Use of Soil-Rock Mixtures in Dam Construction

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Abstract: The employment of nontraditional 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, and for the determination of the embankment characteristics 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. The 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 in addition to the dam performance during the construction and first filling phases as a validation of implemented construction procedures. **DOI: 10.1061/(ASCE)CO.1943-7862.0000864.**

Author keywords: Soil-rock mixtures; Construction control; Compaction; Weathered schist; Dam performance; Construction materials and methods.

Introduction

The use of nontraditional 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 (approximately 0.5 m or larger).

Soil-rock mixture material must comply with the following conditions (JAE 1998): (1) The fraction retained in a 19 mm (3/4 in.) sieve must be between 30 and 70%; (2) the fraction passing through a No. 200 (0.074 mm) sieve must be between 12 and 40%; and (3) 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 because of their anomalous deformation behavior, 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. Therefore, 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, in addition to short and long-term behavior 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 of the characteristics determined in the laboratory investigations.

This paper describes two dams that were built with these types of materials and with a defective behavior in terms of deformation. Then, studies already done on the construction quality control of soil-rock mixtures are reviewed, with the aim to improve the performance of the dams, in addition to its application to a new dam—the 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 the Odelouca dam.

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Behavior of Dams Consisting of 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 behavior of the Beliche and Meimoa dams.

The 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-rock mixture essentially constitute the dam body. During construction, in the inner part of the shells, 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 (Fig. 1). For the construction specification of the soil-rock mixture, Maranha das Neves and Veiga Pinto (1983) performed one-dimensional compression

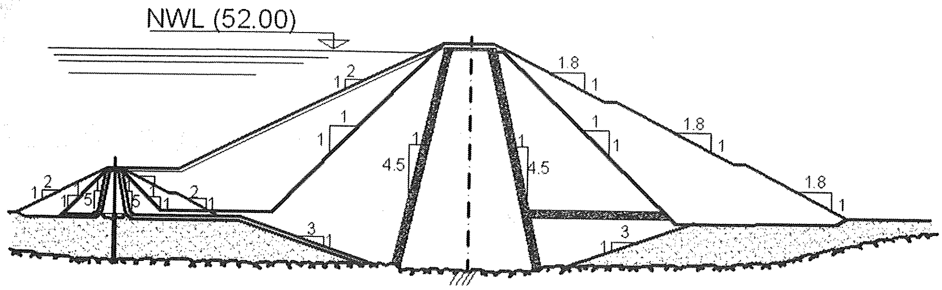


Fig. 1. Cross-section of Beliche dam

deformation tests with different dry density values, water contents, and percentages of coarse fraction. They intended to control the material's oedometric modulus 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 de Santayana 2006), a very heavy rainfall occurred in the winter of 1984–1985 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 (Fig. 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 detect the following:

- 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 resulting from differential settlements, most frequently 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.

These materials exhibit a percentage of rockfill material, controlled during placement by only one parameter—the void ratio. However, they exhibit a percentage of soil, characterized by two parameters—the dry unit weight and the water content. There are doubts about how to treat soil-rock mixtures. Their behavior 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 rockfill 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 19-mm ASTM series sieve).

In support of this hypothesis, USCOLD (1998) recognized 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, on 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

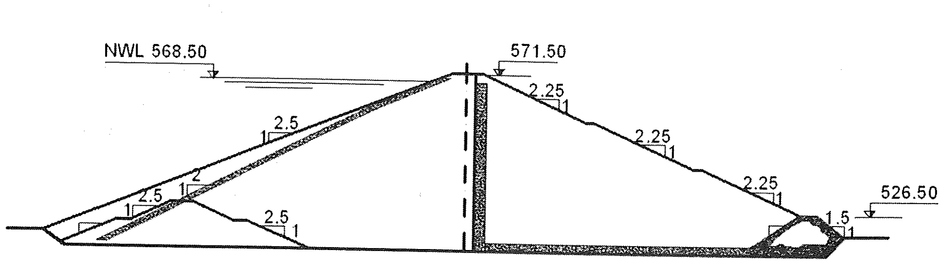


Fig. 2. Cross-section of Meimoa dam

maximum dry density), 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, in addition to the degree of weathering.

