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MEASURED VERSUS COMPUTED SETTLEMENTS OF LA YESCA DAM UPON ITS CONSTRUCTION

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Dedication

In memory of Professor and Researcher Jesus Alberro Aramburu.

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Resumen

Una de las principales limitaciones en el análisis de presas de enrocamiento es la definición apropiada del comportamiento de los materiales gruesos (enrocamientos) bajo la gran variedad de presiones intergranulares y trayectorias de esfuerzos que se desarrollan en el cuerpo de una presa de enrocamiento durante su construcción. Una de las razones (quizá la más importante) por la que resulta difícil definir las características esfuerzo-deformación de los enrocamientos es el gran tamaño de sus partículas, las cuales normalmente exceden las capacidades de los laboratorios convencionales (y de investigación) existentes. En vista de esta limitación, hace tiempo se inició una investigación con el fin de monitorear con rosetas integradas con celdas de presión y extensómetros, colocadas a diferentes elevaciones del enrocamiento a medida que la construcción de la presa proseguía. Este proceso se llevó a cabo en varias presas de enrocamiento, incluyendo una de enrocamiento con cara de concreto (CFRD), construidas en la República Mexicana. La evaluación de los resultados del monitoreo, acoplados con los resultados de pruebas de laboratorio en odómetros gigantes muestran que la relación esfuerzo-deformación (en particular el módulo de Young E) puede simularse con una ley de potencia y así modelar con métodos numéricos (*i.e.* elementos finitos, diferencias finitas, etc.) la construcción de presas de enrocamiento. La interpretación de las mediciones obtenidas, en términos de esfuerzos y deformaciones octaédricos, las relaciones esfuerzo-deformación resultan para todos fines prácticos lineales, al menos, hasta el fin de la construcción de las presas. Esto significa que el módulo de elasticidad depende del nivel de esfuerzos, conforme a una ley de potencia como se indica en este documento técnico. A la luz de estos resultados, y para evaluar la confiabilidad de este simple modelado, se calcularon los esfuerzos y deformaciones en la presa La Yesca desarrollados durante su construcción. En este artículo se muestra una comparación entre los asentamientos medidos y calculados (usando un modelo 3D de diferencias finitas, en el cual se integró el modelado del módulo de elasticidad del enrocamiento con una ley de potencia) en esta presa durante su construcción. Esta comparación muestra que las computaciones analíticas coinciden bastante bien con las medidas, por lo que se puede concluir que el modelado de la presa La Yesca y la simulación del comportamiento del enrocamiento son correctos.

Palabras clave: Presas de enrocamiento, Comportamiento esfuerzo-deformación, Modelado numérico, Módulo de Young, Ley de Potencias.

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Abstract

The definition of rockfill-loading behaviour is one of the difficulties the dam engineer faces to properly model a rockfill dam's behaviour upon their construction and hence reach confident results that lead to safe designs. Perhaps, the main reason why the stress-strain rockfill properties are difficult to obtain is the large size of the rockfill particles that usually exceed the capabilities of most existing geotechnical laboratories. Accordingly, field research was carried out to monitor the behaviour of several rockfill dams built in Mexico with several rosettes of pressure cells and of extensometers, placed at several elevations within the rockfill embankments as they were built. Therefore, it was possible to define the stress-strain characteristics of the rockfill when the monitoring results were coupled with giant odometer laboratory test results. On the basis of these results, a power law was found to be good enough to model the rockfill behaviour under increasing normal loading. To assess the confiability of the 3D finite differences-power law model, the stress-strain behaviour of La Yesca dam was calculated throughout its construction. This paper shows a comparison between the monitored settlements and those computed using the above mentioned power law included in a 3D finite differences model and clearly indicates that this analytical model reproduces quite well the measured settlements, which leads to the conclusion that the proposed modelling of La Yesca dam is adequate and, henceforth, this modelling can be applied confidently to design rockfill dams.

Keywords: Rockfill dams, Stress-strain behaviour, Numerical modelling, Young's Modulus, Power Law.

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1. Introduction

Concree` face rockfill dam` (CFRD’)` a.e composed of a zoned rockfill embankment, which is sealed on the upstream slope with a thin slab of hydraulic concrete that rests on a concrete foundation, commonly known as plinth. This dam’s type is considered an attractive option in most sites with rocky foundations and they are generally competitive with rockfill dams with clay core, and even with concrete dams, mainly due to a number of improvements in their design and construction procedures (Sherard & Cooke, 1987). Consequently, the total number of CFRD’s that are in operation, under construction or in the planning stage around the world, has increased swiftly. However, it seems that this trend now has levelled off. The maximum height of this type of dams already exceeds 170 m (*i.e.,* Ma & Chi, 2016; Jia, 2016). Examples of high CFRD are Shuibuya and Tianshengqiao dams in China (233 and 178 m, respectively), Campos Novos dam in Brazil (202 m), Bakun dam in Malaysia (205 m), Aguamilpa (187 m), El Cajon (189 m) and La Yesca (210 m) dams in Mexico.

