Hydro-mechanical characterization of soil-rockfill mixtures

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ABSTRACT

In order to improve the rather insufficient knowledge about the hydro-mechanical behavior of soil-rockfill mixtures (SRM) applied in embankment dams, a testing program was undertaken in which several mixtures with different coarse fractions (CF) were tested (30%, 40%, 50% and 70%). It is an environmentally friendly material that takes advantage of the material excavated for spillways, cut-off trenches, outlet works and other structures that would go to deposit and can be reused. Recently, especially in road engineering, it has been giving particular attention to SRM by the fact that many embankments, constructed with such materials, have presented a poor performance and required, in many cases, the implementation of corrective measures. The same thing happened with some dams constructed with this type of material started to present substantial deformations. Meimoa dam and Beliche dam are examples of dams that have this type of material in their body and presented misbehavior .This study presents, for the first time, a systematic experimental program of this type material that exist in nature, involving isotropic and K0 compression tests, undrained triaxial tests and permeability tests for different coarse fractions, and aims improving the knowledge about the behavior of this material.

1. INTRODUCTION

The use of non-traditional materials such as soil–rockfill mixtures (SRM) in earthworks construction, for economic and environmental reasons, brings new challenges to dam engineering. Usually, this kind of material results from bulky rock extraction without explosives, and it can include some oversized particles (about 0.5 m or larger).

Soil–rockfill mixture material must comply with the following conditions (JAE, 1998): (i) the fraction retained in a ¾" (19 mm) sieve must be between 30% and 70%; (ii) the fraction passing through a No. 200 (0.074 mm) sieve must be between 12% and 40%; (iii) and the maximum particle dimension must be less than two-thirds of the layer thickness after compaction and not larger than 0.40 m. A significant fraction of oversized particles, sufficient to form a structure associated with a fine matrix, which plays an influential role, characterizes these materials.

Recently, soil–rockfill mixtures have gained some attention due to their anomalous deformation behaviour, resulting in the need to take corrective measures in many cases. Their grain size distribution, construction techniques, applied loads, and environmental conditions greatly affect their mechanical properties. So, a thorough and comprehensive experimental investigation is needed and is under development to identify the most important parameters related to construction conditions and quality control as well as short and long-term behaviour as a function of the rockfill lithological constitution and relative percentages of soil and rock present in the mixture. These kinds of studies will converge to establish correlations between the compaction characteristics and the hydro mechanical design parameters and to calibrate existing constitutive models or to develop a new constitutive model that integrates all the characteristics determined in the laboratory investigations.

This paper describes two dams built with these types of materials and with a defective behaviour in terms of deformation. Then, it presents the case study of this research – Odelouca dam. Finally, the paper presents the main results obtained in order to the hydro-mechanical characterization of soil-rockfill mixtures.

2. BEHAVIOUR OF DAMS CONSTITUTED BY SOIL–ROCKFILL MIXTURES

In recent years, soil–rockfill mixtures have been used in the construction of several dams in Portugal. As an example, the following briefly reports on the behaviour of Beliche and Meimoa dams.

Beliche dam is an embankment dam built between 1982 and 1985 in Algarve, in the South of Portugal, for irrigation and water supply purposes, with a total reservoir capacity of 47 hm^3 and a height of 55 m. Schist and greywacke with a considerable degree of weathering constitute the dam foundation. The core foundation is a rock substrate, which resulted from the removal of an alluvial layer of silty sand and gravel, with a maximum thickness of 10 m at the valley bottom. An impervious grout curtain represents the foundation treatment.

A central core and shells made of rockfill and soil–rockfill mixture essentially constitute the dam body. During the construction, in the inner part of the shells, that is, in the transition zones between the core and the outer zones of the shells, highly weathered schist and greywacke (a soil–rockfill mixture) were used (Figure 1). For the construction specification of the soil–rockfill mixture, Maranha and Veiga Pinto (1983) performed one-dimensional compression deformation tests with different dry density values, water contents, and percentages of coarse fraction. They intended to control the material's oedometric modulus in order to ensure that it was between 30 and 40 MPa, so that the contrast between the stiffness of the core and the shells materials would be low.

