Analysis of movements of Foz do Areia rockfill dam

L'analyse des mouvements du barrage d'enrochement de Foz do Areia

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SYNOPSIS The present work deals with an analysis of the vertical movements of a rockfill dam with the use of the finite element method and field measurements. Based on this analysis it was possible to propose a simple relationship between modulus of deformation and height of overburden for different zones of the embankment.

INTRODUCTION

During the construction of Foz do Areia rockfill dam, vertical movements of 40 points were monitored allowing evaluation of the performance of the embankment.
The case history presented herein exhibits certain features which recommend some investigation. Foz do Areia dam consists of a 160m high embankment (14.000.000 m^3) of angular basalt rockfill with an impermeable concrete face (130.000 m^2) upstream. Due to high level of stress, vertical movements of 2,6 m were observed in certain zones. The grain size distribution of the rockfill (50% with diameter larger than 10 cm) led the owner of the dam not to perform oedometer tests. Field instrumentation results associated with a finite element analyses made it possible to infer compressibility characteristics of the rockfill and to identify changes in performance due to different energy of compaction.

CASE HISTORY

Foz do Areia rockfill dam located in the southern part of Brazil, 240 km west of the city of Curitiba in the state of Parana, is part of a system of dams on Iguaçu river. During the construction of the dam 2 tunnels with 12 meters diameter and 600 meters long were excavated in order to divert the river (average flow 550m^3/s). The dam has a 830 meters long crest and was designed to retain 6.066 x 10^9 m^3 which inundates an area of 167 km^2. The river valley profile is composed of a 12 meters thick layer of altered rock and crushed rock overlaying 25 to 55 meters of basalt. To monitor rockfill movements, two sections, 100 meters appart, were instrumented with 40 single point settlement gages. Due to the uniform grain size distribution (figure 1) there was a major concern regarding vertical displacements of the embankment (figure 2). Rockfill dams with impermeable face upstream are subjected to hydrostatic water thrust much smaller than vertical load due to selfweight, therefore safety factors against sliding are high. Also slope stability is not critical since shear strength parameters of rockfills are favourable. Embankment displacements, though, can induce unacceptable movements to the impermeable face which can eventually cause failure.

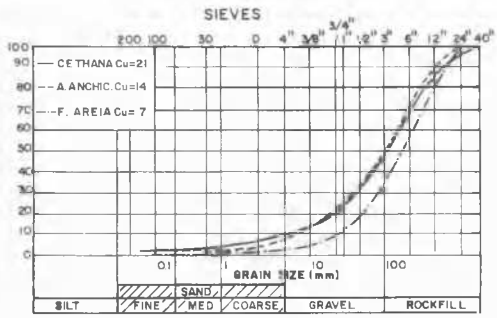
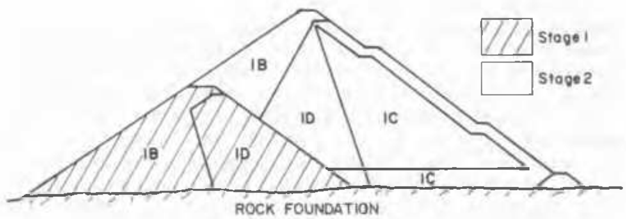


FIG.1 - GRAIN SIZE DISTRIBUTION



CROSS SECTION

Material	Classification	Zone	Compaction
I	BASALT (up to 25% of basaltic breach)	IA	PLACED
		IB	COMPACTED IN 0.80m LAYERS 4 PASSES VIBRATING ROLLER (10TONS) 25% WATER
		IC	COMPACTED IN 1.6m LAYERS 4 PASSES VIBRATING ROLLER (10 TONS) 25% WATER
	BASALT and BASALTIC BREACH	ID	COMPACTED IN 0.8m LAYERS 4 PASSES VIBRATING ROLLER (10 TONS) 25% WATER

FIG.2- EMBANKMENT ZONING

ROCKFILL PROPERTIES

In order to test rockfill samples with large diameters with meaningfull results one should have testing apparatus with unconventional size. Holtz and Gibbs(1956) propose cell diameters at least 6 times larger the maximum diameter of the sample to obtain shear strength properties. To overcome this problem Zeller and Wulliman (1956) suggest the testing of sample with diameters smaller than 10 cm and the establishment of some correlation with the full sample. Rockfill material exhibit certain features which are significantly different from gravel. Angle of friction of rockfill decreases with the increase of the grain size; this can be due to the greater probability of occurrence of frature in large grains. Another useful approach to evaluate angle of friction of rockfill was proposed by Barton and Kjaernsli (1981) based on particle size (d50), uniaxial compressive strength of the rock, origin, roundedness, porosity and basic angle of shear strength.
The determination of deformation characteristics is even more dependent on the particle size, and scale effects are difficult to be considered. Under a vertical stress corresponding to an overburden of 6 meters, contact forces in sand of 0.2 mm diameter are about 0.018N while for rockfill of 200 mm diameter the contact forces are around 4000N (Marsal 1963, Penman 1971). These high values of contact forces crush the contacts until it reaches the strength of the parent rock. Compression is caused by crushing and rearrangement of the grains. To reduce compression the rockfill should have a wide range of grain size to reduce the possibility of rearrangement as well as the contact forces.Well rounded rockfill also decreases the possibility of crushing. Penman (1971) proposed expression (1) which incorporates the grain size effect.