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

Previous Studies of Soil-Rock Mixture Compaction

Embankment construction using soil-rock mixtures adopts mixed procedures: construction techniques employed in the rockfill 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, 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 behavior of the soil-rock mixture. Its behavior 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 recommended the adoption of different procedures for compaction control according to the coarse fraction present in the mixture. For materials in which less than 45–50% of the 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 eliminating the coarser particles). If the proportion of blocks rises significantly above 45–50% 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.

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

- Method 1 (integral sample): Performed in large molds, using the integral field material, for evaluation of the maximum dry density and optimum water content;
- Method 2 (scalp and replace): Performed in molds of smaller dimensions, removing the material larger than 19 mm (3/4 in.) and replacing it with an equal weight of No. 4 to 19 mm (3/4 in.) sieve material;
- Method 3 (elimination and correction): Performed using only the material passing through a 19 mm (3/4 in.) sieve in a 15.24 cm (6 in.) mold; material larger than 19 mm (3/4 in.) is discarded; and
- 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 (ASTM 2007), Standard T224 from ASSHTO, or Standard 5515-89 from the U.S. Dept. of the Interior, Bureau of Reclamation (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 10–60% coarser particles. Method 3, valid when the material larger than 19 mm (3/4 in.) represents less than 10% of the mixture by weight [ASTM D698 (ASTM 2012)], is only suitable for mixtures with reduced percentages of coarser particles and whose behavior is clearly controlled by the finer matrix.

The appendix presents the correction equations used most frequently 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 19 mm (3/4 in.) sieves of the ASTM series], the following expressions are proposed:

$$\gamma_{d\,\max}^T = \frac{100}{\frac{P_C}{\gamma_d^C} + \frac{P_F}{\eta\gamma_{d\,\max}^F}} \tag{1}$$

$$w_{\mathrm{opt}}^T = \frac{P_F w_F + P_C w_C}{100} \tag{2}$$

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

$$\eta = -5 \times 10^{-5} P_C^2 + 0.0013 P_C + 0.9958 \tag{3}$$

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 molds and with materials consisting of gravel, sand, and nonplastic silts or high-plasticity clays. To analyze the results, they defined two additional quantities: the density interference coefficient, I_c, and the optimum water content factor, F_{opt}, expressed as

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

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

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

To calculate F_F, the authors applied the following equation:

$$F_F = \frac{\gamma_{d\max}^T G_M \gamma_w P_F}{100\gamma_{d\max}^F G_M \gamma_w - \gamma_{d\max}^T \gamma_{d\max}^F P_C} \quad (6)$$

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 on 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 approximately 415-m long (Fig. 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. Fig. 4 presents the grain-size distribution curves of the materials used in the construction of the 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 19 mm (3/4 in.) sieves, respectively; and average and maximum particles diameters between 2 and 50 mm 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 program 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 because of 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 light 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 BS 1377 (BSI 1990). This standard uses the following equipment: a CBR mold, an electric vibrating hammer, a steel tamper for attachment to the vibrating