It is well known that there are several constitutive models that have been defined from stress trajectories developed mainly in triaxial laboratory tests that lay down the bases to model the behaviour of coarse granular particles for the design of rockfill dams. Evaluation of the properties of large particles assemblies such as the boulders used in rockfill dams is a difficult task mainly because of particle sizes. For example, to perform a triaxial test on a sample formed with, say, 0.2 m size particles, and fulfilling the requirements of aligning at least eight particles along the diameter of the sample and having a minimum slender ratio of 2.5, the cylindrical sample would be 4.0 m high and have a 1.6 m diameter. To perform triaxial tests on such a big sample, the laboratories would require colossal installations.

Several researchers have overcome this problem by scaling down the particle’s size and developed sophisticated constitutive models. For example: (Zou, Xu, Kong, Liu & Zhou 2013); (Varadarajan, Sharma, Venkatachalam & Gupta, 2003); (Xiao *et. al*, 2016) and

(Honkanadavar & Sharma, 2016). Their great contributions to the knowledge of static and seismic behaviour of the rockfill is beyond any doubt, but still there are unanswered questions as to how the scaling factor would affect the tests results; ergo, the constitutive models. In practice, several rockfill constitutive models allow an approximate representation of some of these material characteristics such as inelastic behaviour, non-linearity and high dependence on the stress confining characteristics. According to their nature, the models that have been proposed up to now can be classified as elastic (linear and non-linear such as the hyperbolic approach), Elastoplastic, and Viscoelastic, as well as Empirical procedures.

Perhaps the first model used to capture the stress-strain behaviour of rockfill materials and applied to analyze, using the finite element method, rockfill dams was the Hyperbolic one (Alberro & Romo, 1969), (Kulhawy & Duncan, 1972), (Saboya Jr & Byrne, 1993). Afterwards, hypoplastic constitutive models to describe the mechanical characteristics of the rockfills were proposed (Bauer, 2009). Moreover, specific models to address singular problems such as settlements by collapse in rockfill embankments due to material saturation were developed (Escuder, Andreu & Rechea, 2005).

Numerical procedures such as finite element or finite difference methods have been widely used in the calculation of the stress-strain state of earth and rockfill dams during their construction and reservoir filling. In this paper only the former is considered. The benefits of these numerical tools are already remarkable; however, theoretical predictions are sometimes unreliable, because the properties of rockfill material samples are not fully representative of the *in situ* ones because they were tested following stress trajectories that most of the times do not accurately reflect the *in situ* loading conditions the embankment endures during its construction. It is important to express the author's belief on the fact that properly focused analysis of data obtained directly from the monitoring of prototypes provide valuable information to assess the model that better reproduce the actual behaviour of rockfill dams upon their construction.

With the purpose of shedding some light on this subject, this work presents the monitoring stress-strain results recorded during the construction of Aguamilpa dam built in Mexico. From this research (that included the monitoring of a number of rockfill embankments), a simple yet reliable stress-strain power law to model the rockfill behaviour of dams upon construction is properly established. Based on the *in situ* measurements of the stresses and deformations generated during the construction of this and other equally monitored rockfill dams as well as corresponding laboratory tests on odometers of 1.0 m diameter coupled with *in situ* embankment tests, the authors after detailed studies came to the conclusion that the stiffness modulus of rockfill materials depend heavily on the confining octahedral stresses, their stress trajectory and the all-around nearby material stiffness. It is needless to stress the fact that, due to the irregular construction of rockfill dams all these conditions are difficult if not impossible to be jointly reproduced in laboratory tests.

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2. Aguamilpa CFRD

This dam is a concrete face rockfill dam having a maximum height of 187 m and a 642 m crest length located on the Santiago River, in Nayarit State, Mexico. The maximum cross section and most of all the instrumentation installed in this dam section, whose upstream and downstream slopes are 1.5H:1V and 1.4H:1V respectively, are indicated in Figure 2.1.

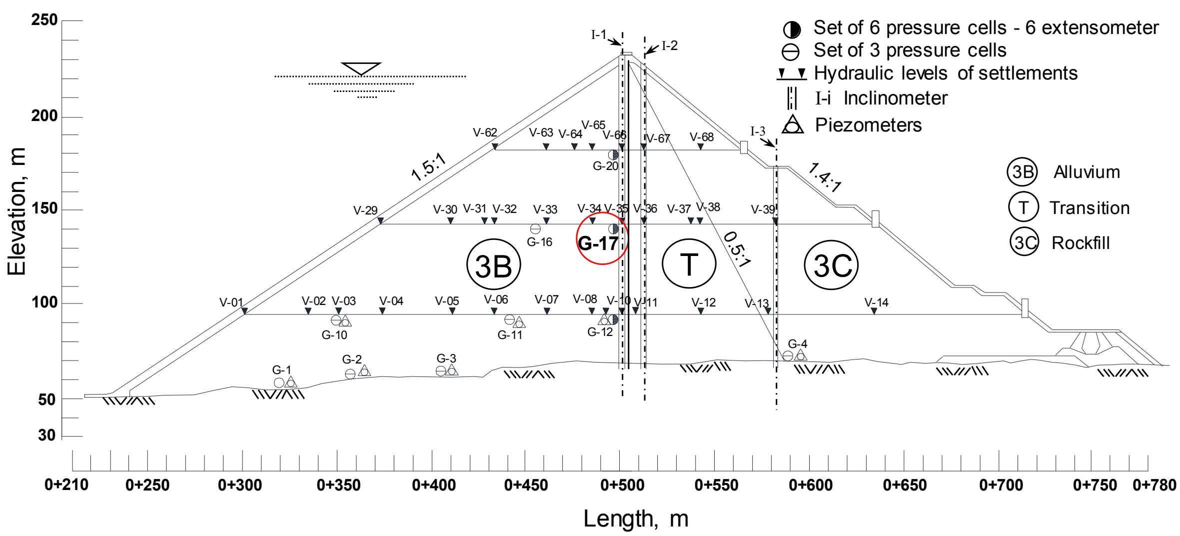

Figure 2.1 Instruments at the Aguamilpa dam main section (modified from (Alberro, Macedo & González, 1998))

