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Application of the Newmark method to the results obtained from a dynamic finite element analysis of an earth dam

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ABSTRACT: Since Newmark published, in 1965, a method for estimating the permanent displacement that can be expected in the crest of an earthdam subjected to earthquake loadings, that topic has been studied by different investigators. The paper shows the results of in situ tests (seismic refraction and down-hole) and laboratory measurements (resonant column tests and cyclic triaxial tests) carried out to characterize the dynamic geotechnical parameters of a 155 m high earthdam. First, the results obtained incorporating those parameters in a dynamic Finite Element analysis of the dam are given. Then, using Newmark's method basic hypothesis, the acceleration time hystories obtained at different points of the dam body are integrated twice in order to get permanent displacements. Finally maps showing the distribution of expected permanent displacements in relation with seismic yield coefficients over the whole dam are presented.

INTRODUCTION

Today it is very common to use the Finite Element method together with the linear equivalent constitutive model to carry out in the frequency domaine the integration of the equations that govern the behaviour of earthdams under earthquake loadings.

One of the greatest drawbacks to the procedure arises from the difficulties that exist in deducing the permanent deformations at the crest of the dam. A simple linear constitutive model does not allow to assess directly those displacements, and one must resort to the results of laboratory tests to estimate the potential strain field of the dam.

By contrast, the Newmark method (1965) makes it possible to estimate the permanent strains at the crest in a very simple way, but on the basis of certain assumptions which, in the light of the highly complex nature of the dynamic behaviour of an earth dam, can be regarded as oversimplifications.

The aim of this article is to show that it is possible to combine the two methods, in such a way as to apply the hypothesis originally devised by Newmark, to the results obtained using the Finite Elements technique together with the linear equivalent model.

By way of example, this overlapping of the two procedures is applied to the analysis of the Canales Dam which, being close to the city of Granada, lies in the heart of one of the most seismic areas in Spain.

BASIC DATA CONCERNING THE DAM

General Properties

The dam, with a high of 155 m, is 300 m long at the crest and has a slightly curved layout in plan. It has been founded in a relatively narrow valley. Figure 1, shows its situation in the Iberian Peninsula.

The cross-section of the embankment consists of three different kinds of materials; the clay core, the transition zone made up of a mixture of coarse sand and fine gravel, which being known locally as kakerita is the result of the mylonitization of a Miocene limestone, and the outer rockfill shell.

The outer slopes are of 1.7H:1V downstream and 1.8H : 1V upstream. Figure 2 shows schematically the cross-section of the dam.

The foundation materials are of two types, depending on their situation within the site. An extremely compact marly limolite, and above a specific elevation, a resistant calcareous sandstone. Figure 3 shows a longitudinal section.

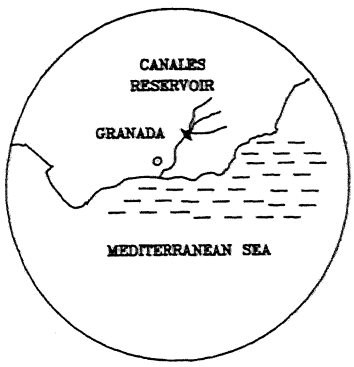


FIG. N. 1 Dam's site

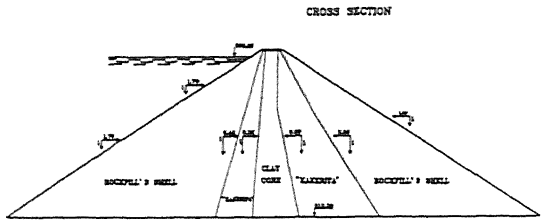


FIG.2. SCHEMATIC CANALES DAM MAXIMUM SECTION

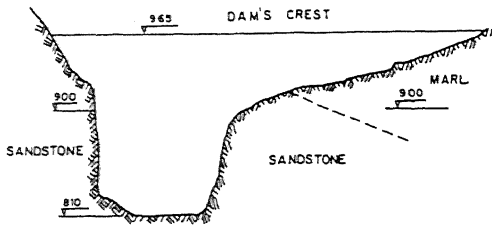
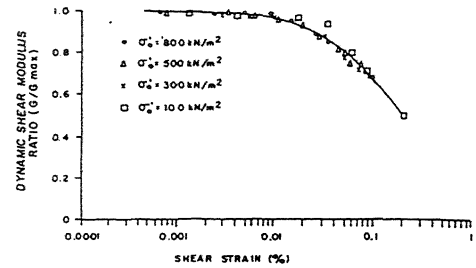


