THE USE OF CYCLIC TRIAXIAL TESTS IN THE CHARACTERIZATION OF PROTUGUESE UGM

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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 still has not been properly characterized, in Portuguese conditions, especially due to reasons connected to the heterogeneity of the rock masses from which they come from. This has special importance for Portuguese pavement technology. In the attempt of contributing for a better knowledge of that behaviour, a work was developed having the aim of obtain 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 the work and pointing out the main directives that can be extracted from it, in terms of the global behaviour of a road pavement.

USED MATERIALS

The materials used in these work were limestone and granite, Figure 1. They were characterized 5 samples of crushed limestone, from Pombal, centre of Portugal, and 3 samples of crushed granite, 2 of them outcrops near Celorico da Beira and the 3rd near Braga, interior centre and north of Portugal.

All the materials were used in granular subbase of pavements constructed in Portugal, namely in the motorway A23, fragment of Castelo-Branco Sul - Fratel, center of Portugal, where it has been used the limestone.

GEOTECHNICAL AND MECHANICAL BEHAVIOUR CHARACTERIZATION

Geotechnical Characterization

On the collected samples a set of lab tests was per-formed in view to the evaluation of their characteristics in what concerns the granulometry distribution, hardness, resistance and water susceptibility.

a) b)

Figure 1. Used materials: a) limestone; b) granite

For that the following tests were performed: Los Angeles (LNEC E 237), the micro-Deval, Figure 2 (NP EN 1097-1), the sand equivalent (LNEC E 199) and the methylene blue (NF P 18-592), the California bearing ratio (CBR) (LNEC E 198) 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 60 seconds each.

The results of the grading analysis are presented in Table 1 and Figure 3 (using the Portuguese road national administration specifications as reference) and the results of geotechnical characterization are presented in Table 2.

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.

Figure 2. Micro-Deval test equipment

Table 1. Results of the grading analysis

Sieve		Passed (%)	
nº	Opening (mm)	Limestone	Granite
2”	50.80	100	100
1” 1/2	38.10	99.8	99.6
1”	25.40	96.6	89.9
3/4”	19.10	90.4	80.8
1/2”	12.70	78.8	68.3
3/8”	9.520	69.9	61
4	4.760	49.8	46.3
10	2.000	28.5	34.4
20	0.840	16.1	24.5
40	0.420	10.5	17.2
80	0.177	6.9	10.2
200	0.074	5.0	5.8

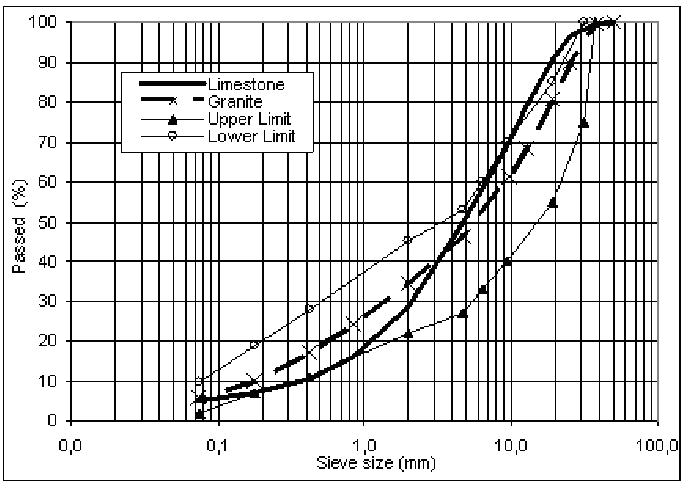

Figure 3. Gradation analysis results using as reference the upper and lower limits of the Portuguese specifications

Table 2.. 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	g/100g	0.88	1.55
Methylene blue (0/38.1 mm)	g/100g	0.05	0.07

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.

From the test is obtained the resilient modulus, Mr in Equation 1, corresponding to each one of the 16 sequences. This value is the average found for the 5 last cycles of each sequence.

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

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 4, 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.

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

The specimens tested 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. Average values of these quantities are for limestone and laboratory conditions 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 4. Triaxial equipment of Lab of Road Pavement Mechanics of the Department of Civil Engineering of the University of Coimbra

All the cyclic triaxial tests were performed using the conditions of load presented in Table 3. In the same table is presented the resilient modulus obtained for each material and in the aforementioned conditions.

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

To the resilient modulus, we tried to adjust some behaviour models (Lekarp et al., 2000; NCHRP, 1998) generally used in granular materials mechanical behaviour modelisation, namely Dunlap ($Mr = k_1\sigma_3^{k2}$), k-θ ($Mr = k_3\theta^{k4}$), differential stress ($Mr = k_5\sigma_d^{k6}$), Tom and Brown ($Mr = k_7(p/q)^{k8}$), Pezo ($Mr = k_9q^{k10}\sigma_3^{k11}$) and Uzan ($Mr = k_{12}\theta^{k13}q^{k14}$). The results of this modeling are presented in Table 4.