As depicted in this figure, the half upstream of Aguamilpa dam was built using only material 3B, followed by a transition material T having a vertical slope on the upstream side and a 0.5H:1V slope on the downstream side. The material 3B consists of natural alluvium (diameters < 10 cm and a volume of fines < 2 %) compacted in layers 0.6 m thick with four roller passes using a vibratory steel drum roller weighing 100 kN; the material T is composed of rock fragments (diameters < 0.6 m) and gravels also compacted in 0.6 m thick layers, with four roller passes using similar vibratory rollers as for material 3B; the material 3C, which complements the dam section, consists of healthy rock fragments (diameters < 1.0 m) compacted in layers 1.2 m thick with four roller passes using also vibratory rollers similar to those employed for compacting materials in zones 3B and T. The construction of Aguamilpa dam began in 1990 and its operation started in 1994. Aside from the stress and strain rosettes, the whole instrumentation of the dam includes inclinometers, pressure cells, pneumatic piezometers, extensometers and hydraulic levels to measure dam settlements; therefore, providing redundancy to the rosettes measurements. The spatial distribution of the monitoring instruments, including the deployed rosettes, is depicted in Figure 2.1 for the main dam section.

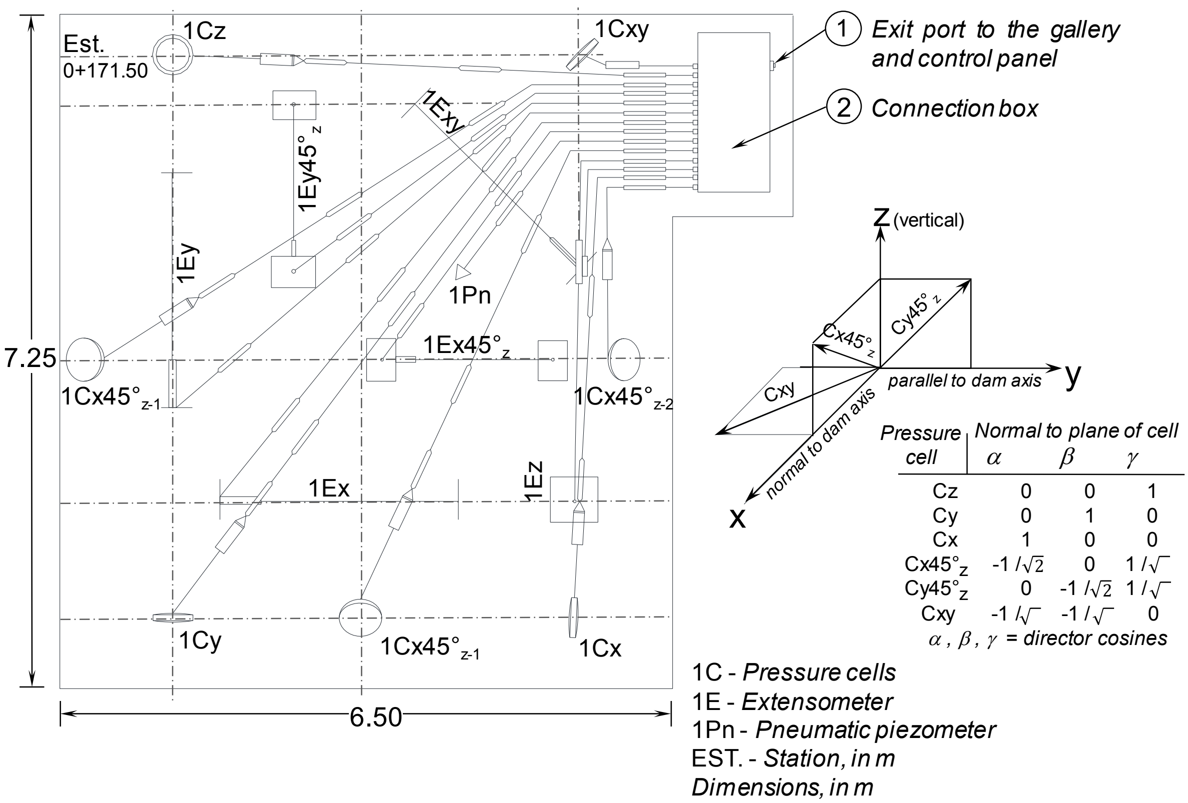

Figure 2.2 Schematic layout of stress and strain rosettes (modified from (Comisión Federal de Electricidad, CFE, 1985))

To monitor, and hence, to define *in situ* the stress-strain behaviour of the rockfill, the rosettes were located at a certain number of points throughout the embankment as its construction progressed. The rosettes (schematically depicted in Figure 2.2) allow the 3D stress-strain measurement at the points where they were located. Since they were set up at various zones (see Figure 2.1), the stresses and strains developed correspond to the actual response of the rockfill upon the actual uneven loading. As already mentioned, each of these rosettes (G-i) is composed of pressure cells and extensometers, and located within the 3B, T and 3C rockfill zones. The information presented in this paper only refers to the data gathered with rosette G-17 (see Figure 2.1). The location of the cells and extensometers integrating each rosette, together with the reference coordinate system for instruments orientation, is depicted in Figure 2.2.

