Article title

The Use of Soil–Rock Mixtures in Dam Construction

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ABSTRACT

The employment of non-traditional materials such as soil–rock mixtures, for economic and environmental reasons, in the construction of earthworks poses some new challenges for compaction techniques and their control as well as for the determination of the characteristics of the embankment that result from the compaction method. Those characteristics experience important changes according to the relative percentages of the existing fractions. Usually, this kind of material results from bulky rock extraction without explosives, and it can include some large size particles (greater than 0.5 m). In addition, the measured deformations associated with these materials have been larger than expected. For the execution control of the soil–rock mixtures from the Odelouca dam borrow areas, a series of vibratory and standard compaction tests was performed to estimate reference values for the maximum dry density and optimum water content of these materials and a new methodology was proposed. Odelouca dam is a zoned embankment dam, 76 m high, with clayey soil in the core and weathered schist with a significant fraction of oversized particles in the shells. This paper presents the results of the compaction control as well as the dam performance during the construction and first filling phases as a validation of implemented construction procedures.

KEYWORDS: Soil–rock mixtures, Construction control, Compaction, Weathered schist, Dam performance

Introduction

The use of non-traditional materials such as soil–rock mixtures 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–rock 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–rock 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 rock fill 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 reviews the studies already done on the construction quality control of soil–rock mixtures, aiming to improve the performance of the dams, as well as its application to a new dam – Odelouca dam. Finally, as a validation of the compaction control used, the paper presents a set of monitoring data obtained during the construction and first filling phases of Odelouca dam.

Behaviour of dams constituted by soil–rock mixtures

In recent years, soil–rock 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 rock fill and soil–rock 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–rock mixture) were used (Figure 1). For the construction specification of the soil–rock 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.

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–rock 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.

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.

Construction quality control of soil–rock mixtures

The construction quality control of dams constituted by soil–rock mixtures is still a subject that needs investigation, considering that it is necessary to extrapolate current test results reached by truncated gradation for the construction conditions of the embankments.

On the one hand, these materials exhibit a percentage of rock fill material, controlled during placement by only one parameter – the void ratio, on the other hand, they exhibit a percentage of soil, characterized by two parameters – the dry unit weight and water content. There are doubts about how to treat soil–rock mixtures. Their behaviour depends on the relative percentages of their constituents, becoming close to a soil if the fine fraction is large and the coarser material is scattered in it, or close to a rock fill if the coarser materials touch each other and the fines occupy the voids left by them. Therefore, it is necessary to consider an additional parameter: the percentage of coarser material (the percentage of material retained in the ¾" ASTM series sieve).

In support of this hypothesis, USCOLD (1998) recognizes that the inclusion of rock particles in otherwise fine-grained soils can have a significant influence on the engineering properties of the material, depending, among others things, upon the relative percentages of soil and rock present in the mixture.

For the quality control of soil–rock mixtures in embankment construction, the usual practice is to apply corrective expressions to the standard Proctor test results (optimum water content and maximum dry density), in order to take into account the influence of the coarser material on the reference properties. Previous studies carried out on soil–rock mixtures showed a strong dependence between corrective expressions and rock fragment strength, as well as the degree of weathering.

Using as a reference the construction of Odelouca dam, with shells constituted by weathered schist and greywacke with a significant fraction of oversized particles, the authors carried out a laboratory test programme for the deduction of specific corrective expressions for application in the embankment quality control.

Previous studies of soil–rock mixtures compaction

Embankment construction using soil–rock mixtures adopts mixed procedures: construction techniques employed in the rock fill construction (such as crash-roller spreading and vibrator roller compaction) and construction quality control techniques used in soils embankments, in conjunction with some properly deduced correction equations. Additionally, in order to reduce the energy taken in by the larger particles, the maximum dimension is limited to two-thirds of the layer thickness (Winter and Suhardi, 1993).

Extraction, transportation, placement, spreading, and compaction of these materials induce, in general, a grain size evolution expressed by an increase in fines. In this way, the materials resulting from this kind of process are broadly graded and present high compactness and good mechanical strength. However, the construction processes, the atmospheric conditions (such as wetting and drying cycles), and the imposed mechanical actions can significantly change their mechanical and physical properties.

Some laboratory (one-dimensional strain compression) and in situ (plate load) tests were performed to assess the mechanical characteristics of soil–rock mixtures in the earthworks of the Via Longitudinal do Algarve highway. The results of tests and studies (JAE/LNEC, 1994) made it possible to verify that both coarser and finer fractions influence the stress–strain behaviour of the soil–rock mixture. Its behaviour was dependent on the material characteristics (such as type, heterogeneity, evolutionary nature, susceptibility to wetting collapse, expansibility, and creep) and construction conditions (spreading technique, amount of water present, atmospheric conditions, and compaction methodology).

Winter and Suhardi recommend the adoption of different procedures for compaction control according to the coarse fraction present in the mixture. For materials in which less than 45 to 50% of particles are larger than 20 mm, the matrix properties (i.e. the fine material in the mixture) will control those of the embankment structure, so its construction control can be carried out using truncated grain-size distributions (by elimination of the coarser particles). If the proportion of blocks rises significantly above 45 to 50% of the total mass, then physical and mechanical characteristics based on truncated grain-size distributions may not be representative. Large-scale testing, including in situ testing, may be necessary.