To control the structural performance of the dam, a complete monitoring system was installed. During the construction of the dam (Pardo, 2006), a very heavy rainfall occurred in the winter of 1984/85 and caused large settlements of the already built embankment. In 1987, during the first filling of the reservoir, the dam experienced relatively important settlements at the crest, with settlement rates of 15 to 18 mm/month, attributed to wetting-induced collapse and creep of the inner shell zones. At the end of filling, the crest settlements reached the value of 0.65 m and the loss of freeboard was 0.50 m. The average settlement rate at the crest decreased to 5 mm/month at the end of the first filling. The surface markers located in the upstream and downstream crest alignments, at the end of the first filling, registered maximum values of horizontal displacements, respectively, of 0.18 and 0.32 m, in the downstream direction. As a result, longitudinal cracking was detected at the crest. In 1996, the crest settlements reached a maximum value of 0.70 m, the internal horizontal displacements a maximum value of 0.55 m, and the horizontal displacements at the crest a maximum value of 0.40 m in the downstream direction.

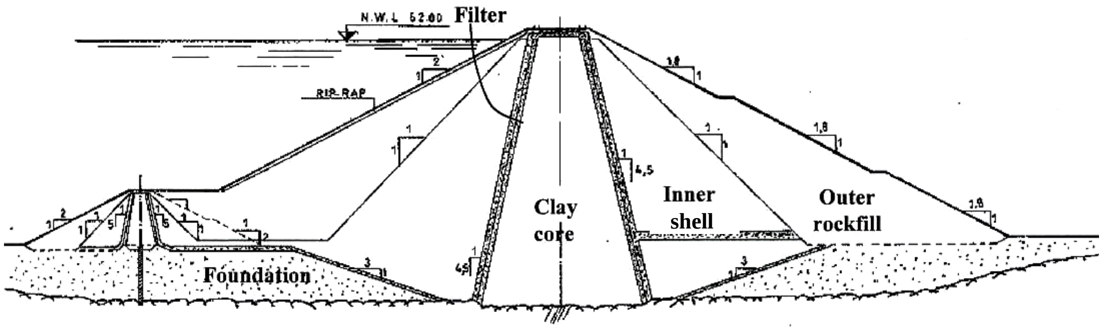


Figure 1. Cross-section of Beliche dam (Caldeira & Brito, 2014)

Meimoa dam is a zoned dam completed in 1985, near Castelo Branco in the central part of Portugal, for irrigation and water supply purposes, with a total reservoir capacity of 40.9 hm^3 and a height of 56 m. Clayey soils constitute the core and soil–rockfill mixtures (Figure 2) constitute the upstream and downstream shells. The dam foundation is composed of highly weathered schist and greywacke, after removal of a superficial 4 m thick layer before construction. An inclined grout curtain executed from the crest ensures the water-tightness of the foundation.

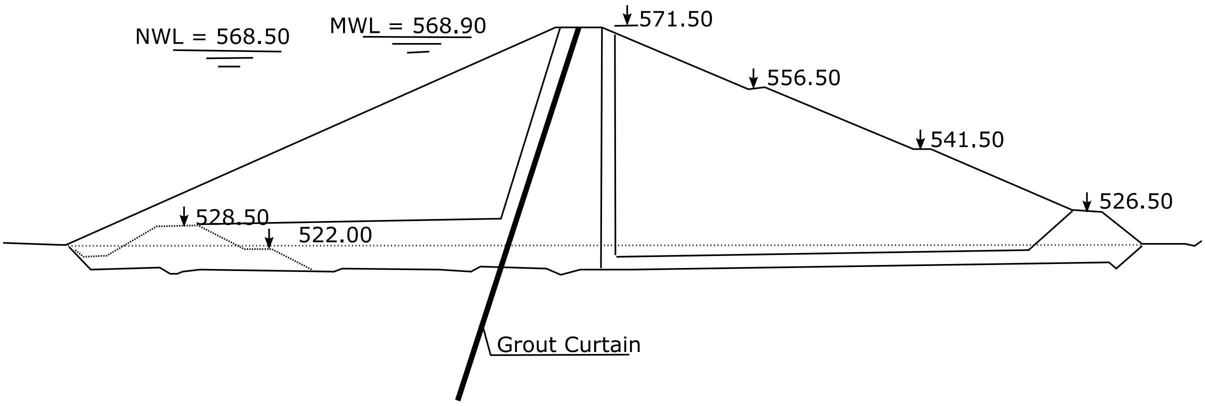


Figure 2. Cross-section of Meimoa dam (Caldeira & Brito, 2014)

During the first filling of the reservoir, visual inspections and the monitoring system allow to detect: a horizontal displacement, in the upstream direction, of the concrete beams of the downstream footway; longitudinal (in the upstream zone) and transversal cracking at the crest pavement due to differential settlements, most frequent near the abutments; visible settlements of the embankment in relation to the top protection blocks of the monitoring equipment; readings of water levels at piezometers installed at elevations higher than the reservoir water level; and large displacement rates at the surface markers and inclinometers.