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3. Rockfill stress-strain relationships: monitoring and laboratory tests

From *in situ* measurements carried out with the rosettes placed in the embankment rockfill zones of the Aguamilpa dam, the magnitude and direction of the effective principal stresses and principal strains are determined throughout the dam construction, at all the points where the rosettes are located. From the arrangement of the pressure cells and extensometers, stress-strain tensors can be obtained as depicted in Figure 2.2. Based on the knowledge of stress and strain tensors, it is a straightforward issue to obtain the magnitudes and directions of the principal stresses and strains. With the above information, the magnitudes of σ_{oct}, ε_{oct}, τ_{oct} and γ_{oct} defined by Eqs. 3.1-3.4:

$$\sigma_{oct}=\frac{1}{3}(\sigma_1+\sigma_2+\sigma_3) \tag{3.1}$$

$$\varepsilon_{oct}=\frac{1}{3}(\varepsilon_1+\varepsilon_2+\varepsilon_3) \tag{3.2}$$

$$\tau_{oct}=\frac{1}{3}\sqrt{[(\sigma_1-\sigma_3)^2+(\sigma_1-\sigma_2)^2+(\sigma_2-\sigma_3)^2]} \tag{3.3}$$

$$\gamma_{oct}=\frac{1}{3}\sqrt{[(\varepsilon_1-\varepsilon_3)^2+(\varepsilon_1-\varepsilon_2)^2+(\varepsilon_2-\varepsilon_3)^2]} \tag{3.4}$$

where σ_1, σ_2 and σ_3, are the effective principal stresses, and ε_1, ε_2 and ε_3, are the corresponding principal strains. From the measurements obtained during the construction of Aguamilpa dam, with the rosette G-17 installed in the 3B rockfill (see Figure 2.1), the curves among the octahedral normal and octahedral shear stresses versus octahedral normal and octahedral shear strains, are plotted in Figure 3.1.

Similar results, in terms of the stress-strain behaviour, were obtained in other heavily instrumented rockfill dams in Mexico (*e.g.*, El Caracol and Peñitas dams); the corresponding field stress-strain relationships are plotted in Figure 3.2. Their description and location, as well as their overall instrumentation are beyond the purpose of this paper; accordingly, the reader is adverted to reference (Alberro, 1998).

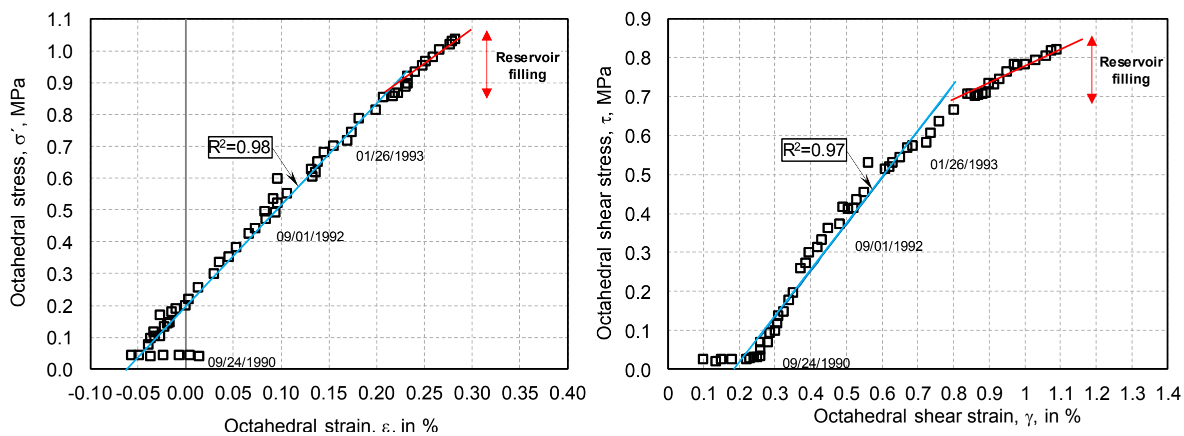

Figure 3.1 Octahedral stress-strain relationships measured in the rockfill of Aguamilpa dam