In order to take into account the influence of the coarse fraction in the quality control, Houston and Walsh (1993) report on different laboratory compaction testing methodologies:

- *Method 1* (integral sample) – performed in large moulds, using the integral field material, for evaluation of the maximum dry density and optimum water content;
- *Method 2* (scalp-and-replace) – performed in moulds of smaller dimensions, removing the material larger than ¾" (19 mm) and replacing it with an equal weight of No. 4 to ¾" sieve material;
- *Method 3* (elimination and correction) – performed using only the material passing through a ¾" sieve in a 6" (15.24 cm) mould; material larger than ¾" is discarded;
- *Method 4* (elimination and correction) – performed using only the fraction passing through the No. 4 sieve and correcting the results obtained taking into account the percentage of coarse fraction in the field mixtures (using ASTM D4718, Standard T224 from ASSHTO, or Standard 5515-89 from USBR).

Method 1 is a very expensive and time-consuming technique and is therefore rarely used. Methods 2 and 4 are frequently employed for materials containing 10 to 60% coarser particles. Method 3, valid when the material larger than ¾" represents less than 10% of the mixture by weight (ASTM D698), is only suitable for mixtures with reduced percentages of coarser particles and which the behaviour is clearly controlled by the finer matrix.

Table 1 presents the correction equations most frequently used for the determination of the maximum dry density of the integral material.

Studies carried out by LNEC (1987) suggest the use of the scalp-and-replace method. For the correction of the values obtained in the compaction tests (w_{opt}^{F} and $\gamma_{d\,\max}^{F}$) to calculate the optimum water content and the maximum dry density of the mixture as a function of the percentage of coarser particles (retained in No. 4 or ¾" sieves of the ASTM series), the following expressions are proposed:

$$\gamma_{d\,máx}^{T} = \frac{100}{\dfrac{P_C}{\gamma_d^C} + \dfrac{P_F}{\eta \gamma_{d\,máx}^{F}}} \quad (1)$$

$$w_{opt}^{T} = \frac{P_F w_F + P_C w_C}{100} \quad (2)$$

where $\gamma_{d\,máx}^{T}$ and $w_{\mathrm{opt}}^{\mathrm{T}}$ are the maximum dry density and the water content of the integral material, γ_d^C and w_C are the dry unit weight and water content of the coarse fraction, P_F and w_F are the weight percentage and water content of the fine fraction (usually taken as w_{opt}^{F} obtained in standard compaction tests), P_C is the weight percentage of the coarse fraction, and η is a correction factor, defined as:

$$\eta = -5\times10^{-5} P_C{}^2 + 0.0013 P_C + 0.9958 \tag{3}$$

In general, these formulas give good results in terms of maximum dry density and not such good ones in terms of the optimum water content – the most important compaction parameter for these materials.

In 1994, Torrey and Donaghe introduced a new method, calibrated from data already published for soil–rock mixtures at that time. They performed standard effort compaction tests in different moulds and with materials composed of gravel, sand, and non-plastic silts or high plasticity clays. For the analysis of results, they defined two additional quantities: the density interference coefficient, I_c, and the optimum water content factor, F_{opt}, expressed as:

$$I_c = \frac{100 F_F}{P_C G_M} \tag{4}$$

$$F_{opt} = \frac{100 w_{opt}^F}{P_C w_{opt}^T} \tag{5}$$

where F_F is the fraction density factor, given by $F_F = \gamma_d^F / \gamma_{d\,máx}^F$, γ_d^F is the dry unit weight of the fine fraction in the mixture, and G_M is the soil particle density of the coarse fraction.

To calculate F_F, the authors applied the following equation:

$$F_F = \frac{\gamma_{d\,máx}^T G_M \gamma_w P_F}{100\gamma_{d\,máx}^F G_M \gamma_w - \gamma_{d\,máx}^T \gamma_{d\,máx}^F P_C} \tag{6}$$

The values of I_c and F_{opt} of each test were represented as a function of the coarse content, P_C. This representation made it possible to verify that the proposed parameters were independent of the types of fines present and to suggest correlations between these parameters and the coarse fraction, P_C.

Studies of the construction of Odelouca dam

Odelouca dam is a zoned embankment dam, 76 m high, built in Algarve in the south of Portugal. The crest of the dam, 11 m wide, is about 415 m long (Figure 3).

The embankment materials are clayey soil in the core and weathered schist and greywacke, with a significant fraction of oversized particles, in the shells. Figure 4 presents the grain-size distribution curves of the materials used in the construction of Odelouca dam.

The selected shell materials should meet the following requirements: a maximum fines content of 30%; between 10 and 50%, 18 and 62%, and 33 and 93% of particles passing through the No. 10, No. 4, and ¾” sieves, respectively; and average and maximum particles diameters between 2 and 50 and between 50 and 400 mm, respectively.

Compaction parameters based on vibratory tests

During the trial embankment construction (in 2004), LNEC proposed a laboratory-testing programme for setting up corrective expressions to extrapolate the properties obtained by truncated grading to the actual grading size, according to Torrey and Donaghe’s methodology (1994). The expressions derived by these authors for gravel, sand, silt, and clay mixtures would not be directly applicable due to the different nature of the shell material (schist). The material used in these tests came from the borrow areas used in the cofferdam construction.

In view of the compaction equipment (vibrating roller) used in the dam, laboratory vibratory compaction tests of the integral material were carried out to obtain the maximum dry density and the corresponding optimum water content, and the values obtained were then compared with the standard Proctor reference values found with truncated grading materials.

The tests used samples from different lots collected after material extraction and homogenization for the construction of the trial embankments. The vibratory tests performed were based on suitably adapted specifications from Part 4 of BS1377 (1990). This standard uses the following equipment: a CBR mould, an electric vibrating hammer, a steel tamper for attachment to the vibrating hammer, a device enabling sample depth measurement, and a stop-clock.