$$\varepsilon = c\sigma^n \tag{1}$$

ε - deformation
σ - stress
c - function of particle size and number of particles
n - function of porosity

Fumagalli (1969) through tests using ring diameters and rockfill with different diameters developed a similar relationship which enables one to predict settlement of embankments based on confined compression tests at a reduced geometric scale. It is though necessary to perform the tests with the same degree of initial consolidation; slight deviations can produce considerable differences (Fumagalli, 1969). The determination of the degree of initial consolidation at the design stage is very delicate. When one has the task to design high rockfills, as the case under investigation, it is highly recommended to evaluate compressibility characteristics of the embankment based on field measurements.
The procedure to be pursued in the present contribution consists in using field measurements at the end of construction, to infere deformation characteristics of the rockfill throughout the whole construction period.
If one stress strain law is found to be able to represent the field condition, the overall performance can be re-evaluated with more confidence.

ANALYSIS

The deformation characteristics being searched are modulus of deformation (E) and Poisson's ratio (μ), for each point during construction. Assuming the axisymmetric state of stress with lateral confinement predominant in the embankment and the validity of the theory of elasticity expressions (2) and (3).

$$Ko = \frac{\mu}{1-\mu} \tag{2}$$

$$Ko = 1 - \sin\phi' \tag{3}$$

lead to expression (4)

$$\mu = \frac{1-\sin\phi'}{2-\sin\phi'} \tag{4}$$

Considering the characteristics of the rockfill and the high stress level, $\phi' = 35º$ was considered a good estimate to evaluate Poisson's ratio(0.3). For the sake of simplicity, Poisson's ratio will be kept constant throughout the analyses; this assumption should be re-evaluated latter.
The vertical stresses in the embankment can be estimated with the use of expression (5)(Poulos et al, 1972).

$$\sigma_v = \gamma h \, I \tag{5}$$

h - heigth of overburden
γ - unit weight
I - function of μ and geometry

For the assumed confined compression state of stress the modulus of deformation can be calculated by (6)

$$E = \frac{\gamma h I}{\varepsilon_v} \; \frac{1-\mu-2\mu^2}{1-\mu} \tag{6}$$

ε_v - vertical deformation (field)

Based on equation (6) E values were calculated at the end of construction and tabled against height of overburden.
From the zoning of the dam (figure 2) it seems reasonable to look for different deformation characteristics at stages 1 and 2. A relationship between overburden pressure and modulus of deformation was searched. After attempting linear, exponential, hyperbolic and parabolic relationships, it was concluded the parabolic relationship was more adequate (figure 3). It should be pointed out though, results obtained from equation (6) indicate a trend on the opposite direction of what was originally expected;

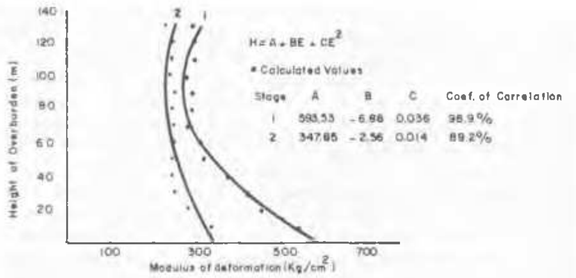


FIG.3- MODULUS OF DEFORMATION (E) VS HEIGHT (H)

modulus of deformation is decreasing with overburden pressure. A finite element analyses was then performed using this relationship. Based on previous experiences (IPT, 1976), it is reasonable to assume the foundation 500 1000 times stiffer than the rockfill which according to Kulhawy (1977) enables one to assume the foundation as rigid. To confirm this assumption, Penman (1971) during the analyses of Scammonden Dam in which the ratio between modulus of deformation of the foundation and the rockfill was 5 to 25, encountered a ratio of deformation between the rockfill and the foundation to be around 8 . Regarding the interface, embankment/foundation it was not clear at the early stages of the analyses which assumption to make about the interface. Assuming fixed end support condition, it became clear mobilized shear stress at the interface was much lower than the available shear strength. Normal stresses were high enough to build up shear strength at the base. Only in a small region near the ends, where the overburden pressure was reduced, the shear stress overcame available shear strength (figure 4). To evaluate the influence of the interface, one run was performed assuming free end support at the base, which indicated stress paths significantly different from the usual assumption of confined compression state of stress (Eisenstein and Law, 1979) and the existence of widespread failure zones (figure 5). It is interesting to observe that even for points close to the slopes, the stress path does not depart significantly from the Ko line.