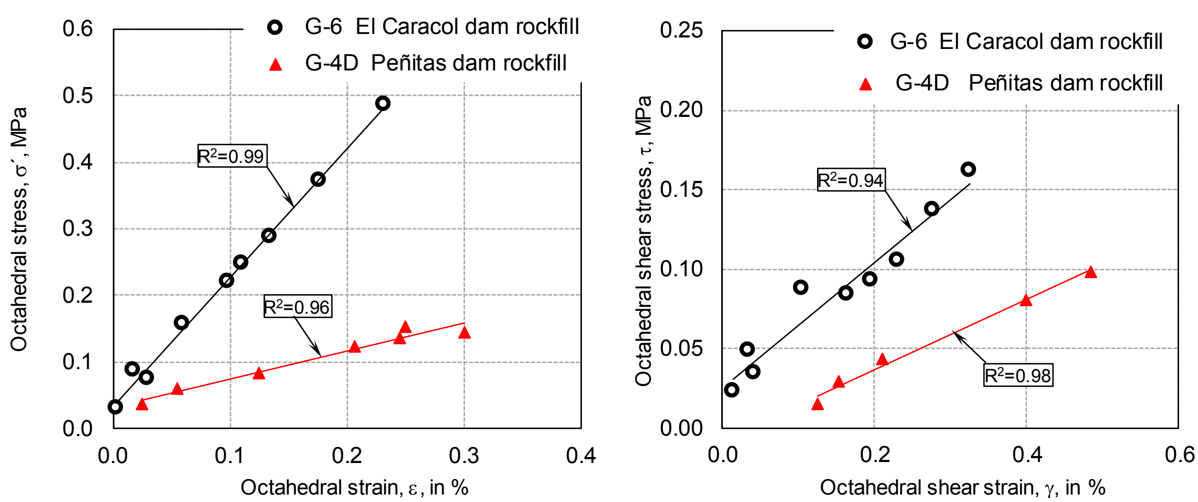

Figure 3.2 Octahedral stress-strain relationships obtained from measurements in the rockfill of El Caracol and Peñitas dams

From the results, in terms of σ_{oct}-ε_{oct} and τ_{oct}-γ_{oct}, obtained with the rosettes set up within the Aguamilpa rockfill and other dams in Mexico (Figures 3.1 and 3.2), it is concluded with a high degree of certainty that the rockfill material behaves linearly during the construction of the embankment. Notice that the statistical R^2 values written next to the straight lines in Figures (3.1 and 3.2) are equal or larger than 0.94, which supports the fact that the relationship mentioned is linear. It is important to note that, even for octahedral strains, up to 0.8 %, the stress-strain relationship is for all practical purposes linear. Accordingly, one can safely assume that the behaviour of the rockfill is practically linear until the end of the dam's construction, and it would likely continue behaving similarly (with slightly lower stiffness) upon completion of the filling of the dam's reservoir, as depicted in Figure 3.1. Accordingly, the slight change on the rockfill stiffness due to reservoir filling (particularly in the upstream side of zoned rockfill dams) must be taken into account when designing this type of structures. Furthermore, from these measurements (mainly from the strains measured on the material 3B of the Aguamilpa dam) one can deduce whether or not the rockfill was well compacted. This statement is backed by the relationship between octahedral shear stress and octahedral shear strain yielded by the data collected monitoring rosette G-17 and plotted at the right-hand side of Figure 3.1. Notice that the straight line slightly curves down as the octadedral strain exceeds 0.8%, meaning that the stiffness of the 3B rockfill has decreased, most likely because the rock particles rearrangement caused some settlements to accumulate towards the upper part of the dam. This causes the concrete slabs that initially were bowl-shaped to bend slightly upwards, causing the concrete face slab to modify its original shape to a kind of an elongated "**S**" that causes tensile stresses in some zones of the upper part of the concrete slab that induced limited fissuring in some areas of the concrete slab. Similar phenomena happened in the transition zone (T) and in the downstream slope (rockfill 3C) that enhanced the effect of the settlements on the final (**S**) shape of the concrete slabs and consequently in their fissuring. It is worth to pinpoint the fact that due to the large size of particles integrating the rockfill of the downstream slope, the compaction achieved was not good enough to avoid further settlements that induced more cracking into the concrete slabs.

From the linear stress-strain relationships and the elasticity theory, it is possible (at least as a first approximation) to determine the Young's modulus, E, and the Poisson's ratio, υ, of the rockfill embankment during its construction. In the case of rockfill 3B of the Aguamilpa dam, the stiffness modulus values, E, obtained varied approximately between 100 and 150 MPa for Poisson ratios, υ, varying from 0.27 to 0.30. Although, from the measurements of both hydraulic levels and inclinometers during the embankment construction, the stiffness modulus of the rockfills can be estimated, the authors believe that those defined using the strain and stress rosette measurements are more reliable, simply because they are not affected by any hypothesis in terms of the stresses distribution in the rockfills.

The values of the rockfill modulus obtained from the *in situ* stress-strain relationships were also compared with those obtained from *in situ* confined plate tests, and 1.0 m diameter odometer tests in the laboratory, obtaining comparable values. It should be mentioned that for the odometer tests carried out for vertical stresses varying within the range of 0.1 to 1.0 Mpa, an average value of the E modulus of 105 Mpa for the 3B material of the Aguamilpa dam was determined (Alberro, Macedo & González, 1998), which falls within the values defined from the rosettes measurements.

3.1 Rockfill modulus variations in terms of the octahedral confinement stresses

It is clear that the rockfill materials in the lower layers of the embankment have appreciably smaller void ratios than at the upper layers; hence, the stiffness modulus (at a specific depth) increases as the height of the embankment rises over it. Similarly, due to the arching-type phenomenon the rockfill becomes stiffer as we move away from the dam abutments. For these and other reasons, the rockfill stiffness varies spatially throughout the dam and hence it is a must to take into account these spatial rockfill stiffness variations in terms of the confinement stresses to obtain more reliable numerical results.