The cylindrical mould (310 mm inside diameter, 300 mm height, sectioned longitudinally, connected with rigidity ribs, and fixed to a metal base – Figure 5) is easily removable and transportable. The material is compacted with a vibratory rammer coupled to a steel plate. The gap between the mould and the plate is about 10 mm. The total force exerted, including the rammer and plate weights, reaches 2,954 N. In view of the mould diameter (300 mm), the maximum dimension of the mixture particles was limited to 2”, and tests were performed (Brito, 2006) with varying percentages of the coarse fraction in the mixture.

During compaction, the following was observed:

- for high values of the water content and high vibration times, reflux of fine materials occurred systematically;

- for the higher water content tested (13.8%), after about 2 minutes of vibration, some swelling was detected, possibly due to material segregation, with a consequent reduction of its dry density;
- for the remainder of the tests, regardless of water content, the maximum dry density was reached after 9 minutes of vibration.

To validate the use of vibration tests in the compaction study, the results obtained by this procedure were compared with those of the conventional Proctor tests, performed with the material passing through the ¾" sieve (Figure 6). Analysis of Figure 6 reveals that:

- on the dry side of the compaction curve and for a water content larger than $w_{opt+}1\%$, it is not possible to achieve, by the vibrating procedure, dry densities comparable to those obtained by the standard procedure, regardless of the vibration time;
- the values of the maximum dry density and the optimum water content obtained by vibration after 9 minutes and by standard compaction are practically coincident, so one may conclude that, at this instant, the amounts of energy applied in the two compaction tests are equivalent;
- the curve obtained by standard compaction presents a far more flattened shape than the one obtained after 9 minutes of vibration, so the identification of the optimum point is facilitated by the vibration procedure.

Then, vibration compaction tests with different percentages of coarse fraction (between 30 and 67.7%) were performed using the available lots. Figure 7 represents the compaction curves obtained after 9 minutes of vibration with different percentages of coarse fraction. The optimum points, considered in the interpretation of the testing that follows, are also marked.

Compaction parameters based on standard Proctor tests

For comparison, conventional standard compaction Proctor tests were performed in a large-scale compactor (Toni-tecnik) using different coarse fractions in a mould of appropriate dimensions (Figure 8). Naturally, it was necessary to adapt the standard procedures of ASTM D698 due to the scope of the equipment and samples. In order to have the same effort as the standard compaction test, the compaction was carried out by the application of 55 blows per layer, in three layers.

Figure 9 presents the compaction curves obtained as well as the reference optimum point found with soil constituted only by the finer fraction.

It can be seen that the maximum dry density values are all between 20.2 and 20.3 kN/m^3 and do not vary with the coarse fraction. With regard to the water content, larger variations occur, with the highest value being recorded for the mixture with the highest percentage of fines, since the fine fraction can absorb a larger amount of water than the coarse fraction.

On comparing Figures 7 and 9 it can be concluded that the vibration compaction method is more efficient for these materials than the standard method, as it causes a larger reduction of the voids in the mixture, probably by changing the arrangements of the coarser particles.

Corrective equations

Torrey and Donaghe's approach (1994) was adopted in the analysis of test results. The values of the interference coefficient of the dry density, I_c, and the corrective factor of the optimum water content, F_{opt}, have been evaluated.

A higher correlation of the data from the tests was found when a bi-logarithmic relationship was adopted between I_c and F_{opt} and the coarse fraction, P_C, as follows, for vibratory tests:

$$\log I_c = 1.7193 - 1.0802 \log P_C(\%) \quad (R^2 = 0.9929) \tag{7}$$

$$\log F_{opt} = 1.873 - 0.845 \log P_C(\%) \quad (R^2 = 0.9769) \tag{8}$$

and, as follows, for standard Proctor tests:

$$\log I_C = 1.9067 - 1.1909 \log P_C \quad (R^2 = 0.9983) \tag{9}$$

$$\log F_{opt} = 1.8535 - 0.8291 \log P_G \quad (R^2 = 0.9983) \tag{10}$$

To improve the correlation between the experimental results and the interpolation expression of the corrective factor of the optimum water content, a new methodology was tested. An almost constant water content of the coarse fraction, equal to about 4.7%, was determined. Therefore, based on the optimum water content of the integral material, the authors derived the following expressions for the corresponding water content of the fine fraction, respectively, from the vibratory and standard compaction tests:

$$w_F = 319.17 P_G{}^3 - 406.84 P_G{}^2 + 177.15 P_G - 11.369 \quad (R^2 = 0.9809) \tag{11}$$

$$w_F = 26.071 P_G^2 - 12.793 P_G + 14.412 \quad (R^2 = 0.9801) \tag{12}$$

Having found the corrective equations for this material, and given the value of I_c, it is very simple to calculate the value of the maximum dry density of the integral material using the following expression:

$$\gamma_{d\,máx}^{T} = \frac{100 I_c P_G \gamma_{d\,máx}^{F} G_M \gamma_w}{I_c \gamma_{d\,máx}^{F} P_G{}^2 + \gamma_w P_F} \quad (13)$$

and the value of the optimum water content of the total material, given F_{opt}, by:

$$w_{opt}^{T} = \frac{100 w_{opt}^{F}}{P_G F_{opt}} \quad (14)$$

or given w_F, by Equation (2).

In the sense of validating the tests' application conditions, Figure 10 shows the fraction density factor and the maximum dry unit weight of the total material versus the percentage of coarse fraction obtained using the original Torrey and Donaghe equation and Equations (1), (7), and (9).

It can be concluded from the figures that Equations (7) and (9) present fraction density factor values higher than 100% for coarse fraction percentages below 20% in the vibratory compaction tests and below 35% in the standard compaction tests. These density factor values do not seem to have any real counterpart. For these coarse fraction percentages, the maximum dry density of the total material is practically equal to the dry density of the fine fraction corrected for the presence of coarse particles. In the remaining domain, the curves have a very regular path and exceed the Torrey and Donaghe expression. In terms of the maximum dry density of the integral material, the values deduced in this way are always larger than the maximum dry density of the fine fraction obtained by the standard Proctor test. These results are the consequence of the presence of coarser particles with higher density in the interior of the fine matrix.