FIG 3.- LONGITUDINAL PROFILE OF THE CANALES DAM



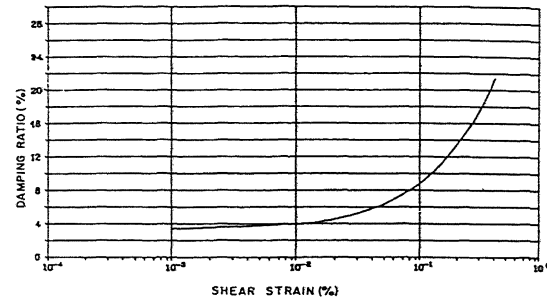


FIG.4.- DYNAMIC SHEAR MODULUS AND DAMPING CHARACTERISTICS (CLAY CORE)

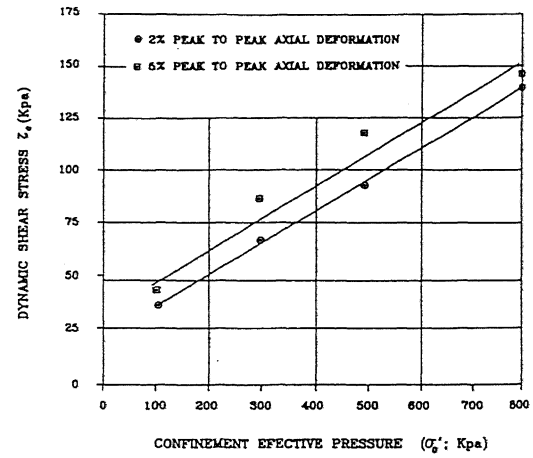


FIG.5. DYNAMIC SHEAR STRENGH OF THE CLAY CORE

Characteristics of the Materials

The following Table shows the most representative parameters of the clay core of the dam.

CLAY CORE	RANGE OF REPRESENTATIVE VALUES
Liquid Limit (%)	55-65
Plasticity Index (%)	30-35
Max.lab.dry density (P.N.) KN/m³	15.0-16.5
Optimum moisture (%)	17-22
Carbonates (%)	15-25
Sulphates (%)	0,5-10
Cohesion (Kpa)	10-20
Angle of internal friction (°)	19°-23°

Data provided by seismic refraction and down-hole tests have been used to assess the value of G_{max}. A shear wave propagation velocity of 225 m/seg. was obtained at a depth of 5 m.

The variations of shear modulus ratio G/G_{max} and damping ratio, D, with shear strain were obtained by means of resonant column tests carried out with samples isotropically consolidated under pressures of 100, 300, 500 and 800 KN/m² (see Fig. 4).

Fig. 5, shows the shear stress levels that cause axial strain between peaks of 2% and 5%, for 30 cycles. These values have been obtained from cyclic triaxial tests carried out on samples isotropically consolidated under different confining pressures.

The most characteristic parameters of the material constituting the transition zone between the core and the outer shells, and known as **"kakerita"**, can be seen in the table below.

KAKERITA	RANGE OF REPRESENTATIVE VALUES
Maximun size (mm)	20
Effective diameter (D_{10})	0.20
Average diameter (D_{50})	2
Maximum dry density (KN/m³)	21.5-2.25
Optimum moisture (%)	2-3
Cohesion (Kpa; effective)	0
Angle of internal friction (°)	40°-43°

Shear wave velocities of the order of 280 m/seg. were obtained in down-hole tests. Fig. 6 shows, for this material, the shear modulus ratio and the damping ratio versus shear strain curves obtained also with the resonant column.