The review of some dams built in Portugal with SRM showed that most of the anomalies emerged during the first filling of the dams. This shows that the SRM used in the construction of these dams are susceptible to collapse. This was probably due to the fact that during construction the material was compacted on the dry side of the compaction curve.

Another important aspect is that almost every SRM used in these dams was made of schist and greywacke with a considerable weathering degree. So the fine fraction present in the SRM was mostly clay and silt. This type of material is very susceptible to wetting. If the dams were compacted on the dry side of the compaction curve and then subject to wetting during the first filling, the material became less resistant and more deformable due to the loss of suction.

3. CASE STUDY: ODELOUCA DAM

The case-study of this research work is Odelouca Dam (see Figure 3). Therefore, all materials used in this study come from this dam, particularly from its shells which are composed of a soil-rockfill mixture. So, a brief description of the dam is now presented.

Figure 3. Odelouca Dam

Odelouca dam is a zoned embankment dam, with 76 m of height, located in Algarve, in the south of Portugal. The crest of the dam, with 11 m wide, is approximately 415 m long. Figure 4 and Figure 5 show, respectively, the dam cross-section, and the developed section.

The reservoir created by Odelouca dam encompasses a 7.8 km2 area, and has a 157 hm3 capacity at the maximum water level. Most of this volume is intended for water supply with a small part used to irrigate the downstream fields.

Odelouca dam is located in the Odelouca Small River, about 1 km upstream of the confluence of this small river with the Monchique Small River, both tributary rivers of the Arade River, at the elevation (35.00).

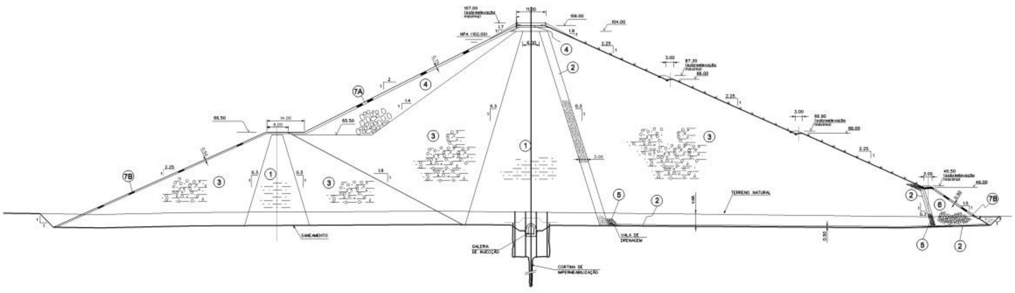


Figure 4. Odelouca Dam cross-section

The Odelouca Dam embankment materials include clayey soil, in the core, and weathered schist and greywacke, with a significant fraction of large sized particles (soil-rockfill mixture), in the shells. Figure 6 presents the grain sizes distribution curves for the materials used in the dam. The use of materials, extracted mainly from the reservoir area, minimizes the negative environmental impacts. The upstream slope incorporates the cofferdam creating a 14 m wide berm.