Figure 11 presents the optimum water content of integral material as a function of the coarse fraction based on Torrey and Donaghe's expressions and Equations (8) and (10) to (12). The results calculated by Equations (10) to (12) show that the optimum water content of the total material always decreases monotonically with the coarse fraction percentage, and the values obtained for the Odelouca mixtures are always larger than those deduced by Torrey and Donaghe. For higher percentages of the coarse fraction, the curves tend asymptotically to water contents of 5.9% (Torrey and Donaghe's expression), 8.5% (Equation (10)), and 8.8% (Equation (8)), well above those obtained for the coarse fraction (about 4.7% on average). Thus, it can be suggested that the field of application of these expressions is limited for coarse fraction percentages below 10% and above 70%.

The alternative expression (11) presents a very different path from the remaining ones, especially for extreme coarse fraction percentages (reduced or very high). Compared with the previous expressions, it will have some meaning for coarse fractions from 25% to about 65%, but does not appear to be believable for higher percentages, where the curve first ascends and then descends quickly on the opposite side until it reaches the water content of the coarse fraction. The alternative expression (12) has a similar course to expressions (8) and (10) until coarse fraction percentages of about 65%, when it starts to descend to the water content of the coarse fraction. Thus, the field of application of this expression is limited only when the coarse fraction is below 10%, but it can be applied in the remaining domain.

Construction of Odelouca dam

Figure 12 presents the grain-size distribution of the shell materials of the dam tested during construction, as well as the granulometric range defined in the design. The construction quality control of the compacted layers applied Hilf's method (Hilf, 1959; ASTM D5080) to the fraction of the soil–rock mixture passing through the No. 4 sieve. Based on Hilf's results ($w_{opt}^{F}, \gamma_{d\,máx}^{F}$), the optimum water content and the maximum dry density of the integral material were obtained ($w_{opt}^{T}, \gamma_{d\,máx}^{T}$) by the application of the deduced corrective formulas.

During the construction of the main dam, between 2008 and 2009, the material characteristics changed to some extent. The equations derived in 2004 did not seem to reflect the current conditions of the embankment. Therefore, additional material was collected from the borrow areas, further vibratory compaction tests were carried out, and the following new corrective equations were determined and used at elevations above 81 m.

$$\log I_c = 1.7371 - 1.0693 \log P_C(\%) \qquad (R^2 = 0.9996) \tag{15}$$

$$\log F_{opt} = 2.3848 - 1.5805 \log P_C(\%) \qquad (R^2 = 1) \tag{16}$$

Figures 13 and 14 represent, in histogram form, the results obtained during the compaction control of the different layers of the upstream and downstream shells of Odelouca dam by the application of Hilf's method and the deduced corrective expressions for soil-rock mixtures. These results are presented in terms of the degree of compaction and the difference between the field water content and the corresponding optimum water content. They show that the materials were compacted on the wet side of the compaction curve for both shells and that a good compaction was achieved, with lower characteristic values of degree of compaction and difference in water content relative to the optimum water content of 98 and –0.4% for the upstream shell and 96 and 0.4% for the downstream shell, respectively.

Figure 15 represents the internal vertical displacements registered during construction by a settlement gauge located in the central (higher) part of the dam. The accumulated settlement during construction attained a maximum value of around 1100 mm.

Examining the incremental settlements of the different layers between the measuring points (Figure 15b), the major contribution is from the layers between elevations of 75.00 and 78.00 m (280 mm) and between elevations of 66.00 and 69.00 m (235 mm). Also apparent were the different values of stiffness of the layers in the central third of the dam – a zone usually associated with larger settlements, but normally with similar deformability characteristics. The use of inadequate corrective equations caused this stiffness contrast. The adoption of the new equations solved the problem, as Figure 15 (b) confirms.

From the comparison of the construction settlements of Odelouca and Beliche dams it can be concluded that the maximum values are of the same order of magnitude, taking into account the different heights of the dams. Nevertheless, the progression of the settlements is more regular at Odelouca. After heavy rains, the settlement only increased in the exposed layer and did not affect the remaining embankment. As the compaction specifications adopted water contents larger than the optimum one, the displacements during the first filling of the reservoir were small and were less than those observed in Beliche and Meimoa dams, as Figure 16 shows, proving the very good performance of the dam.

3. CONCLUSIONS

Using the shells of Odelouca dam for reference, this paper presents the results of vibratory and standard compaction tests carried out for the deduction of corrective expressions for application in the embankment quality control of soil–rock mixtures. The vibratory compaction tests, introduced for the first time in the compaction control of embankment dams, seem to provide a better reproduction of the field compaction characteristics obtained with the vibratory rollers usually used for soil–rock mixtures.

Torrey and Donaghe’s methodology (1994), which is the only one that properly takes into account the water content of the integral material, was adopted in the analysis of test results. The test results allow the establishment of correlation equations for the determination of the maximum dry density and the optimum water content of the integral material and of the optimum water content of the fine fraction as a function of the coarse fraction percentage. The comparison and representation of those equations allows to draw conclusions about their range of application. Most of those equations can be applied to all soil-rock mixtures to control their quality based on the current practice of embankment construction using traditional Proctor tests. This constitutes a significant advantage, as no new procedures, besides the coarse fraction determination, are required for the quality control of these materials.

The quality control of the construction of Odelouca dam used Hilf's method and these corrective expressions, conveniently adapted to the extracted materials, evidencing the dependence of the corrective equations on the material. The shell materials were compacted on the wet side of the compaction curves.

