BEHAVIOUR MODELS FOR UGM USING CYCLIC TRI-AXIAL TESTS

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ABSTRACT: The behaviour of unbound granular materials (UGM) on pavement granular layers, in spite of several studies already performed on this matter, is not enough characterized, especially under Portuguese conditions due to reasons connected to the heterogeneity of the rock masses from which they come from. In the attempt of contributing for a better knowledge of that behaviour, a research programme is currently underway with the main objective being the mechanical characterization and the establishment of behaviour models for two types of crushed materials i.e. granite and limestone, susceptible of being used as unbound base and sub-base of Portuguese road pavements. This paper describes the results found to date showing the differences and the meeting points with some previous studies on similar materials conducted with other approaches and pointing out the main directives that can be extracted in terms the global behaviour of pavements.

KEY WORDS: Behaviour models, cyclic tri-axial test, granular layers, pavement, resilient modulus, UGM

1. INTRODUCTION

This paper considers the behaviour of crushed limestone and granite for use as unbound granular subbase in Portuguese road constriction. Geo-technical characterization includes such test s as the methylene blue and micro-Deval test methods. Their mechanical behaviour is assed using cyclic tri-axial tests according to the standard AASHTO TP 46 [1]. The aim is to contribute to the behaviour modelling of these types of material.

2. MATERIALS USED

The materials used in this work were limestone and granite. These are shown in Figure 1a and 1b. They consisted of 5 samples of crushed limestone from Pombalin in the centre of Portugal. There was 3 samples of crushed granite, two from granite outcrops near Celorico da Beira with the third from near Braga. All the materials have been as granular subbase in pavements construction. For example, the A23 motorway at Castelo-Branco Sul - Fratel, in the centre of Portugal.

(a)

(b)

Figure 1. The materials used in the study (a) limestone (b) granite

3. GEOTHECNICAL CHARACTERIZATION

A set of laboratory tests was carried out on the aggregate samples to evaluate of their geothecnical characteristics. This included the Los Angeles test [2], micro-Deval test [3] (see equipment in Figure 2a), Sand Equivalent test [4], methylene blue test [5] (see equipment in Figure 2b) and the California Bearing Ratio test (CBR) [6].

(a) (b)

Figure 2. (a) Micro-Deval test equipment (b) Methylene blue test equipment

Due to the grading characteristics of the material, the compaction was done by vibration according to the BS 1377: Part 4 standard [7]. This compacted the specimens in 3 layers with compaction lasting 60 seconds for each layer. The compactor is shown in Figure 3 and had the following characteristics:

- Frequency of percussion: 1800-3000 impacts by minute
- Absorbed power: 750-1200 W
- Diameter of compactor head: not less than 146 mm

The results of geo-technical characterization are given in Figure 4 and Table 1.