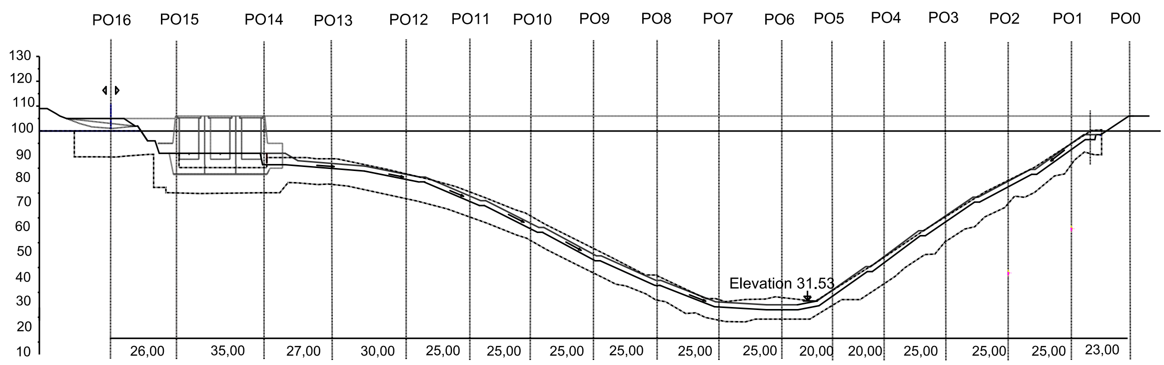


Figure 5. Odelouca' developed section

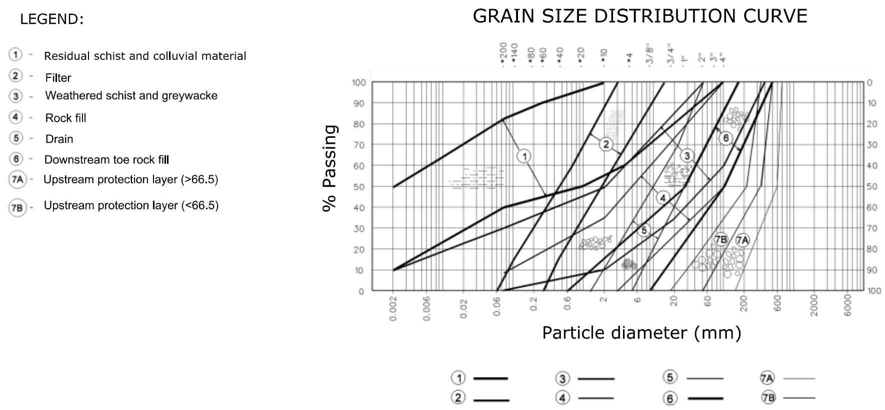


Figure 6. Grain-size distribution curves of Odelouca dam materials (Caldeira & Brito, 2014)

4. CHARACTERIZATION OF SOIL-ROCKFILL MIXTURES

In order to improve the rather insufficient knowledge about the mechanical behaviour of soil-rockfill mixtures, a testing program was undertaken, in which several mixtures with different coarse fractions (CF) were tested.

The soil-rockfill mixtures studied had the following coarse fractions: 30, 40, 50 and 70%. Nevertheless, materials with 0 (merely soil) and 100% (purely rockfill) of coarse fraction were also tested, in order to identify the threshold behaviours of the mixture.

The material passing No.200 (0.074mm) sieve has a plasticity index of 12.0% and a liquid limit of 37.3%. According with the Unified Soil Classification System, the fines of the SRM used in the studies is classified as CL – a lean clay with sand. A standard Proctor test was performed on the fine fraction (FF) and the optimum point obtained was: $\gamma_d^{\max} = 19.1$ kN/m^3 and $w_{opt} = 12.3\%$.

The tests performed in order to characterize the coarse fraction (CF) are very similar to the ones performed with rockfill material. The rockfill behaviour involves three aspects: its structure and lithological composition, its physical and mechanical properties, and durability. The laboratory tests carried out aimed to determine the physical and mechanical properties, and to evaluate the durability of this material.

The average values obtained for the density, the dry density, the saturated density, the porosity and the absorption for the >3/4" material were, respectively: 2820 kg/m^3, 2510 kg/m^3, 2383 kg/m^3, 12.6% and 5.3%.

The degradability test consists in determining the evolution of about 2 kg of rock material, with a selected grain size distribution, due to four cycles of wetting and drying, during 8 and 16 hours, respectively. The test result is expressed by the degradability coefficient, DG, that is given by the ratio of effective diameters (corresponding to the 10% passing material) of the initial sample and the sample after application of such cycles. In general, if DG> 20 the material is very degradable, if 5 <DG <20 the material is degradable and if DG <5 the degradability of the material is small. The test follows the French Standard NF P94-067 (1992).

The fragmentability test is run on a grain size distribution identical to the one of the degradability test. The sample is compressed within the CBR mould by application of 100 strokes of the Normal Proctor test pestle. The coefficient of fragmentability (FR) is set identically to the degradability coefficient, i.e. the ratio between the effective diameters before and after compression. If FR <7 the rock is considered with small fragmentability, but if FR> 7 the rock is considered fragmentable. The test follows the French Standard NF P94-066 (1992).

The result obtained for the degradability coefficient was equal to 1.1 so the degradability of the material is small and also the fragmentability because the FR is equal to 2 (less than 7).