The simulation of the construction procedure was performed in 8 increments as indicated in figure 6. Increments 1 to 3 are in the first stage while increments 4 to 8 are in the second stage. Each stage uses a different relationship between modulus of deformation and overburden pressure as mentioned previously.

A comparison between measured and calculated vertical displacement is indicated in figure 7, where it can be observed a reasonably good prediction; no significant differences were encountered for other phases of construction . Predictions are closer to reality in the first stage which can be accounted for the best coefficient of correlation in this material. Other aspects of the analyses will now be explored. Figure 8 indicates contours of equal displacements, where it can be observed larger movements in the second stage for points symmetricaly located from the axes of the dam. It is interesting to notice that points near the downstream slope at stage 1 move downstream while they move upstream during the construction of the second stage. Considering the analyses was concentrated on the determination of the modulus of deformation, keeping a constant Poisson's ratio, a parametric study was performed allowing μ to change from 0.2 to 0.35.

It was found, horizontal movements are much more sensible to changes in μ than vertical movements. Considering the predominancy of vertical over horizontal movements, and the range of investigated μ, it seems correct to state one should have much more prudence to evaluate modulus of deformation when analysing movements of embankments.

One of the most interesting aspects of this analyses is with regard to the relationship between modulus of deformation and overburden pressure. The whole analyses was performed based on decreasing values of modulus of deformation with overburden pressure up to a limiting value beyond which the rockfill became stiffer with overburden pressure. The hypothesis of precompression of the rockfill during compaction as suggested by de Mello (1981) seems appropriate . The high energy of compaction produces high initial values of the modulus of deformation which decreases with stress level. When the embankment imposes level of stresses comparable