From the results of several *in situ* plates tests performed on test embankments and laboratory tests (a number of studies have shown that one-dimensional compression odometer tests are useful to estimate reliably the stiffness modulus of rockfill (Charles, 1976)), for various rockfill materials included in several dam projects in Mexico: Aguamilpa, El Cajon, and La Yesca dams [*i.e.*, (Romo, *et. al*, 2002), (Romo, Botero, Méndez, Hernández & Sarmiento, 2007)], the following power expression that considers the dependence of the rockfill modulus on the confinement stress, was proposed:

$$E=E_0\left(\frac{\sigma_{oct}}{\sigma_{oct\ i}}\right)^{\alpha} \tag{3.1.1}$$

Here, σ_{oct} is the octahedral stress for any dam construction stage and $\sigma_{oct\ i}$ corresponds to the minimum confinement octahedral stress used in laboratory tests carried out on several rockfill types that were used to build the above mentioned dams; α is a fitting parameter, which depends on the type of rockfill material and stress trajectories yielded by the rather uneven dam construction. It is worth to mention that in practice these rather random stress trajectories are accounted for considering a large number of constant confining octahedral stresses to englobe all possible confining stresses; the values of α, which vary from 0.4 to 0.8 for the materials used, were estimated from the results yielded by 0.30 to 1.13 m diameter odometer tests, as those shown in Figure (3.1.1). Finally, E_0 is the rockfill modulus for the lowest normal octahedral confining stress.

Power expressions similar to Eq. (3.1.1) have been proposed by a number of investigators (Janbu, 1963), (Lade & Nelson, 1987). The power expressions proposed by these authors were obtained from a large number of experimental tests in granular soils, and from the development of theoretical considerations involving the principle of energy conservation. In these investigations, the parameters of the power expressions were estimated from conventional triaxial and three-dimensional cubic triaxial tests on soil samples. The results of these tests allow evaluating the exponent α value as a function of the soil type: a value of zero for solid rock, from 0.35 to 0.55 for sands and silty sands with porosities of 35-50 per cent, and 1.0 for wet clay. It is worth to note that the determination of the exponent value for granular soils in these cases is affected by the lack of adequate representation of the stiffness of the boundary conditions considered in triaxial tests, as compared with those existing within the rockfill dam embankment.

The variation of the elastic modulus as a function of the octahedral confinement pressure (σ_{oct}) is depicted for several materials in Figure 3.1.1. These results were obtained from tests on medium and giant odometers (specimens ranging between 0.30 and 1.13 m in diameter) on several rockfill types. It is important to mention that the three curves plotted in Figure 3.1.1 were obtained considering $E_0 = 45$ MPa, a minimum confining pressure of 0.196 MPa, and three values of the α exponent: 0.4, 0.6 and 0.8, to cover the range of possible changes in the rockfill actually used to integrate the 3B material of La Yesca dam.

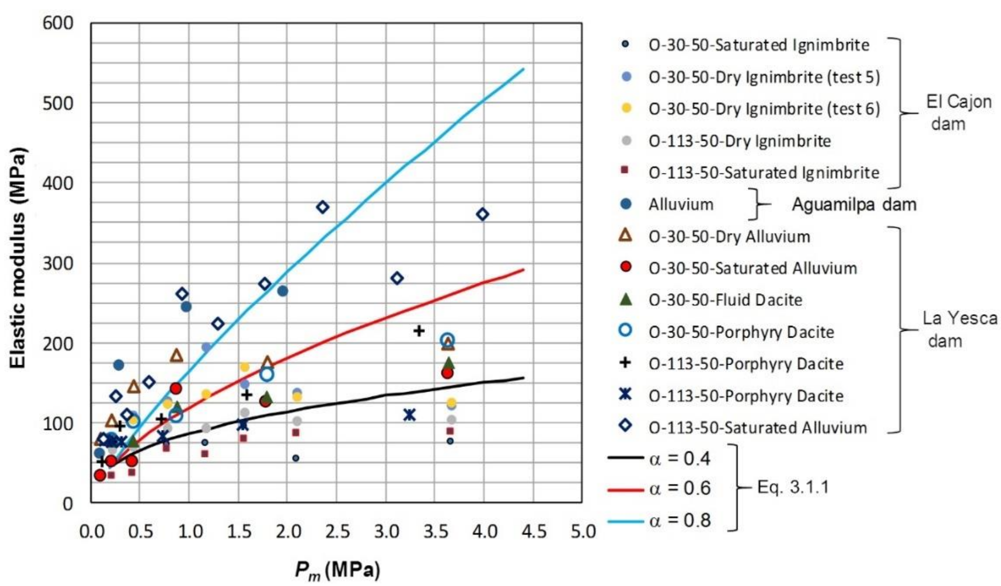

Figure 3.1.1 Mean stress (σ_{oct}) versus elastic modulus obtained from medium and giant odometers tests for various rockfill types. The three curves were computed using Eq. 3.1.1.

It should be noted that the solid curves in Figure 3.1.1 shift upwards or downwards according to the E_0 magnitude, hence the analyses results are highly dependent on the lower normal octahedral stress values. Acceptable E_0 values can be estimated using the unit weight of the rockfill and considering half the thickness of the first dam layer set down. It is important to point out that once the Eq. (3.1.1) is incorporated in the computer code to carry out parametric dam analyses, the spatial variation of the rockfill modulus E is considered automatically since the large number of stress trajectories developed within the embankment during its construction is accounted for in the analyses (Sarmiento & Romo, 2018). It may be argued that this assessment is also valid for the first filling of the reservoir, mainly for CFRDs.