The Los Angeles test is intended to determine the wear loss (in percentage) of aggregates with a particle size selected due to abrasive filler consisting of steel balls. The test follows the Portuguese Standard NP EN 1097-2. The results prove that this material show great sensitivity to abrasion because the wear value obtained was very high: 46%.

A study performed in LNEC (Veiga Pinto and Prates, 1997) attributes to the three lithologic types of rockfill (greywacke, carbonated and granite), three different strength classes according with the values of porosity, dry density of the coarse particles, one-dimensional compression strength, point load test parameter (PLS), crushing strength for a theoretical average particle size of 50 mm, percentage of wear by the Los Angeles machine (LA), durability test parameter and fragmentability index (FR). Table 1 presents such classification for greywacke rockfills, considered closer to the samples tested.

Given the values obtained in terms of porosity, dry density, one-dimensional compressive strength, resistance to crushing, Los Angeles wear and wear rate in wet, this material is classified by Class 3 (according to Table 1), which corresponds to a material of low mechanical properties, typically classified as soil-rockfill mixtures.

Table 1. Classification for greywacke rockfill (Veiga Pinto & Prates, 1997)

Parameter	Class 1	Class 2	Class 3
n (%)	1-3	3-6	6-10
γ_{dg} (kN/m^3)	26.5-27.5	25.5-26.5	24.0-25.5
σ_c (MPa)	130-170	90-130	50-90
PLS (MPa)	9-12	6-9	2-6
P_a50 (kN)	14-18	8-14	4-8
LA (%)	15-30	30-40	40-50
Id_7 (%)	0.5-1.0	1.0-1.5	1.5-4.0
Expansibility ($\Delta l / l$ 10^{-4})	2-6	6-10	10-14
FR	1-3	3-5	5-7

4.1.1 Isotropic compression tests

In order to determine the slope of the normal consolidation curve (λ) and the slope of the swelling curve (k) of the different mixtures, isotropic compression tests were performed with unloading and reloading cycles tests.

Figure 7 presents the results for all the SRM tested in terms of specific volume versus the mean stress logarithm. The SRM presented the same type of curves and the values obtained for the normal consolidation line and for the swelling curve (presented in Table 2) are very similar. The two extreme materials (0% and 100%CF) presented the same type of curves, but larger values for the slopes of the consolidation and swelling lines. Also the SRM present almost similar yield stress (between 270 and 310 kPa). The material that has the lowest yield stress is the 100%CF. This is probably due to the particles breakage.

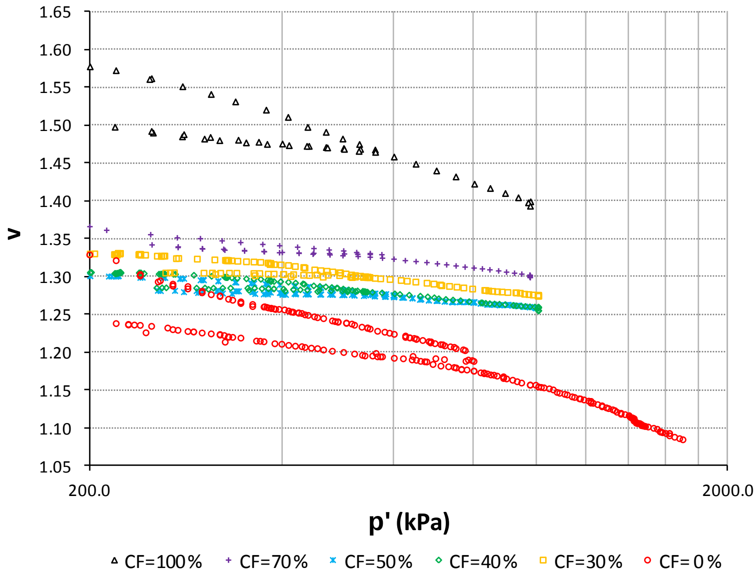


Figure 7. Isotropic compression tests results

The variation of N and the slope of the normal consolidation line with the coarse fraction is presented in Figure 8. Based on those results, it can be deduced the following expressions:

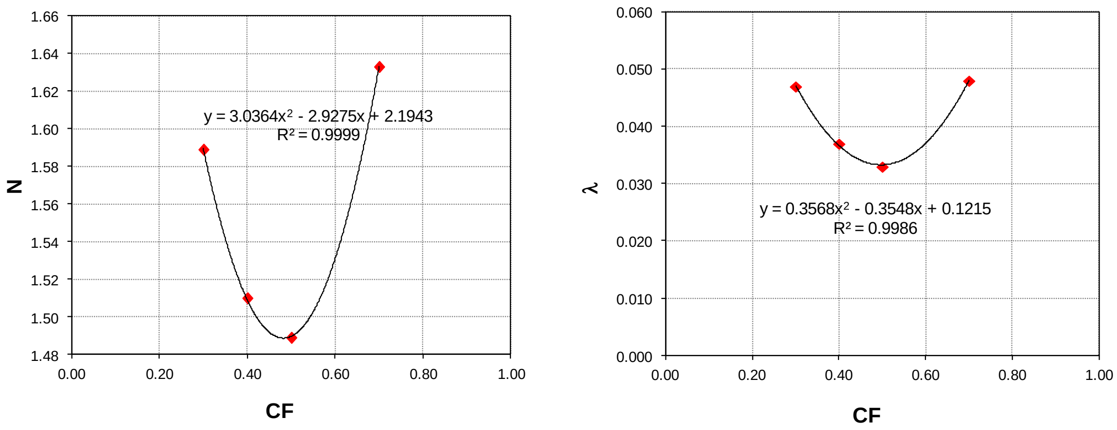
$$N = 3.0364CF^2 - 2.9275CF + 2.1943 \quad \left(R^2 = 0.9999\right) \tag{1}$$

$$\lambda = 0.3568CF^2 - 0.3548CF + 0.1215 \quad \left(R^2 = 0.9986\right) \tag{2}$$

Table 2. Parameters of the normal consolidation line and of the swelling curve and the yield stress for all the SRM tested

Sample	N	λ	k	p' (kPa)
0% CF	2.166	0.145	0.022	340
30% CF	1.589	0.047	0.008	280
40% CF	1.510	0.037	0.006	310
50% CF	1.489	0.033	0.008	280
70% CF	1.633	0.048	0.006	270
100% CF	2.254	0.124	0.038	250

Table 2 shows the maximum, minimum, average and the coefficient of variation values for the SRM samples. These results show that the mechanical behaviour of the SRM is very similar, since, although there were some differences between the maximum and minimum values of N, λ and k, the coefficient of variation is very small (less than 0.2) showing that the dispersion of the results is low.



a) b)

Figure 8. a) *N versus* coarse fraction, and b) *λ versus* coarse fraction

Table 3. Representative values considering only the *SRM*

	N	λ	k	p' (kPa)
Maximum	1.633	0.048	0.008	310
Minimum	1.489	0.033	0.006	270
Average	1.555	0.041	0.007	285
COV	0.04	0.18	0.16	0.06

4.1.2 Undrained Triaxial Compression Tests

The samples dimensions are about 230 mm in diameter and 450 mm in height. Each sample is obtained by the vibrating compaction of about eight layers. Each sample was composed by three specimens that were consolidated isotropically to three values of effective stress: 200, 400 and 800 kPa. The sample 40% CF have a fourth specimen that was consolidated isotropically to 1000 kPa, because it was considered relevant the material behaviour to higher pressures.

Figure 9 shows the variation of the void index (e) after preparation with the coarse fraction present in the mixture (CF).