Figure 3. Vibrating hammer used for compaction

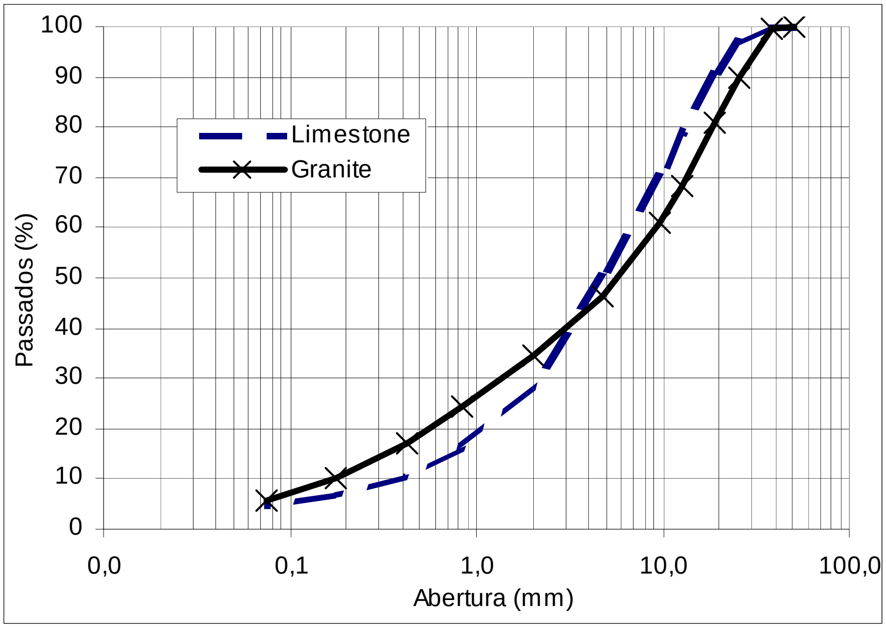


Figure 4. Results of the grading analysis [8]

Table 1. Results of the characterization tests

Parameter	Unit	Average value	
		Limestone	Granite
Optimum moisture content	%	3.5	3.7
Maximum dry density	g/cm^3	2.26	2.13
CBR	%	92	84
Swell	%	0	0
Los Angeles	%	33	37
Micro-Deval	%	14	21
Sand equivalent	%	65	65
Blue methylene (0/0,075 mm)	g/100g	0.88	1.23
Blue methylene (0/38,1 mm)	g/100g	0.05	0.07

4. MECHANICAL BEHAVIOUR CHARACTERIZATION

The laboratory mechanical characterization was done using cyclic tri-axial testing according to the AASHTO TP 46 standard [1]. The test has 16 sequences, with variation of the stresses. The first sequence of 1000 cycles corresponds to confinement of the sample with the remaining 15 sequences of 100 cycles, corresponding to the resilient modulus. As shown in figure 5 the duration of each cycle was 1 second. The load phase lasted 0,1s with a rest of 0,9s.

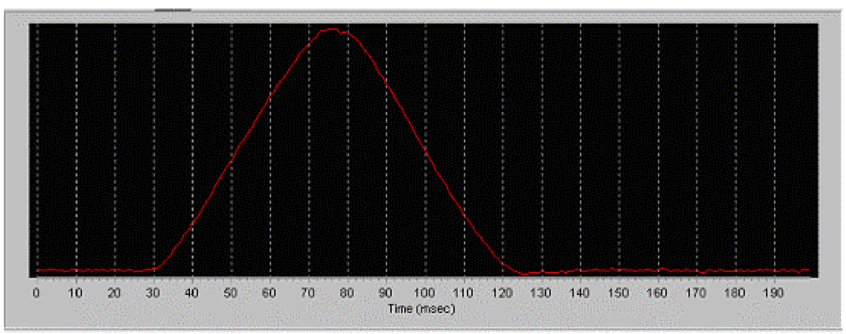


Figure 5. Part of the sinusoidal curve during the tri-axial test

During testing the vertical movements were measured and recorded at two points using LVDTs placed outside the tri-axial camera. Equation 1 shows Resilient Modulus may be calculated.

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

Where:

Mr - resilient modulus

σ_{cyclic} - resilient stress;