For validation of the construction specification and quality control procedures implemented in this dam, the settlements during its construction and first filling phases were analysed and compared with the monitoring results from Beliche and Meimoa dams.

The settlements which occurred during the construction phase of Odelouca dam presented the same order of magnitude as those of Beliche dam, due to the wet compaction. Nevertheless, during the first filling, the monitoring system allows smaller displacements to be observed in Odelouca dam compared with the other dams, evidencing an adequate performance with no wetting collapse or larger deformability of the shell materials.

References

American Association of State and Highway Transportation Officials – AASHTO – T224-10 (2010). Standard method of test for correction for coarse particles in the soil compaction test.

American National Standard – ASTM – D4718-87 (2007). Standard practice for correction of unit weight and water content for soils containing oversize particles.

American National Standard – ASTM – D5080-08 (2008). Standard test method for rapid determination of percent compaction.

American National Standard – ASTM – D698-12 (2012). Standard test methods for laboratory compaction characteristics of soil using standard effort (12 400 ft-lbf/ft^3 (600 kN-m/m^3)).

Brito, A. (2006). Compaction of dam embankments. New methodologies of control. (In Portuguese.) Tese de mestrado de Geotecnia para Engenharia Civil, Instituto Superior Técnico.

British Standard 1377-4:1990 (1990). Methods of test for soils for civil engineering purposes. Compaction-related tests.

Caldeira, L. and Brito, A. (2007). Compaction control of soil–rock mixtures of Odelouca Dam through vibratory tests. (In Portuguese.) Revista Luso-Brasileira de Geotecnia, no. 109, pp. 43–75.

Hilf, J. W. (1959). A rapid method of construction control for embankments of cohesive soil. Engineering Monograph No. 26, Bureau of Reclamation, p. 23.

Houston, S. L. and Walsh, K. D. (1993). Comparison of rock correction methods for compaction of clayey soils. Journal of Geotechnical Engineering, vol. 119, no. 4, April, pp. 763–778.

Junta Autónoma de Estradas - JAE (1998). Junta Autónoma de Estradas Standards. (In Portuguese.) Lisboa, 1998.

Junta Autónoma de Estradas - JAE /Laboratório Nacional de Engenharia Civil - LNEC (1994). Construction control and physical and mechanical characterization of soil–rock mixtures. (In Portuguese.) February.

Laboratório Nacional de Engenharia Civil (1987). Embankments with soil and rock mixtures. The particular case of weathered schists occurring in Portugal. (In Portuguese.)

Maranha das Neves, E. and Veiga Pinto, A. (1983). Compaction control and random fill Materials. 8th European Conference of Soil Mechanics and Foundation Engineering, Helsinki, Finland.

Pardo de Santayana, F. (2006). Structural behaviour of Beliche earth-rock dam after rehabilitation works. 22nd Congress of Large Dams, Barcelona, Spain.

Torrey, V. H. and Donaghe, R. T. (1994). Compaction control of earth-rock mixtures: a new approach. Geotechnical Testing Journal, ASTM, vol. 17, no. 3, September, pp. 371–386.

USBR 5515-89 (1989). Procedure for performing laboratory compaction of soils containing gravel. U.S. Dept. of the Interior, Bureau of Reclamation. Denver, Colorado.

USCOLD (1998). Construction testing of embankment materials containing large particles. U.S. Committee on Large Dams, April.

Winter, M.G. and Suhardi (1993). The effect of stone content on the determination of acceptability for earthworking. Proc. Conf. Engineered Fills. University of Newcastle upon Tyne, pp. 312–319.

Appendix 1 - Most frequently used corrective equations (adapted from Houston and Walsh, 1993)

1. AASHTO Equation (AASHTO T224, 2010):

$$\gamma_{d\ máx}^{T} = \frac{\gamma_w \gamma_{d\max}^{F} G_M}{\gamma_{d\max}^{F} P_c + \gamma_w G_M \left(1 - P_c\right)}$$

where $\gamma_{d\ máx}^{T}$ is the maximum dry density of the integral material, $\gamma_{d\ máx}^{F}$ is the maximum dry density of the fine fraction (determined using Method A or B, AASHTO T99 or T180), G_M is the solid particle density of the coarse fraction and P_C is the weight percentage of the coarse fraction.

2. ASTM equation (ASTM D4718, 2007):

$$\gamma_{d\ máx}^{T} = \frac{\gamma_w}{\dfrac{P_C}{G_M} + \gamma_w \dfrac{1 - P_C}{\gamma_{d\ \max}^{F}}}$$

The application of this equation is limited to a coarse fraction representing less than 40% and $\gamma_{d\ máx}^{F}$ is determined using ASTM D698 ou D1557.

3. USBR equation (USBR 5515-89, 1990):

$$\gamma_{d\ máx}^{T} = \frac{\gamma_w}{\dfrac{P_C}{G_M} + \gamma_w \dfrac{1 - P_C}{r_u \gamma_{d\ \max}^{F}}}$$

$\gamma_{d\ máx}^{F}$ is determined using Method 5500-89 from USBR. Also, correction factor r_u depends on the coarse fraction and fines composition (clayey, silty, or sandy).

