Mechanical behaviour of UGM for Portuguese conditions

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ABSTRACT: Coarse aggregate is widely used in the unbound granular layers (UGM) of roads, in particular as granular sub-base and base. However, although various studies have been conducted on these materials, their mechanical behaviour has not been yet properly characterized, in Portuguese conditions. This has special importance for Portuguese pavement technology. In the attempt of contributing for a better knowledge of that behaviour, an experimental work was undertaken to access the structural performance and to establish 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 pointing out the main directions that can be extracted from it, in terms of the global behaviour of a road pavement.

1 INTRODUCTION

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

2 MATERIALS

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

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.

3 GEOTHECNICAL CHARACTERIZATION

The collected samples were subjected to a set of laboratory tests to evaluate their geotechnical characteristics: the Los Angeles test (LNEC, 1970), the micro-Deval test (IPQ, 2002), the sand equivalent test (LNEC, 1967b), the methylene blue test (AFNOR, 1990) and the California Bearing Ratio (CBR) test (LNEC, 1967a).

Figure 1. Used materials: limestone.

Figure 2. Used materials: granite.

The results of the grading analysis are presented in Figure 4 (using the Portuguese road national administration specifications as reference) and the results of geotechnical characterization are presented in Table 1.

Figure 3. Location of the quarries and the pavements where the materials were used

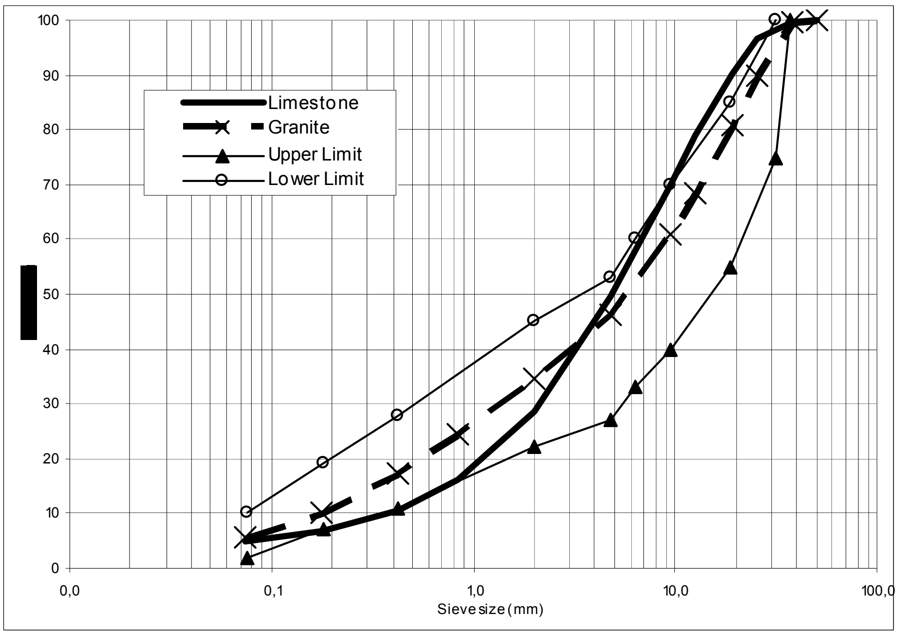

Figure 4. Gradation analysis results using as reference the upper and lower limits of the Portuguese specifications

Table 1. Results of the identification tests.

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

4 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, consisting of 1000 cycles, produces the confinement of the sample, and the next 15 successive loading sequences of 100 cycles, provide the resilient modulus. The load conditions are the ones presented in Table 2.

Table 2. Load conditions of the cyclic triaxial tests

Sequence	Base/subbase materials σ_3 kPa	σ_{max} kPa	σ_{cyclic} kPa	$\sigma_{contact}$ kPa	Cycles
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	62.5	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

σ_3 Confining pressure
σ_{max} Maximum axial stress
σ_{cyclic} Cyclic axial stress (resilient stress)
$\sigma_{contacto}$ Contact stress

The duration of each cycle is 1 second, Figure 5. The phase of load corresponds to 0.1 second and the phase of rest to 0.9 second.