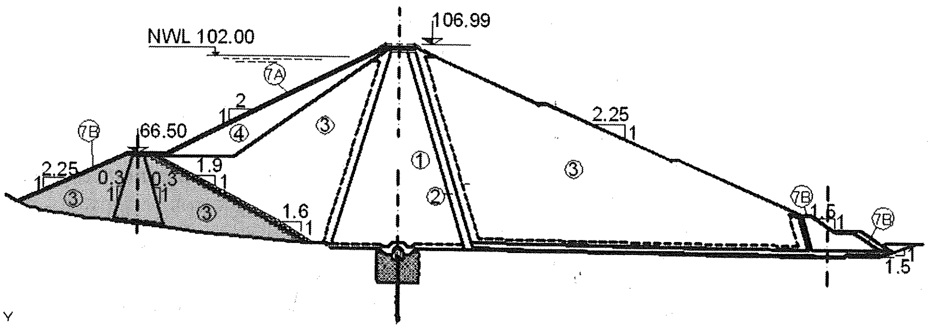


Fig. 3. Cross-section of Odelouca dam

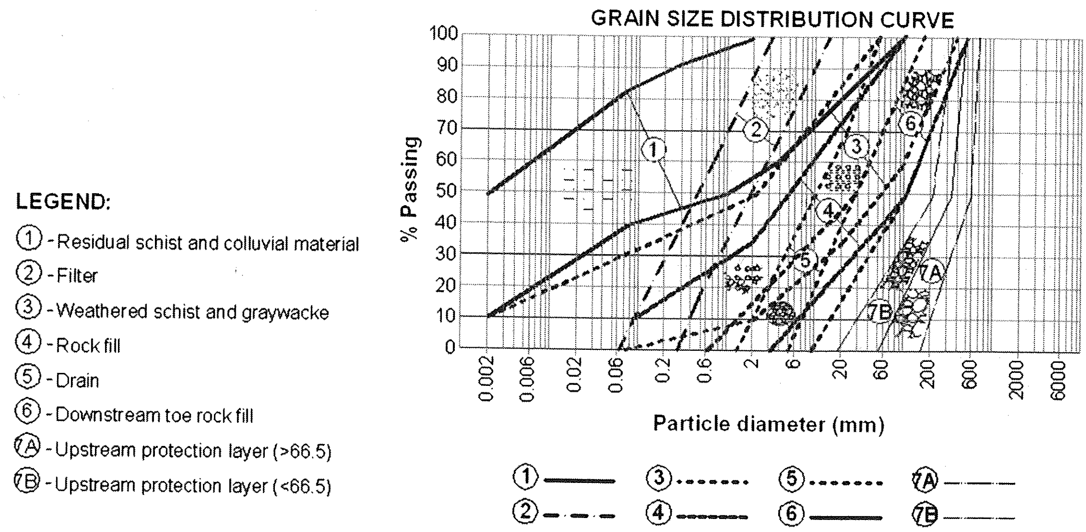
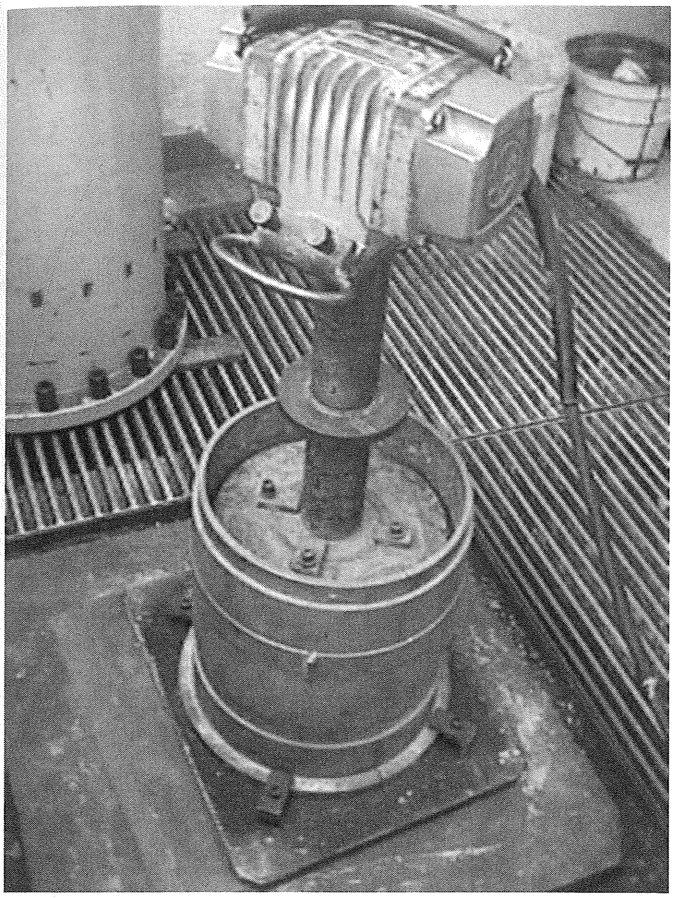


Fig. 4. Grain-size distribution curves of Odelouca dam materials



(a)

(b)

Fig. 5. (a) Vibratory rammer and mold; (b) cylindrical mold

hammer, a device enabling sample depth measurement, and a stop clock.