4. Numerical modelling of La Yesca dam

La Yesca dam is located on the Santiago River, upstream of El Cajon dam, at the Nayarit and Jalisco States border; it is a concrete face rockfill dam with maximum height of 210 m and a crest 629 m long. The rockfill embankment is integrated by alluvial materials (3B zone) and sound boulder rocks (T and 3C zones) as schematically depicted in Figure 4.1 Both exterior (upstream and downstream) dam slopes are 1.4H:1V. The transition material (T) has inner slopes of 0.5H:1V; the upstream slope is in contact with rockfill 3B and the downstream slope is in contact with rockfill 3C. The concrete face is the impervious element with approximately 13 m wide slabs, placed on silty sand gravel, and supported by a concrete perimeter foundation known as plinth (Romo, *et. al*, 2002), (Romo, Botero, Méndez, Hernández & Sarmiento, 2007).

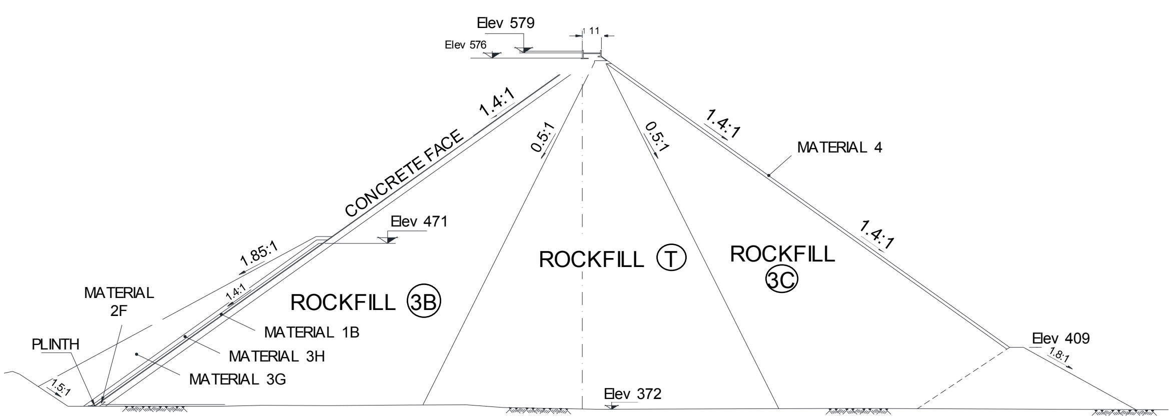

Figure 4.1 La Yesca dam maximum cross section

The three-dimensional numerical model of the dam (Figure 4.2) was analyzed using as platform the finite difference code FLAC3D (Itasca Group Consulting (ICG), 2005). To account automatically for the spatial variation of the dam materials stiffnesses a number of modifications were performed (Romo, *et. al*, 2002), (Romo, Botero, Méndez, Hernández &

Sarmiento, 2007) and (Sarmiento & Romo, 2018). The dam construction was simulated by setting down sequentially several rockfill layers until the total height of the dam was reached. The analyses were carried out assuming the rockfill materials had linear-elastic behaviour (in terms of the octahedral stress-strain relationship), but the rockfill Young's modulus values were updated at each construction stage, using the power law presented in Eq. (3.1.1). The properties of the rockfill materials (3B, T and 3C zones) are included in Table 4.1.

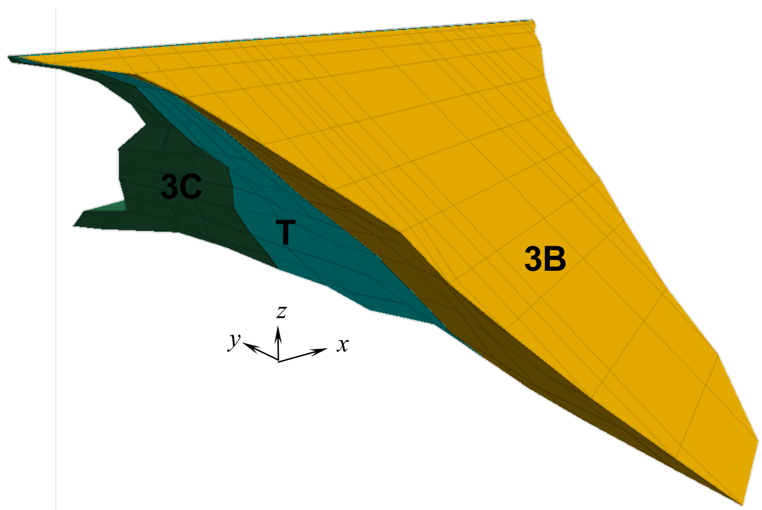

Figure 4.2 Three-dimensional finite differences numerical model of La Yesca dam

Table 4.1 Properties of the rockfill materials; La Yesca dam

Material / Property	γ (kN m^{-3})	E_0 (MPa)	υ	Factor α
Rockfill 3B	22.0	150.0	0.33	0.45
Rockfill T	22.0	86.0	0.33	0.42
Rockfill 3C	21.0	56.0	0.35	0.40

The results of the finite difference numerical analyses in terms of settlement contours (color scale) at the end of the dam construction are shown in Figure 4.3. These numerical results are compared with the geotechnical instrumentation measurements of the CFR dam in terms of curves of equal settlements (continuous lines) elaborated from data yielded by hydraulic levels installed inside the embankment and topographic references (Hernández, Mena &

Perez, 2012). It is seen that the larger settlements on the curtain during its construction were concentrated in the rockfill 3C, which is understandable since the compaction energy used to lay down this material was lower. According to this comparison, it could safely be said that the overall dam settlement is properly reproduced when considering the simple procedure briefly outlined above to obtain the spatial variation of the rockfill stiffness properties, and hence of the stresses and strains.