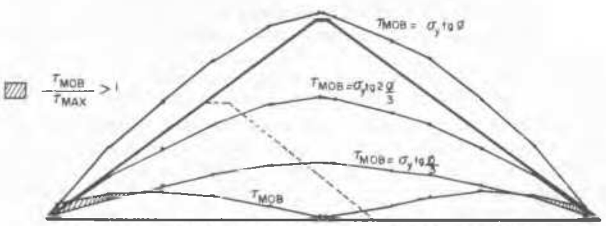


FIG.4- MOBILIZED SHEAR STRESS AT THE ROCKFILL/FOUNDATION INTERFACE

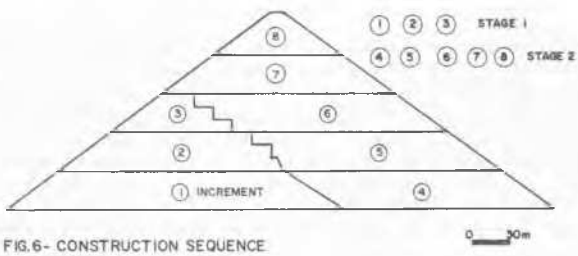


FIG.6- CONSTRUCTION SEQUENCE

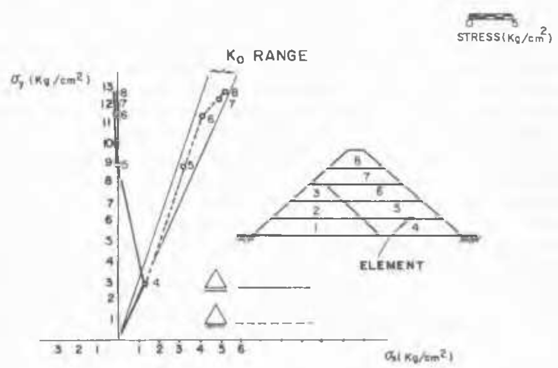


FIG.5- STRESS PATHS

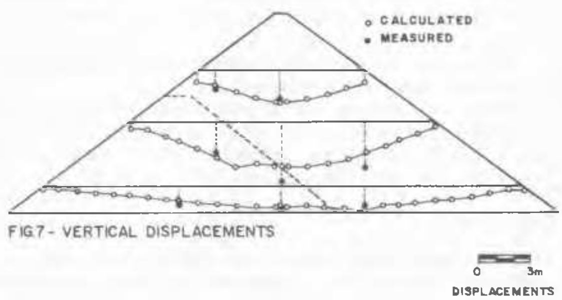


FIG.7- VERTICAL DISPLACEMENTS

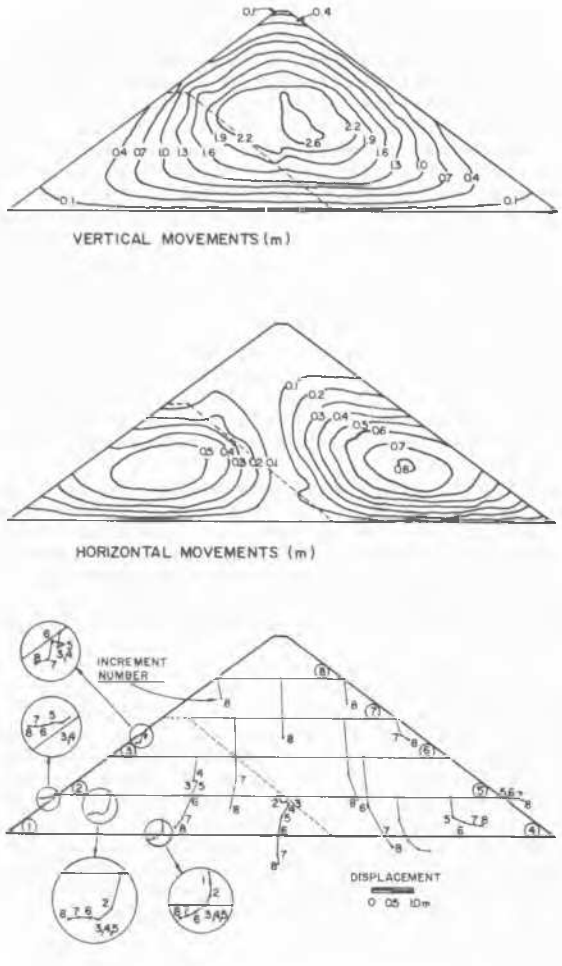


FIG.8 - EMBANKMENT DISPLACEMENTS

to the ones obtained during the compaction, re - orientation of the grains and crushing takes place giving rise to stiffening of the embankment. Rockfill of the second stage was compacted with thicker layers which resulted in a less stiff material and did not exhibit a pronounced peak value of modulus of deformation. It seems the deformation characteristics of the rockfill in stage 2 could be considered as stress independent.

CONCLUSION

Field observation of vertical movements of a rockfill dam associated with finite element analyses allowed to establish a simple relationship between modulus of deformation and height of overburden.
High energy of compaction can reduce movements of rockfill through precompression.
Proportional loading is a reasonable assumption for embankments even for points close to the slopes, since slippage between the rockfill and the foundation is prevented.

REFERENCES

Barton, N. and Kjaernsli, B. (1981). Shear Strength of Rockfill. Journal of the Geotechnical Engineering Division, ASCE, vol. 107, GT7, July 1981, pp. 873-891.

De Mello, V.B.F. (1981). As Lições de Foz do Areia para Barragens Altas. Revista Técnica e Materiais.

Eisenstein, Z. and Law, S.T.C. (1979). The Role of Constitutive Laws in Analysis of Embankments. 3rd Int. Conf. Numerical Methods in Geomech., Aachen, April, 1979.

Fummagalli, E. (1969). Tests on Cohesionless Materials for Rockfill Dams. Journal of the Soil Mechanics Division, ASCE, vol. 95, SM1, pp.313-330.

Holtz, N.M. and Gibbs, H.I. (1956). Triaxial Shear Tests on Previous Gravelly Soil. Proc. of 6th ICSMFE, vol. II.

IPT (1976). Stress and Deformation Analyses of Foz do Areia Rockfill Dam. Internal report nº. 9227, Instituto de Pesquisas Tecnológicas, São Paulo (In Portuguese).

Kulhawy, K. (1977). Embankment and Excavations. Numerical Methods in Geotechnical Engineering, MacGraw Hill, 1977.

Marsal, R.J. (1963). Contact Forces in Soils and Rockfill Materials. Proc. Pan-American Conf. on Soil Mech. and Found. Eng., vol. 2, pp. 67-97.

Penman, A.; Burland, J. and Charles, J.A. (1971). Embankment During Construction. Proceedings of Institution of Civil Engineers, vol. 49, pp. 1-21, May, 1971.

Penman, A. and Charles, J.A. (1972).Constructional Deformations in a Rockfill Dam, Journal ASCE, vol. 98, SM2, pp. 139-163.

Penman, A. (1971). Rockfill Building Research Station. Current Paper 15/71.

Poulos, H.D.;Brooker, J.R. and Ring, G.J. (1972). Simplified Calculation of Embankment Deformations. Soils and Foundations, vol. 12, nº 4, Japanese Society of Soil Mechanics and Foundation Eng., December, 1972.

Rossi,M.N. (1983). Foz do Areia. Backanalysis by FEM. M.Sc. Thesis. Catholic University of Rio de Janeiro, Brazil (In Portuguese).

Zeller, J. and Wulliman, R. (1956). The Shear Strength of the Shell Materials for the Goschenenalp Dam. Proc. 6th ICSMFE, vol. III.