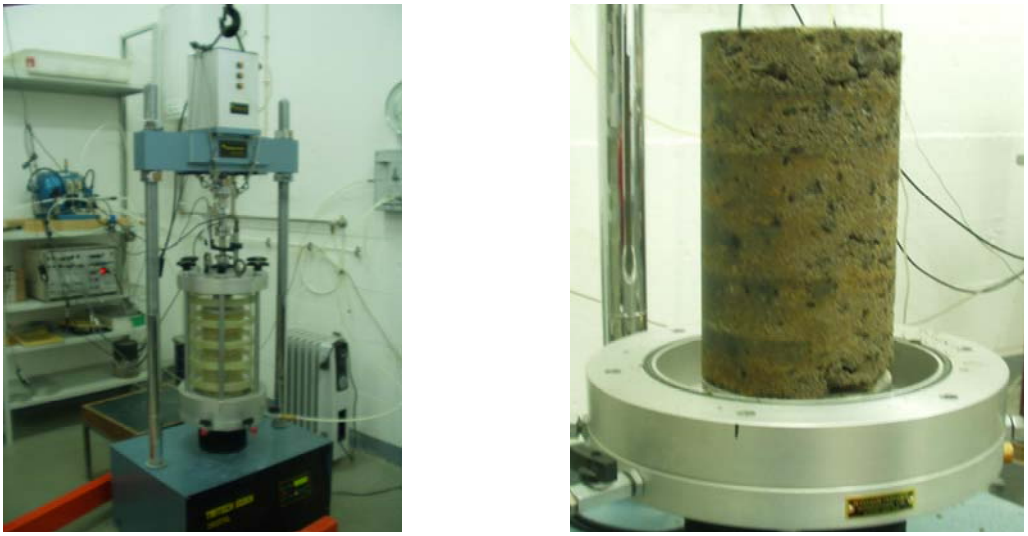
ε_r - resilient axial strain

σ_1-σ_3 - differential stress

The tri-axial equipment was located in the Road Pavement Mechanics laboratory of the Department of Civil Engineering of the University of Coimbra. Figure 6a shows the 100kN Wykheam Farrance tri-axial load frame. This has a tri-axial cell for testing 160mm x 300 mm specimens, 8 channels for control and data acquisition, and a 25kN load cell and compressor.

The test specimens were compacted using a vibrating hammer to be 150mm in diameter and 300mm in height. A compacted test specimen is shown in Figure 6b.

All materials were assessed using the test conditions given in Table 2. The degree of compaction is summarised in Table 3 with the resilient modulus given in Table 4.



(a) (b)

Figure 6. (a) Tri-axial equipment (b) Specimen of compacted granite

Table 2. Load conditions for each sequence of cyclic tri-axial test

Sequence	Base/subbase materials				n^{er} cycles
	σ_3 (kPa)	σ_{max} (kPa)	σ_{cyclic} (kPa)	$\sigma_{contact}$ (kPa)	
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

Table 3. Compaction conditions for each material

Material	Laboratory conditions		In situ conditions	
	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 4. Average values for the resilient modulus

Sequence	n^{er} cycles	Resilient modulus(MPa)			
		Limestone		Granite	
		L. C.	In situ C.	L. C.	In situ C.
1	100	163	164	88	80
2	100	201	196	102	91
3	100	214	222	112	102
4	100	207	221	116	103
5	100	240	273	136	122
6	100	259	301	153	138
7	100	293	339	187	164
8	100	331	414	212	194
9	100	352	450	228	212
10	100	318	381	217	186
11	100	341	425	231	210
12	100	392	514	269	245
13	100	376	479	265	236
14	100	394	498	284	250
15	100	453	612	317	294
Max		453	612	317	294
Min		163	164	88	80
Std Dev		85	134	73	67

LC: Laboratory conditions; in situ C: in situ conditions

The permanent deformation of the materials during testing varied from 0.4 % to 1.4 % for the limestone and 1.2 % to 2.4 % for the granite. The resilient modulus data was as expected i.e. generally higher for higher confining pressures and showing a positive increase for increasing differential stresses (σ_{cyclic}).

Different behaviour models were applied to the to the resilient modulus data [9, 10]. These included Dunlap, k-θ, differential stress, Tom and Brown, Pezo and Uzan, (equations 2 to 7). The results of this modelation are presented in Tables 5 and 6.

Model	Equation
$Mr = k_1\sigma_3^{k2}$	(2)
$Mr = k_3\theta^{k4}$	(3)
$Mr = k_5\sigma_d^{k6}$	(4)
$Mr = k_7(p/q)^{k8}$	(5)
$Mr = k_9q^{k10}\sigma_3^{k11}$	(6)
$Mr = k_{12}\theta^{k13}q^{k14}$	(7)

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 5. Modelation results for limestone

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
$Mr = 288.82(p/q)^{0.0533}$	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

Table 6. Modelation results for granite

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
$Mr = 177.49(p/q)^{0.1718}$	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

When using the Tom and Brown (p/q) model, there was no correlation because the determination coefficient, r^2, was about zero for the two materials and the two conditions of compaction. For the other models better correlations were obtained, with determination coefficients varying between 0.769 and 0.999. The best simulation, i.e. best r^2 for the two compaction conditions was obtained using the Pezo Model. The best results were obtained for the granite with in situ conditions of compaction, for which a determination coefficient of 0.999 was obtained.

