.8914	$Mr = 744.47\theta^{0.5832}$	0.9857
$Mr = 771.22\sigma_d^{0.3854}$	0.8347	$Mr = 1256.10\sigma_d^{0.5140}$	0.8423
$Mr = 583.98\theta^{0.3672}q^{0.0821}$	0.9963	$Mr = 883.67\theta^{0.4647}q^{0.1301}$	0.9981
$Mr = 973.52q^{0.1930}\sigma_3^{0.2543}$	0.9973	$Mr = 1681.55q^{0.2696}\sigma_3^{0.3215}$	0.9988

Table 5. Model results for granite

Laboratory conditions	r^2	in situ conditions	r^2
$Mr = 863.241\sigma_3^{0.5521}$	0.9401	$Mr = 770.65\sigma_3^{0.5495}$	0.9213
$Mr = 406.38\theta^{0.6067}$	0.9981	$Mr = 366.57\theta^{0.6088}$	0.9945
$Mr = 654.05\sigma_d^{0.5078}$	0.7691	$Mr = 607.53\sigma_d^{0.5204}$	0.7995
$Mr = 417.43\theta^{0.5902}q^{0.0193}$	0.9982	$Mr = 408.43\theta^{0.5482}q^{0.0753}$	0.9982
$Mr = 945.90q^{0.1954}\sigma_3^{0.4093}$	0.9986	$Mr = 872.65q^{0.2388}\sigma_3^{0.3798}$	0.9990

Analysing the results we can say that for all models, the correlations obtained are of reasonable to very good quality, with determination coefficients varying between 0.7691 and 0.9990.

Aiming at establishing a unique model to represent the material's behaviour, the better and more conservative one was elected. Better means the model with determination coefficient closest to 1 and, conservative means the model that delivers lower values of resilient modulus. The elected model is presented in Equation 7 (Luzia, 2005).

$$M_r = 877{,}37 q^{0{,}2384} \sigma_3^{0{,}3828} \tag{7}$$

where: M_r - resilient modulus;
σ_3 - confining stress;
q -differential stress.

The in situ mechanical characterization was made with the Falling Weight Deflectometer of Coimbra and Minho Universities and the deformability modulus obtained to the sub-base layer was, approximately, 570 MPa for the limestone and 250 MPa for the granite.

ANALYSIS OF MODELISATION RESULTS

On trying to confirm the practical applicability of the aforementioned model, a simple parametric study for a typical Portuguese flexible pavement was performed using a linear-elastic structural approach.

Regarding the granular layers, the parametric study mainly consisted in the stresses determination at the centre of the granular layer, taking into consideration the linear-elastic behaviour for materials, modulus (granular layer values from 100 MPa to 250 MPa are current) and Poisson coefficients generally used in Portuguese pavement design practice, and then, calculate the modulus falling back upon the found model, Equation 7, with the obtained stresses. This has given in the first approach a very different granular layer modulus than the one that the calculation departs from.

Proceeding now with the modulus obtained by Equation 7 (maintaining all other characteristics for all the layers), the analysis stops when the stress state obtained by the calculation with that modulus equals the stress state that produces the modulus (with Equation 7) which has launched the calculation. It has been found that the obtained modulus varying from 40 MPa to 60 MPa. This means that they are much lower, 2.5 to 3 times, than the ones from which we have departed in the beginning and also very different from the ones obtained with AASHTO TP 46 procedure and from those resulting from FWD results analysis.

The explanation for this could be:

- For the cyclic triaxial tests, the fact that the confining stress used during the test is always higher than the effectively installed in an unbound granular layer of a real traffic loaded flexible pavement.
- For the in situ characterization using the FWD, it is probably because there was a suction phenomenon in the unbound granular layers, caused by the variations in the

moisture content after compaction due to climatic changes during summer time and some moisture reposition during winter period. This phenomenon could result in higher stress state for the unbound layers and then in higher modulus.

CONCLUSIONS

Analysing the characterization results of the two materials, we may conclude that they are not plastic, given the values of adsorption of the methylene blue obtained.

We also conclude that it is a material with good overall strength regarding average CBR values, which range between 85 % and 99 %, as well as a good resistance to wear by abrasion and impact, taking into account the results of the Los Angeles and micro-Deval tests.

With respect to the mechanical behaviour, we found values of the resilient modulus variable between, approximately, 160 MPa and 600 MPa, according to the limestone and between 80 MPa and 300 MPa according to the granite.

We verified, on the other hand, that the permanent deformation during the test, varied between 0.4 % and 1.4 % for the limestone and 1.2 % and 2.4 % for the granite.

In terms of resilient modulus, the modelling verified that the better simulation of the resilient behaviour of the two materials is obtained by Equation 7, which relates the modulus with the differential stress (q) and the confining stress (σ_3).

The resilient modulus obtained from a parametric study aimed to represent the site performance for unbound granular layers leads to values of 40 to 60 MPa, which is 2.5 to 3 times lower than those usually used in the pavement design and obtained, most of the time, from laboratory and in situ characterization.

That means that the usual flexible pavement design approach is missing the real stress state in unbound granular layers failing to use a truly mechanical characterization of these layers. This is probably the reason responsible for design failures on pavements with low thickness bituminous mixtures layers (under 15 cm in total thickness). Everyone could witness this if they made the comparison between a traditional empirical-mechanical design (for instance using Shell approach) with an analysis made in the same way as described above, which basically means using equation 7 to characterise the unbound layers and the stress state at its mid thickness and doing the same analysis for the subgrade characterization, in this case using the stress state 1 mm under sub-base.

Finally, these alerts aim to underline the importance of the research that clarifies the design stress states for unbound layers and subgrades, in such a way that one could make a precise mechanical characterization when designing flexible road pavements.

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