The cylindrical mold [310-mm inside diameter, 300-mm height, sectioned longitudinally, connected with rigidity ribs, and fixed to a metal base (Fig. 5)] is easily removable and transportable. The material is compacted with a vibratory rammer coupled to a steel plate. The gap between the mold and the plate is approximately 10 mm. The total force exerted, including the rammer and plate weights, reaches 2,954 N. In light of the mold diameter (300 mm),

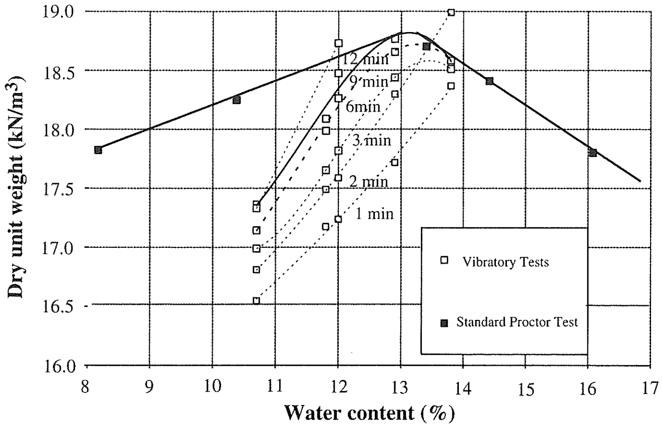


Fig. 6. Comparison of compaction curves obtained by vibration and by the standard Proctor test (data from Caldeira and Brito 2007)

the maximum dimension of the mixture particles was limited to 50.8 mm (2 in.), 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 approximately 2 min of vibration, some swelling was detected, possibly the result of material segregation, with a consequent reduction of its dry density; and
- For the remainder of the tests, regardless of water content, the maximum dry density was reached after 9 min 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 19 mm (3/4 in.) sieve (Fig. 6). Analysis of Fig. 6 reveals the following:

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

Next, vibration compaction tests with different percentages of coarse fraction (between 30 and 67.7%) were performed using the available lots. Fig. 7 represents the compaction curves obtained after 9 min 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 (FröWag, model 2.172, Germany) using different coarse fractions in a mold of appropriate dimensions (Fig. 8). Naturally, it was necessary to adapt the standard procedures of ASTM D698 (ASTM 2012) because of

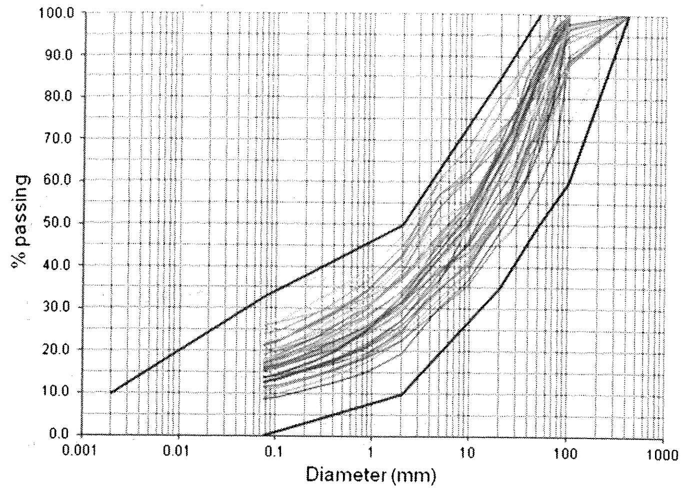


Fig. 12. Grain-size distribution of shell materials of Odelouca dam

The alternative Eq. (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 approximately 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 Eq. (12) has a similar course to Eqs. (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

Fig. 12 presents the grain-size distribution of the shell materials of the dam tested during construction, in addition to the granulometric range defined in the design. The construction quality control of the compacted layers applied Hilf's method [ASTM D5080 (ASTM 2008)] (Hilf 1959) to the fraction of the soil-rock mixture passing through the No. 4 sieve. Based on Hilf's results (w_{opt}^F, $\gamma_{d\,\max}^F$), the optimum water content and the maximum dry density of the integral material were obtained (w_{opt}^T, $\gamma_{d\,\max}^T$) 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:

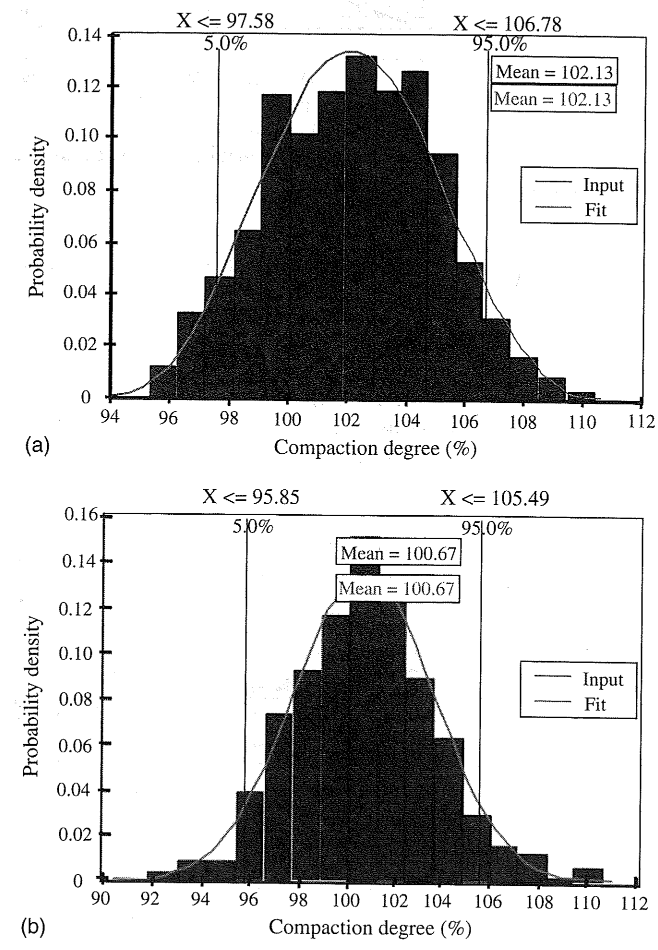


Fig. 13. Histogram and density probability functions of the degree of compaction of Odelouca dam: (a) upstream shell; (b) downstream shell

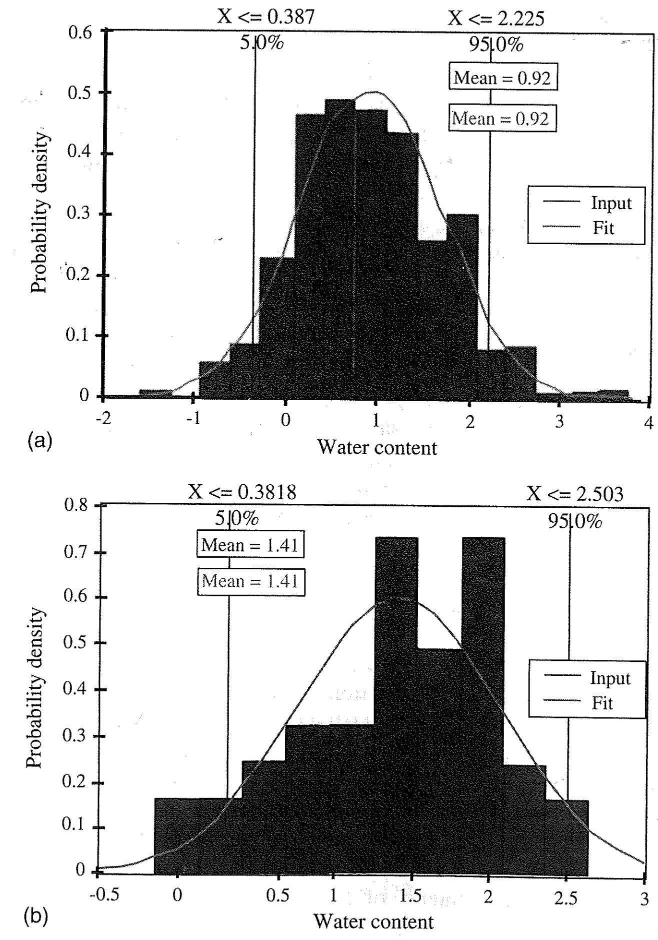


Fig. 14. Histogram and density probability functions of the water content difference (or deviation) of Odelouca dam: (a) upstream shell; (b) downstream shell