This testing procedure is used for the determination of the resilient modulus, M_r in Equation 1, corresponding to each one of the 16 sequences. This value is the average derived from the 5 last cycles of each sequence.

$$M_r = \frac{\sigma_{cyclic}}{\varepsilon_r} = \frac{\sigma_1 - \sigma_3}{\varepsilon_r} \quad (1)$$

where σ_{cyclic} - resilient stress, ε_r - resilient axial strain and σ_1-σ_3 - differential stress

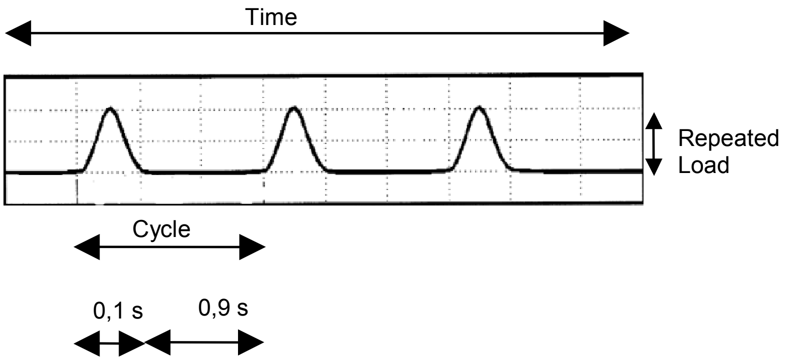

Figure 5- Typical load versus time relationship (adap. FHWA, 1996)

The triaxial equipment of the Laboratory of Road Pavement Mechanics of the Department of Civil Engineering of the University of Coimbra, Figure 6, 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 6 Triaxial equipment of Laboratory of Road Pavement Mechanics of the Department of Civil Engineering of the University of Coimbra

The compaction of the specimens, 150 mm diameter and 300 mm high, was carried out by means of a vibrating hammer with the characteristics presented in Table 3.

Table 3. Vibrating hammer characteristics.

Parameter	Value
Frequency of percussion	2750 impacts by minute
Absorbed power	750 W
Diameter of compactor head	147 mm

For all the materials cyclic triaxial tests were performed using the load conditions presented in Table 2 and 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. Average values of these parameters are presented in Table 4 and the resilient modulus values obtained for each material, are presented in Table 5.

Table 4. Compaction conditions used on specimens

	Lab. Conditions		In situ conditions	
Material	w (%)	γ_d (g/cm^3)	w (%)	γ_d (g/cm^3)
Limestone	3.6	2.17	3.5	2.27
Granite	4.3	2.11	4.2	2.21

Table 5. Average values for the resilient modulus

	M_r (MPa)			
Sequence	Limestone		Granite	
	Lab. Cond.	Lab. Cond.	Lab. Cond.	Lab. Cond.
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

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 % to 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 6 and 7 (Luzia, 2005).

$$M_r = k_1\sigma_3^{k2} \tag{2}$$

$$M_r = k_3\theta^{k4} \tag{3}$$

$$M_r = k_5\sigma_d^{k6} \tag{4}$$

$$M_r = k_7 q^{k8}\sigma_3^{k9} \tag{5}$$

$$M_r = k_{10}\theta^{k11}q^{k12} \tag{6}$$

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 6. Modelisation results for limestone modulus

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

Table 7. Modelisation results for granite modulus

Lab. Conditions	r^2	In situ conditions	r^2
$M_r = 863.241\sigma_3^{0.}$	0.9401	$M_r = 770.65\sigma_3^{0.5495}$	0.9213
$M_r = 406.38\theta^{0.6067}$	0.9981	$M_r = 366.57\theta^{0.6088}$	0.9945
$M_r = 654.05\sigma_d^{0.5078}$	0.7691	$M_r = 607.53\sigma_d^{0.5204}$	0.7995
$M_r = 417.43\theta^{0.5902}q^{0.0193}$	0.9982	$M_r = 408.43\theta^{0.5482}q^{0.0753}$	0.9982
$M_r = 945.90q^{0.1954}\sigma_3^{0.4093}$	0.9986	$M_r = 872.65q^{0.2388}\sigma_3^{0.3798}$	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 the establishment of a unique model to represent the materials behaviour, the better and more conservative one was elected. This means in one hand the one having determination coefficient more closed to 1 and, on the other hand, the one which gives lower values of resilient modulus, being more conservative than the others. The elected model is presented in Equation 7 (Luzia, 2005).

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

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.

5 ANALYSIS OF MODELISATION RESULTS

On trying to confirm the practical applicability of the aforementioned modelisation, 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, considering for that 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.

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 had 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 begin and also very different from the ones obtained with AASHTO TP 46 procedure and from those resulting from FWD results analysis.

The explanation for that 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 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.

6 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, to the limestone and between 80 MPa and 300 MPa 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 the usually used in the pavement design and than the 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 responsible of design failures on pavements with low thickness bituminous mixtures layers (under 15 cm in total thickness). Everyone could witness this if makes the comparison between a traditional empirical-mechanical design (for instance using Shell approach) with an analysis made in the same way 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 to 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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