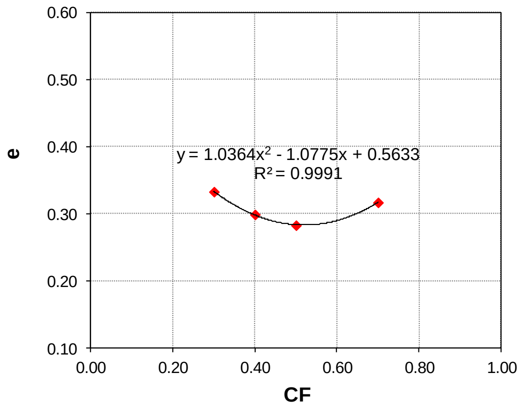
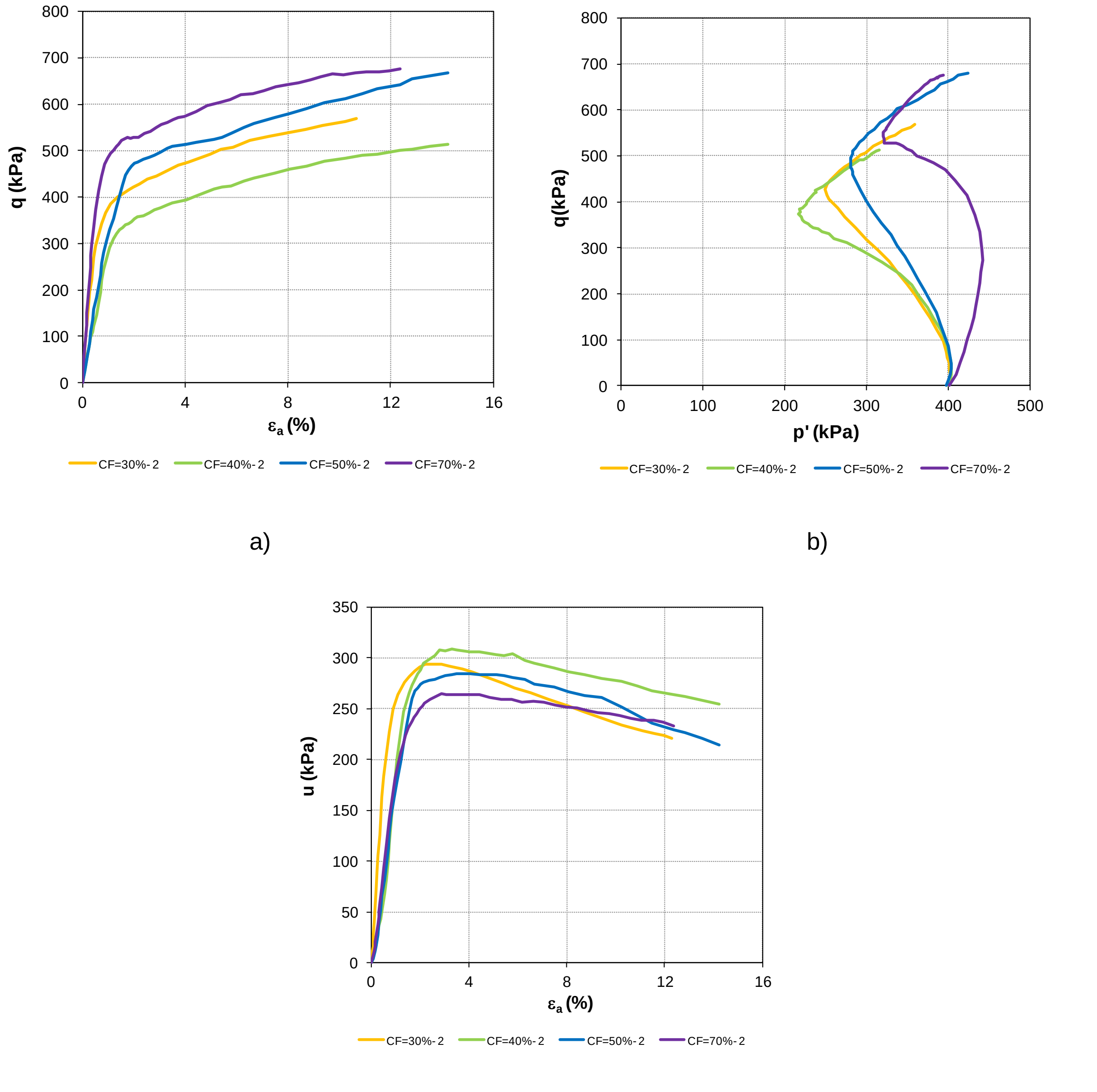


Figure 9. Void ratio average *versus* coarse fraction

The relation between the void ratio in average after consolidation and the coarse fraction gives:

$$e = 1.0364CF^2 - 1.0775CF + 0.5633 \ (R^2 = 0.9991) \qquad (3)$$

As an example Figure 10 presents the results in terms of deviatoric stress and pore pressure as a function of the axial strain and the effective stress paths for all the mixtures tested for samples isotropically consolidated to 400.



a) b) c)

Figure 10. Undrained triaxial compression test for the mixtures isotropically consolidated to 400 kPa: a) deviatoric stress invariant *versus* axial strain, b) effective stress paths and c) pore pressure *versus* axial strain