List of Figures

Figures

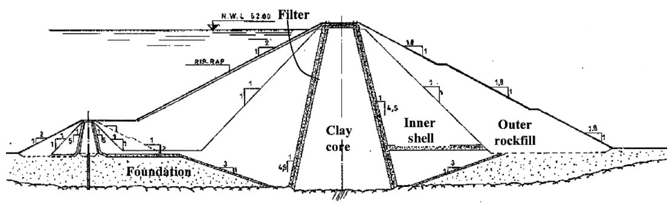


Fig. 1 – Cross-section of Beliche dam

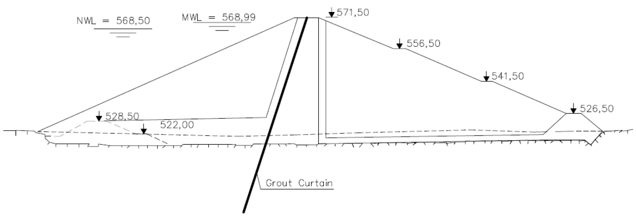


Figure 2

Fig. 2 – Cross-section of Meimoa dam

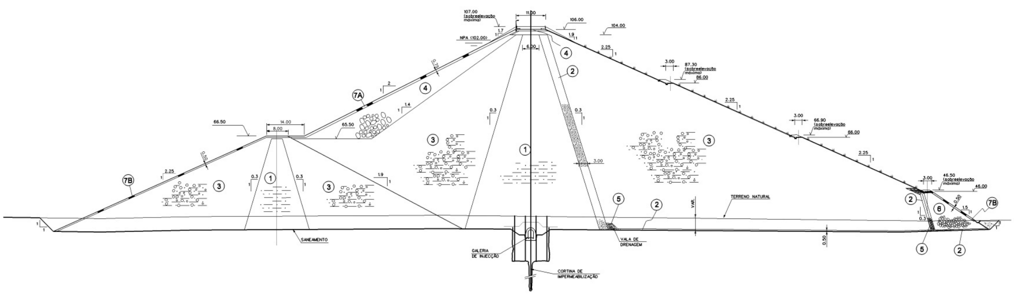


Fig. 3 – Cross-section of Odelouca dam

LEGEND:

1. Residual schist and colluvial material
2. Filter
3. Weathered schist and greywacke
4. Rock fill
5. Drain
6. Downstream toe rock fill
7. Upstream protection layer (>66,5)
8. Upstream protection layer (<66.5)

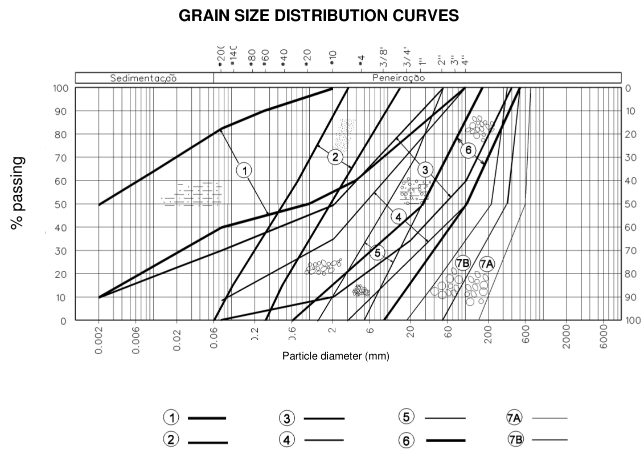


Fig. 4 – Grain-size distribution curves of Odelouca dam materials

a) b)

Fig. 5 – a) Vibratory rammer and mould; b) cylindrical mould

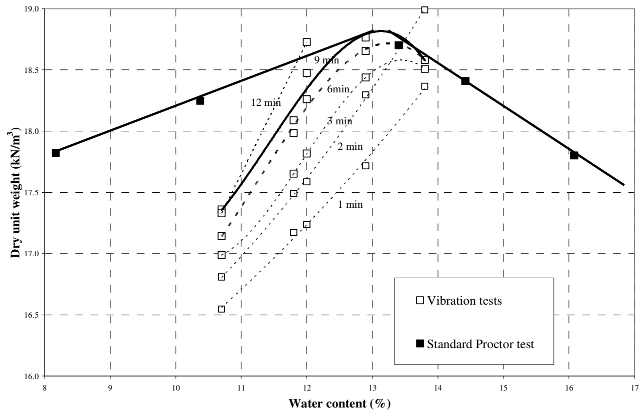


Fig. 6 – Comparison of the compaction curves obtained by vibration and by the standard Proctor test (Caldeira and Brito, 2007)

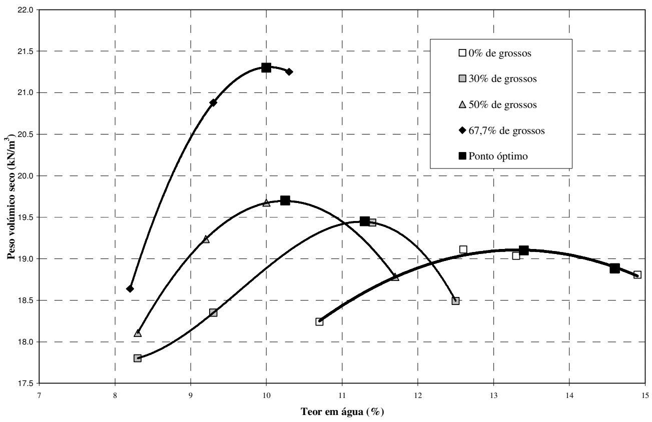


Fig. 7 – Joint representation of compaction curves, after 9 minutes of vibration, and their optimum points

a) b)

Fig. 8 – a) Toni-tecnik compactor; b) mould and pestle used in the standard compaction tests.