After that, it was chosen the better and more conservative one, what means, the one having determination coefficient more closed to 1 and, on the other hand, the one which gives lower values of resilient modulus. The obtained is the model presented in Equation. 2.

$$Mr = 877{,}37q^{0{,}2384}\sigma_3^{0{,}3828} \qquad (2)$$

where:

M_r - resilient modulus

σ_3 - confining stress
q - differential stress

Table 3. Load conditions and resilient modulus obtained from cyclic triaxial tests

Seq.	Load conditions (kPa)				n^{er} cycles	Average Mr. (MPa)			
						Limestone		Granite	
	σ_3	σ_{max}	σ_{cyclic}	$\sigma_{contact}$		L. C.	In situ C.	L. C.	In situ C.
0	103.4	103.4	93.1	10.3	1000	-	-	-	-
1	20.7	20.7	18.6	2.1	100	163	164	88	80
2	20.7	41.4	37.3	4.1	100	201	196	102	91
3	20.7	62.1	55.9	6.2	100	214	222	112	102
4	34.5	34.5	31.0	3.5	100	207	221	116	103
5	34.5	68.9	62.0	6.9	100	240	273	136	122
6	34.5	103.4	93.1	10.3	100	259	301	153	138
7	68.9	68.9	62.0	6.9	100	293	339	187	164
8	68.9	137.9	124.1	13.8	100	331	414	212	194
9	68.9	206.8	186.1	20.7	100	352	450	228	212
10	103.4	68.9	62.0	6.9	100	318	381	217	186
11	103.4	103.4	93.1	10.3	100	341	425	231	210
12	103.4	206.8	186.1	20.7	100	392	514	269	245
13	137.9	103.4	93.1	10.3	100	376	479	265	236
14	137.9	137.9	124.1	13.8	100	394	498	284	250
15	137.9	275.8	248.2	27.6	100	453	612	317	294

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

Table 4. Modelisation results of limestone and granite

Laboratory conditions	r^2	in situ conditions	r^2
Limestone			
$Mr = 880.91\sigma_3^{0.3916}$	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
$Mr = 288.82(p/q)^{0.0533}$	0.0041	$Mr = 339.19(p/q)^{0.0634}$	0.0033
$Mr = 583.98\theta^{0.3672}q^{0.0821}$	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
Granite			
$Mr = 863.241\sigma_3^{0.5521}$	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
$Mr = 177.49(p/q)^{0.1718}$	0.0224	$Mr = 160.33(p/q)^{0.1295}$	0.0126
$Mr = 417.43\theta^{0.5902}q^{0.0193}$	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

The in situ mechanical characterization was made with the Falling Weight Deflectometer of Coimbra and Minho Universities, Figure 5, and the deformability modulus obtained to the sub-base layer was, approximately, 570 MPa for the limestone and 250 MPa for the granite.

Figure 5. Falling Weight Deflectometer of Coimbra and Minho Universities

ANALYSIS OF MODELISATION RESULTS

On trying to confirm the values of resilient modulus obtained from cyclic triaxial tests, was done, in a typical Portuguese pavement, a small parametric study using Elsym5 and Bisar.

It consisted in the determination of the stresses to middle of the granular layer, considering for that the linear-elastic behaviour for materials and typical modules and Poisson coefficients, generally used in Portuguese pavement design practice, and then, calculate the module falling back upon the found model, Equation 2, with the obtained stresses.

The calculated values of resilient modulus, using that procedure, vary from 40 MPa to 60 MPa, so they are much more lower, 2.5 to 3 times, than the ones from which we departed. Because of that, the same procedure has been used with the results of FWD and the calculated values of resilient modulus were comparable.

The explanation for those values could be, for the cyclic triaxial tests, the confining stress used during the test, which is higher than the installed in an unbound granular layer and for the in situ characterization, a suction phenomenon that could happen in the unbound granular layers, caused by the variations in the moisture content after compaction, because of climacteric changes during summer time and some moisture reposition during winter period.

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 resistance regarding average CBR values, which range between 85 % and 99 %, as well as a good resistance to deterioration 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 the resilient modulus modelling it was verified that the better simulation of the resilient behaviour of the two materials is obtained by Equation.2, which relates the modulus with the differential stress (q) and the confining stress (σ_3).

The resilient modulus obtained from a parametric study using Elsym5 and Bisar, 40 a 60 MPa, is 2.5 to 3 times lower than the usually used in the design and generally obtained from tests, which are, probably, the real values of UGM resilient modulus, unless they are subject to suction phenomena.

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