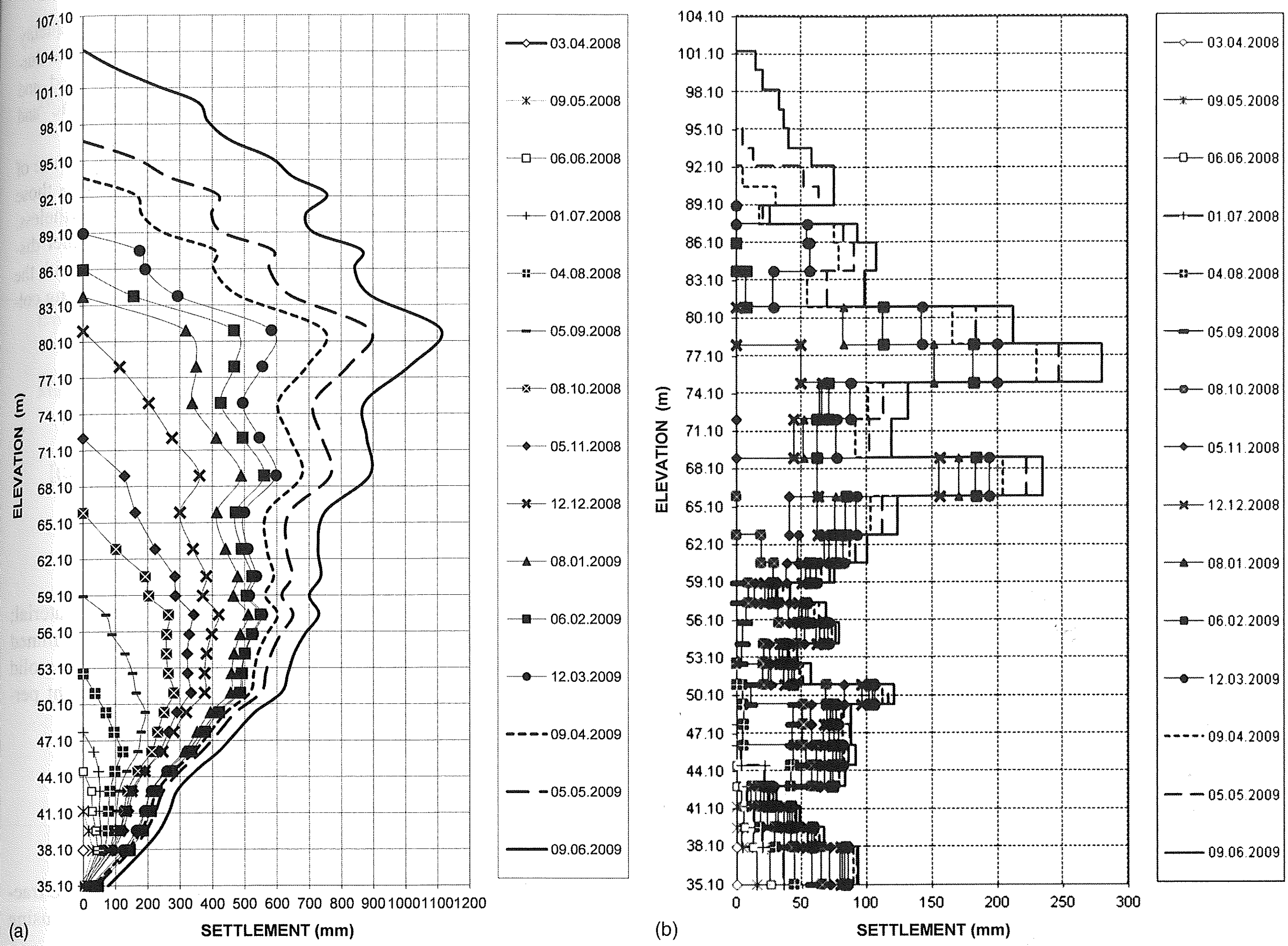


Fig. 15. Settlements during construction of Odelouca dam: (a) accumulated; (b) incremental

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

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

Figs. 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.