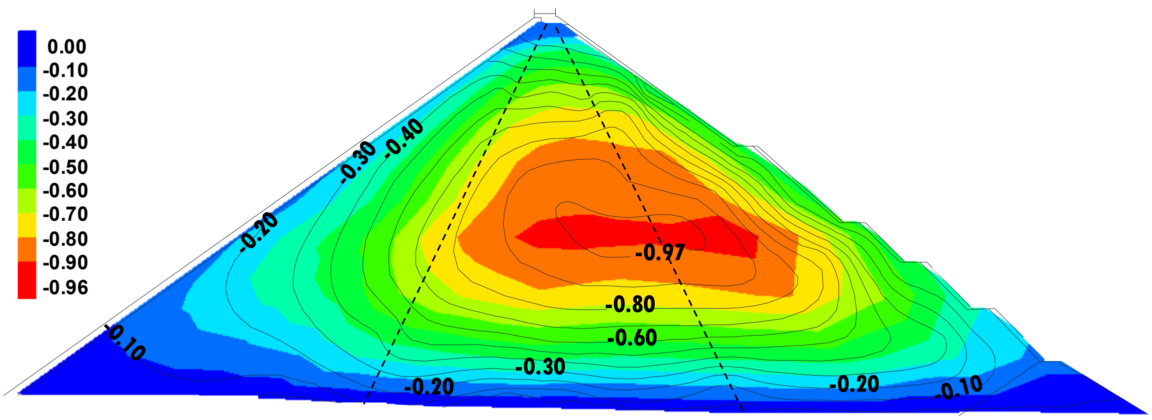

Figura 4.3 Settlement contours measured (solid lines) and calculated (colour scale) at the end of dam construction, all units in m

Finally, it is worthwhile to remark that all the information and comments presented above contradict the laboratory test results obtained from large triaxial chambers (Marsal, 1967), (Marachi, Chan & Seed, 1972) and (Marsal, 1973), which clearly show that the stress-strain relations are nonlinear. This discrepancy can be explained, as mentioned before, in terms of the differences in the boundary conditions in both cases. While the range of confining stresses applied to the samples tested in the large triaxial equipment simulate fairly well the *in situ* stress conditions, the stiffness of the material (a fluid, usually water) that surrounds both the laboratory tested samples and that of the virtual cylindrical samples in the *in situ* dam's rockfill (same rockfill material) are significantly different. While in the former case the shear stiffness is nearly zero in the latter case the shear stiffness is, for all practical purposes, equal to that of the virtual sample, which evidently influences the magnitude of lateral and hence vertical displacements, leading to lower actual lateral and vertical displacements than those computed using laboratory results. Another source of discrepancies among the stress-strain relationships yielded by laboratory tests and field rosettes are the differences among the stress trajectories the laboratory sample and the virtual field sample are subjected to. It must be stressed that in this paper the stress-strain relationship is expressed in terms of the octahedral normal and shear stresses making sure that a better modeling is considered because all developing stresses (and hence stress trajectories) in the field are accounted for.

Also, it should be remarked that the rockfill zoning of the La Yesca dam was first used for building El Cajon dam (Romo, *et. al*, 2002). Given its excellent static behaviour, the international Consultant Barry Cook (colloquially referred to as the "father" of the rockfill concrete face dams) asked the Federal Commission of Electricity its permission (considering the innovation involved in the overall embankment conception, which more often than not led to less expensive and safer structures) to propose it as possible alternative (whenever the rockfill required was available) to other rockfill dams being under design around the world.

5. Conclusions

The octahedral relationships σ_{oct}-ε_{oct} and τ_{oct}-γ_{oct} obtained from the rosette instrumentation placed in the rockfill materials of the Aguamilpa dam and a number of rockfill dams in Mexico clearly show that these materials have a linear relationship throughout the construction of the dams. The results from the *in situ* instrumentation and from odometer tests in the laboratory of the rockfill materials, used in Aguamilpa, El Cajon and La Yesca dams lead to the conclusion that the rockfill stiffness modulus, E, is a function of the all-around octahedral stress following a power law as the one presented in this work (Eq. 3.1.1).

From the instrumentation and the numerical results of La Yesca dam analysis, it is evident and henceforth strongly recommended that the numerical computations of rockfill dams must consider the spatial variation of the rockfill modulus E according to the octahedral stress values and a linear-elastic constitutive model. It can be argued that there is no doubt that this is an easy and economical way to account for the dam's rockfill nonlinear behaviour and its spatial variation as its construction proceeds. Furthermore, including a power law to model the variable stiffness of the rockfill upon dam construction, the effects on the computed results of all stress trajectories developing within the dam are automatically taken into account.

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