All samples present the same type of behaviour: the effective stress paths show an inversion in direction and the samples exhibits a gradual transition in rigidity when going from elastic to elastoplastic behaviour. The SRM consolidated at 200 kPa of effective stress show a slightly larger strength with the increasing of CF. This is less perceptive with the increase of the consolidation effective stress since every SRM consolidated to 400 and to 800 kPa presented the same strength. The only exception was the 30% CF specimen that presented lower strength. This is probably due to the fact that this sample presents a lower permeability value, and so it dissipates less pore pressure. Also the samples consolidated to higher values (800 kPa) presented the effective stress paths very similar in the beginning of the load, showing an almost indifferent behaviour to the coarser fraction present in the mixture, and only diverging where the inversion in direction occurs. In general the 70%CF sample presents slightly larger strength because is the sample that generated less pore pressure (since it is the sample that presents the higher permeability coefficient).

Several studies had concluded that the rockfill failure envelope is curve (Charles and Watts, 1980; Veiga Pinto, 1983; Charles, 1990-a, 1990-b, among others). So, in order to verified if the SRM also present this type of envelope the (p', q) values at the end of the tests, for all the mixtures tested, are presented in Figure 11. It was assumed that all the samples reached the critical state in order to be able to plot these values in this figure.

In the same figure the expression that showed a better adjustment to the results for all the SRM is displayed considering that in (p',q) space the best fitting expression for this type of material is potential. The curved behaviour becomes more important with the increase of the CF value.

The good fitting of the results in (p',q) space, for the mixtures tested, shows that this type of material also presents the same typical behaviour of rockfill, this is, the failure envelope is curved and this phenomenon is more pronounced with the increasing value of CF.

5. CONCLUSIONS

The results of the isotropic consolidation tests show that the mechanical behaviour of the SRM is very similar, since, although there were some differences between the maximum and minimum values of N, λ and p', the coefficient of variation obtained was very small (less than 0.2) showing that the dispersion of the results is low.

All samples presented the same qualitative type of behaviour under undrained triaxial conditions: the effective stress path presented an inversion in direction and the samples exhibit a gradual reduction in stifness when evolving from elastic to elastoplastic behaviour. Also the samples consolidated at higher stress values presented very similar effective stress paths, showing an almost indifferent behaviour to the coarser fraction present in the mixture.

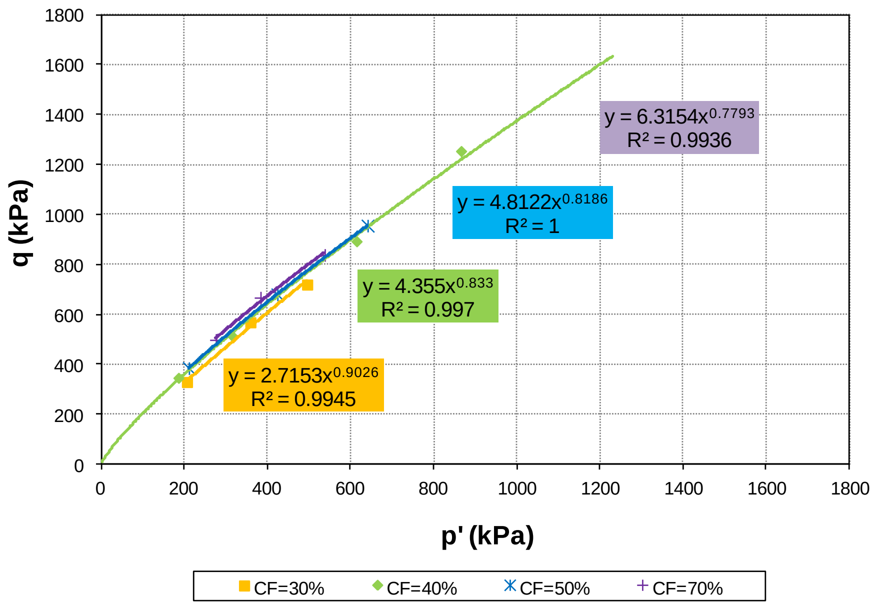


Figure 11. Undrained triaxial compression test for the mixtures isotropically consolidated to 400 kPa: a) deviatoric stress invariant *versus* axial strain, b) effective stress paths and c) pore pressure *versus* axial strain

The good fitting of the results in (p',q) space, for the mixtures tested, shows that this type of material also present the same typical behaviour of rockfill, this is, the failure envelope is curved and this phenomenon is more pronounced with the increasing value of CF.

6. ACKNOWLEDGEMENTS

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