Fig. 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 approximately 1,100 mm.

Examining the incremental settlements of the different layers between the measuring points [Fig. 15(b)], the major contribution is from the layers between elevations of 75 and 78 m (280 mm) and between elevations of 66 and 69 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 Fig. 15(b) confirms.

From the comparison of the construction settlements of the 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 less than those observed in the Beliche and Meimoa dams, as Fig. 16 shows, proving the very good performance of the dam.

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

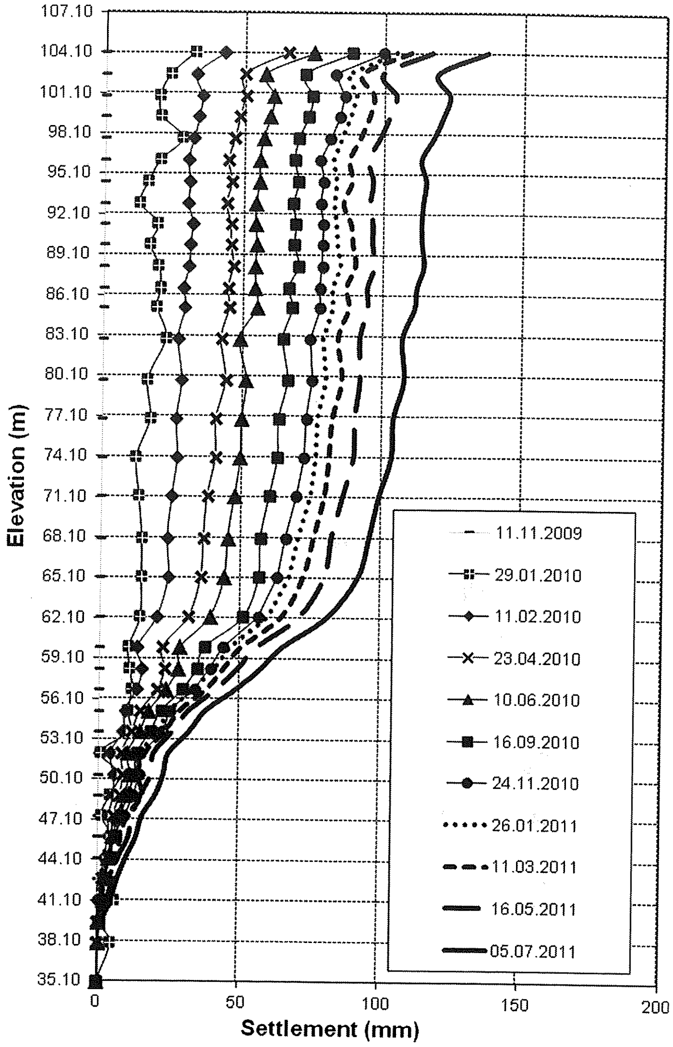


Fig. 16. Settlements during first filling of Odelouca dam

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 one 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, in addition to the coarse fraction determination, are required for the quality control of these materials.

The quality control of the construction of the Odelouca dam used Hilf's method; these corrective expressions, conveniently adapted to the extracted materials, were evidence of 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 analyzed and compared with the monitoring results from the Beliche and Meimoa dams.

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

Appendix. Most Frequently Used Corrective Equations (data from Houston and Walsh 1993)

1. AASHTO equation [AASHTO T224 (AASHTO 2010)]

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

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

2. ASTM equation [ASTM D4718 (ASTM 2007)]

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

The application of this equation is limited to a coarse fraction representing less than 40%, and $\gamma_{d\max}^{F}$ is determined using ASTM D698 (ASTM 2012) or D1557.

3. USBR equation [USBR 5515-89 (USBR 1989)]

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

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

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