5. COMPARISON OF MECHANICAL BEHAVIOUR WITH OTHER MATERIALS

The mechanical behaviour of the two materials characterized in this work was compared with a limestone used as the subgrade layer in the VLA road [11] and a limestone used as the subbase in the A6 motorway [12]. Although the conditions of the tri-axial cyclic test were not exactly the same, the results are comparable for the same level of stresses. These materials were characterized in the Laboratório Nacional de Engenharia Civil (LNEC) using compacted test specimens 300mm in diameter and 600mm high. The modulus test was carried out in 9 sequences of 150 cycles each. The conditions of compaction for each material are shown in Table 7. Table 8 shows the sequences of stresses used in the test and the respective resilient modulus.

The resilient modulus data was subjected to a modelation using equations (2) and (3) [13]. The results are presented in Table 9. Figures 7 and 8 show these models and the ones obtained for the materials characterized in this work.

Table 7. Compaction conditions and layer of VLA and A6 materials ([11,12,13])

Road	Material	Layer	Specimen	$\gamma_{d\ max}$ (g/cm^3)	w (%)
VLA	Limestone	Subgrade	CT1	2.1	5.2
			CT2		4.6
A 6	"Catbritas" Limestone	Subbase	1C	2.25	4.2
			2C		2.2
			3C		4.2

Table 8. Resilient modulus of VLA and A6 materials [11,12,13]

Road	σ_3 (kPa)	σ_1- σ_3 (kPa)	n^{er} cycles	Resilient modulus (MPa) 1C	2C	3C	CT1	CT2	Medium value
A6	35	35	150	269	283	308	-	-	287
		70		297	296	336	-	-	310
		105		329	327	379	-	-	345
	70	50		337	346	406	-	-	363
		100		368	375	442	-	-	395
		150		415	406	485	-	-	435
	105	70		438	436	506	-	-	460
		140		477	464	565	-	-	502
		210		524	526	622	-	-	557
VLA	50	150		-	-	-	582	638	610
	75	220		-	-	-	613	-	613

Table 9. Modelation results of VLA and A6 materials [11,12,13]

Road	Specimen	$Mr = k_1\sigma_3^{k2}$	r^2	$Mr = k_3\theta^{k4}$	r^2
VLA	CT1	$Mr = 1259\ \sigma_3^{0.27}$	0.9600	$Mr = 645\ \theta^{0.18}$	0.4600
	CT2	$Mr = 1257\ \sigma_3^{0.25}$	0.8400	$Mr = 1000\ \theta^{0.38}$	0.4000
A 6	1C	-	-	$Mr = 11{,}519\ \theta^{0.6333}$	0.9762
	2C	-	-	$Mr = 14{,}862\ \theta^{0.5877}$	0.9583
	3C	-	-	$Mr = 11{,}321\ \theta^{0.6650}$	0.9731