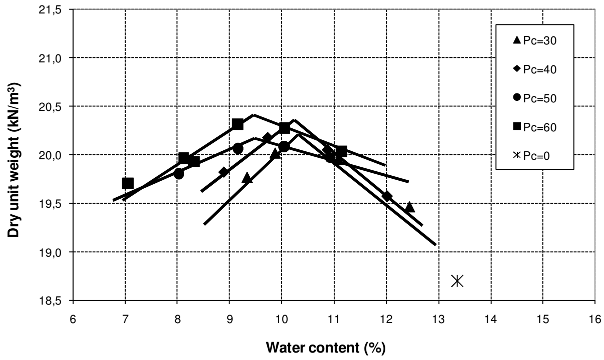
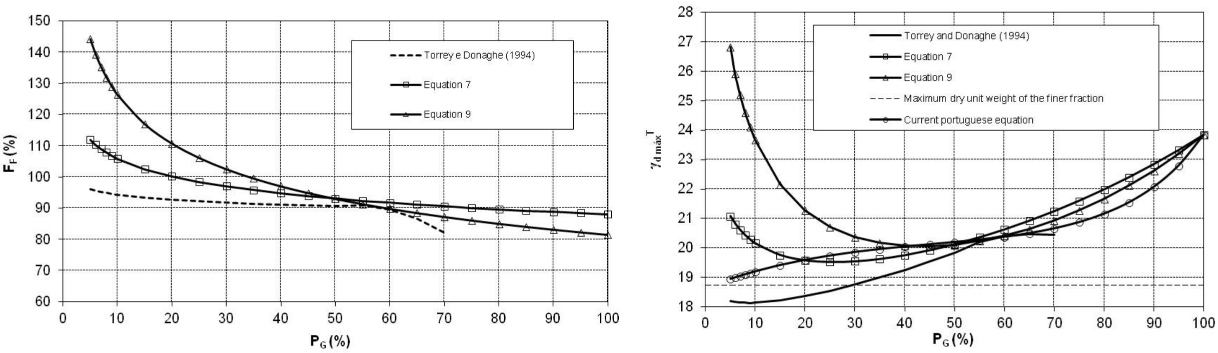


Fig. 9 – Compaction curves from standard compaction tests



a) b)

Fig. 10 – a) Fraction density factor; b) maximum dry density of integral material versus coarse fraction

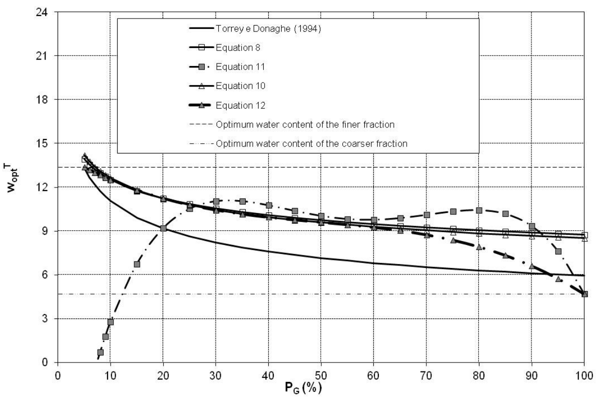


Fig. 11 – Optimum water content of total material as a function of the coarse fraction

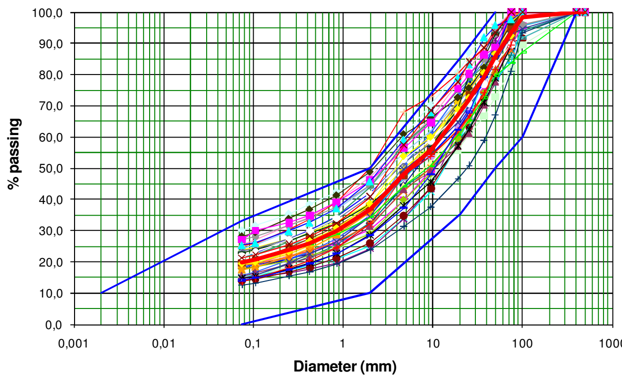


Fig. 12 – Grain-size distribution of shell materials of Odelouca Dam

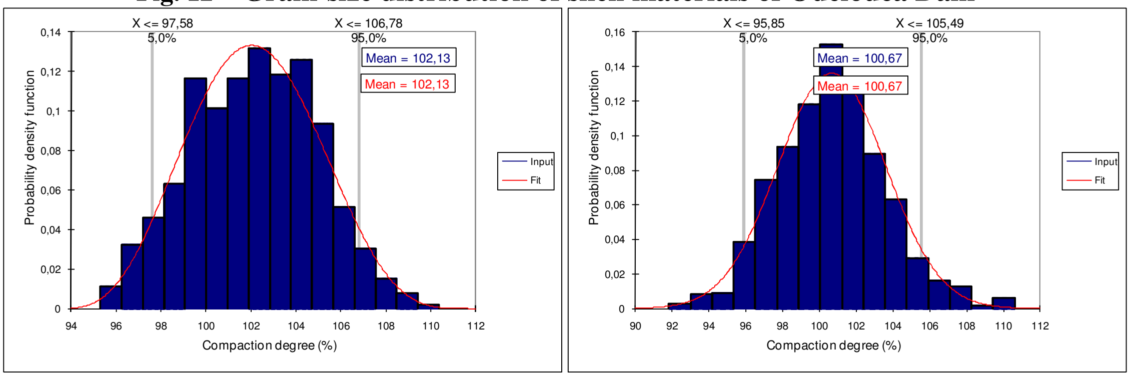


Fig. 13 – Histogram and density probability functions of the degree of compaction of Odelouca dam: a) upstream shell; b) downstream shell.