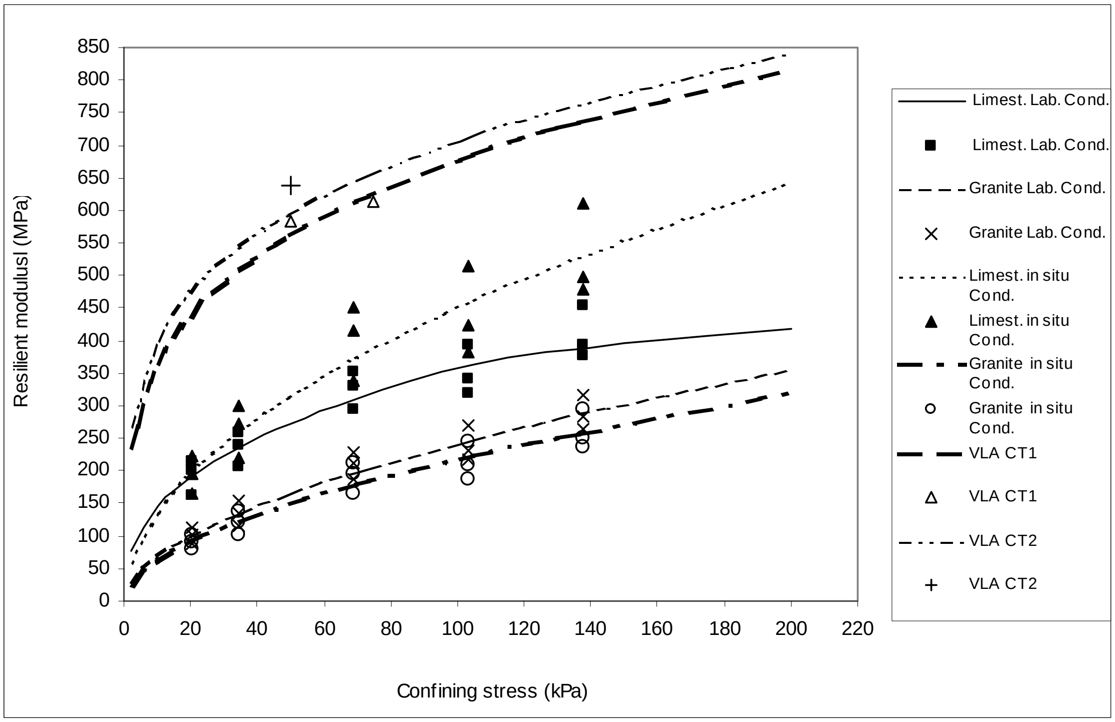


Figure 7- Resilient modulus and behaviour models ($Mr = k_1\sigma_3^{k2}$) of the materials characterized in this work and used in the VLA [11,12,13]

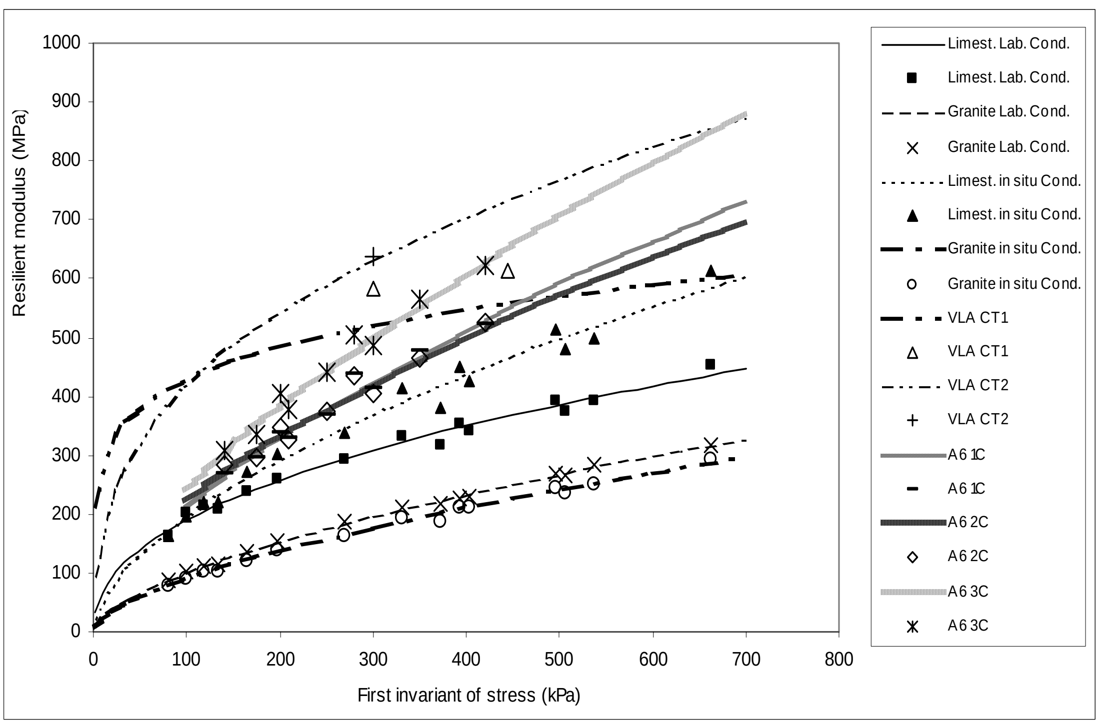


Figure 8- Resilient modulus and behaviour models ($Mr = k_3\theta^{k4}$) of the materials characterized in this work and used in the VLA and A6 [11,12,13]

Comparing the resilient modulus of the different materials, limestone from the A23 had lowest resilient modulus for laboratory compaction conditions. It differed differing significantly to the VLA source. i.e. approximately twice greater. For the in situ compaction conditions , values of the same order of greatness were obtained for the granites but higher for the limestone of A23, presenting values between 160 MPa and 600 MPa, close to the values presented by the limestone of VLA.

Relatively to the mechanical behaviour modelation, for two models referred in the case of VLA and one in the case of A6, is verified that, when the resilient modulus depends on confining stress, the adjustment for all the materials is not very good, since r^2 varies between 0.8898, for the limestone of A23 for conditions in situ, and 0.9600, for the specimen CT1 of VLA.

When the resilient modulus depends of the first invariant of stress (θ), it is verified that the best adjustment is obtained now for the granite in study, for any of the conditions of compaction, with r^2 varying from 0.9945 to 0.9981, and that the worst adjustment is obtained for the materials of VLA with r^2 varying between 0.4000 and 0.4600. The other two materials, limestone from A23 and from A6, present r^2 varying between 0.8914 and 0.9857.

6. CONCLUSIONS

Analysing the characterization results of the two materials, we may conclude that they are not plastic and, according to the Technical Guide for the Construction of Embankments and Subgrade Pavement [14], we may even consider that the fines are not sensible to the water, given the values of adsorption of the blue methylen obtained. We conclude, on the other hand, that it is a material with good capacity of resistance, average CBR values varying between 85 % and 90 %, 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.

In what says respect to the mechanical behaviour we verify, for values of optimum moisture content and 95% of maximum dry density, values of the resilient modulus variable between, approximately, 160 MPa and 450 MPa, to the limestone and between 90 MPa and 300 MPa to the granite. Considering the in situ conditions those values varies between 160 MPa and 600 MPa to the limestone and between 80 and 300 MPa to the granite, function of the load conditions. So, the values of resilient modulus for the limestone laboratory conditions is almost the same of the VLA limestone, which presents the higher values from the 4 that we compared.

We verify, on the other hand, in what says respect to the permanent deformation, that it varies between 0.4 % and 1.4 % for the limestone and between 1.2 % and 3.4 % for the granite.

In terms of the resilient modulus modelation, it was verified that the better simulation of the resilient behaviour of the two materials is obtained for the Pezo model [9], which relates the resilient modulus with the differential stress (q) and the confining stress (σ_3).

When compared the modelation of the mechanical behaviour with the one of other materials, it is verified that when the resilient modulus depends on the confining stress (σ_3), the adjustment found for the materials of VLA, for the limestone and for the granite studied in this work is not very good, since r^2 varied from 0.8898, for the limestone of A23 for conditions in situ, to 0.9600, for the specimen CT1 of VLA.

When the resilient modulus depends of the first invariant of stress (θ), it is verified that the best adjustment obtained is now for the granite in study, for any of the compaction conditions, with r^2 varying from 0.9945 to 0.9981 and that the worst adjustment it verified for the materials of VLA with r^2 varying between 0.4000 and 0.4600. The other two analysed materials, limestone from A23 and limestone from A6, present r^2 varying between 0.8914 and 0.9857.

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