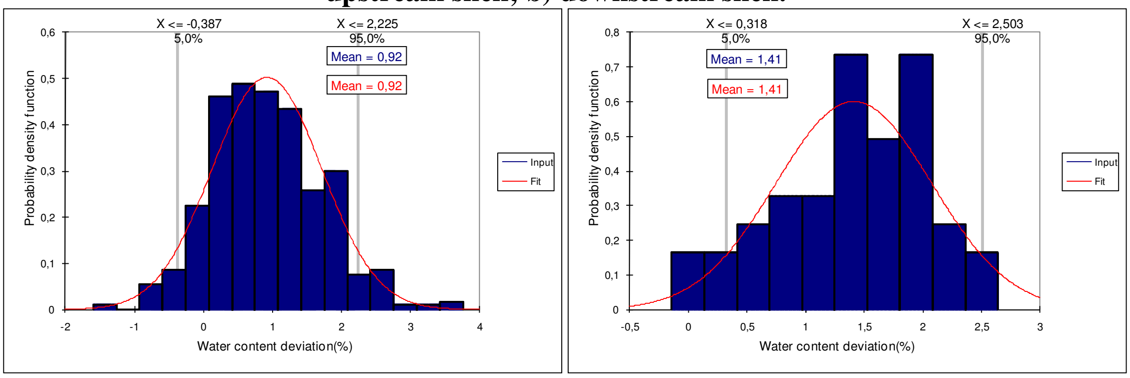
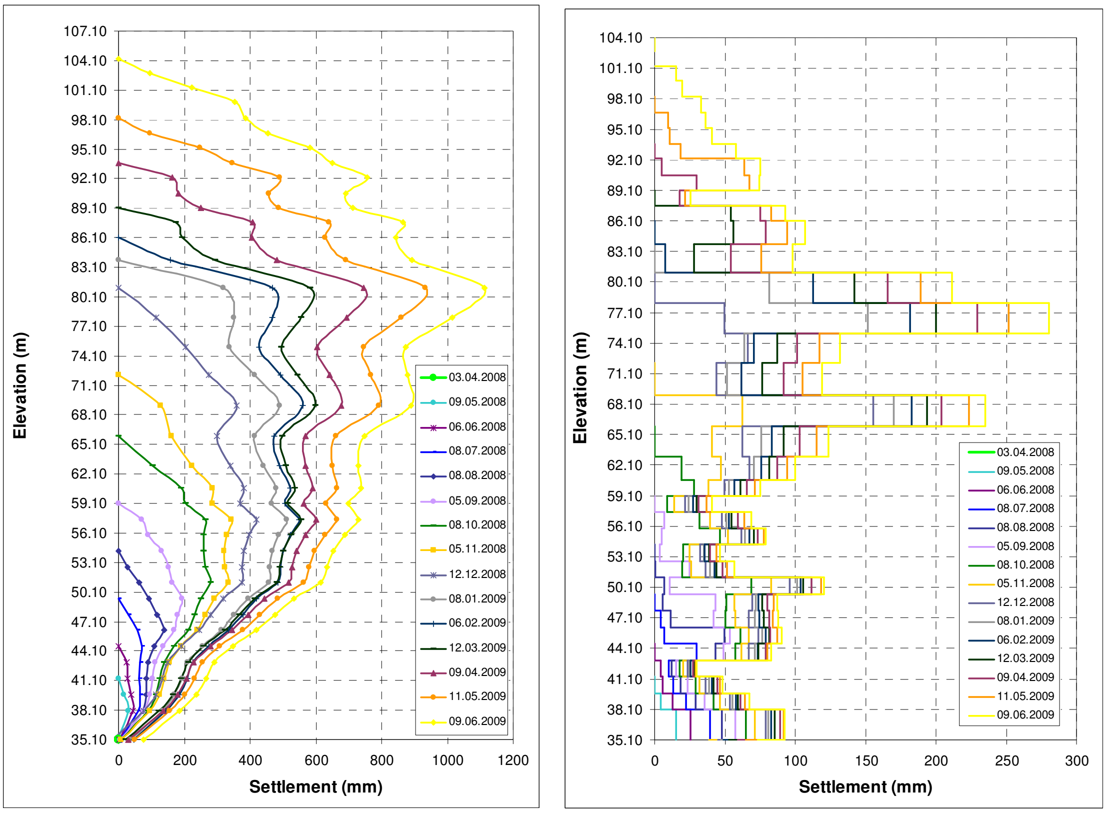


Fig. 14 – Histogram and density probability functions of the water content difference (or deviation) of Odelouca Dam: a) upstream shell; b) downstream shell.



a) b)

Fig. 15 – Settlements during construction of Odelouca dam: a) accumulated; b) incremental

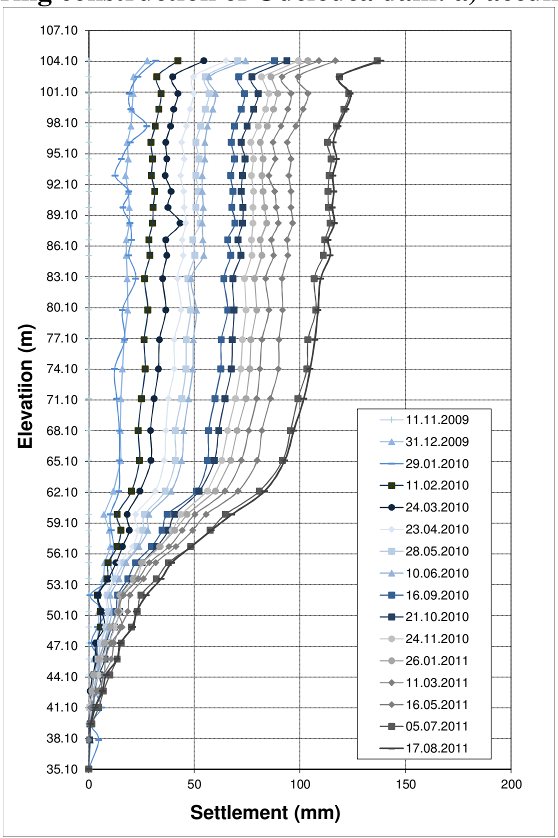


Fig. 16 – Settlements during first filling of Odelouca dam