The dynamic triaxial equipment was used to quantify the cyclic resistance of the material. Different stress controlled tests were carried out under a variety of isotropic and anisotropic consolidation conditions.

The amplitude of cyclic shear stress (τ_c) required to cause a peak to peak axial strain of 5%, for 30 cycles, depends largely on the stress ratio $K_c = \sigma_1/\sigma_3$ imposed to the sample as can be seen in Fig 7.

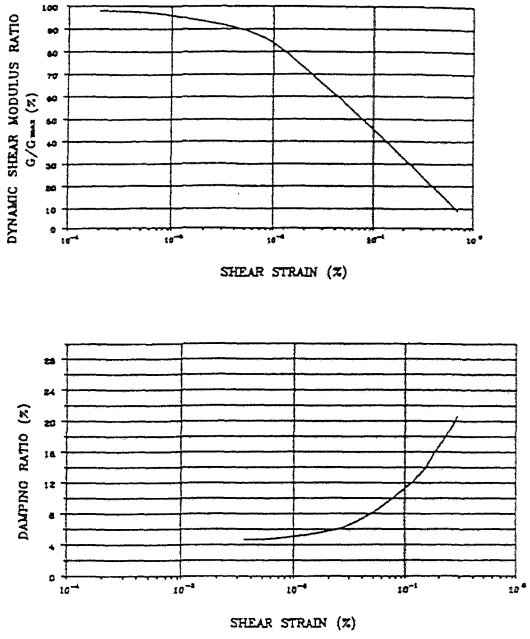


FIG.6. DYNAMIC SHEAR MODULUS AND DAMPING CHARACTERISTICS (KAKERITA)

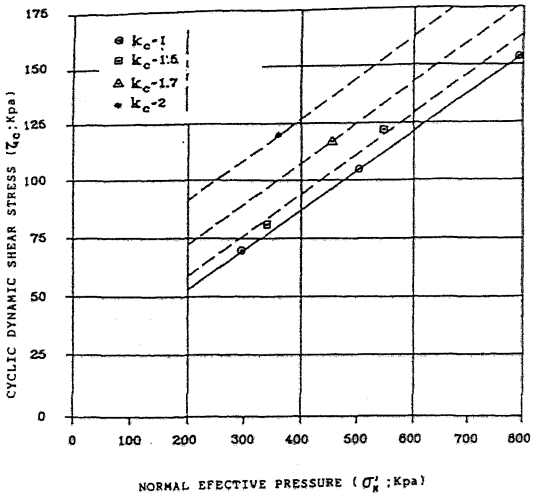


Fig. 7. KAKERITA S DYNAMIC SHEAR STRENGH (FOR A 5% PEAK TO PEAK AXIAL STRAIN DEFORMATION)

SEISMIC INPUTS

The input design motions, accelerograms, have been obtained by carrying out a specific seismic risk analysis, and making a distinction between two types of earthquake, one of which corresponds to the near field motion and the other to the far field motion. The horizontal and vertical scales of the accelerograms have been modified from time histories recorded in places which have basically similar seismic characteristics to the dam area.

The horizontal component SW-NE recorded "in rock" at the El Centro Station, Imperial Valley, California, in 1940, for the near field motion, has been suitably scaled.

The horizontal component N.21.E, recorded "in rock" at the Taft Station in Kern County in 1952 for the far field motion, has also been scaled.

The Table featured below, shows the basic parameters of both earthquakes.

EARTHQUAKE GENERAL DATA	NEAR FIELD	FAR FIELD
Local magnitude	6	7
Max. Horizontal Acceleration	0.45 g	0.20 g
Max. Vertical Acceleration	0.30 g	0.13 g
Total duration	26 sec.	55 sec.
Epicentral distance	20 Km	39 km

DYNAMIC ANALYSIS

The calculation of the acceleration, stress and strain histories throughout all the dam, has been based on the stress state corresponding to the service period of the dam. The distribution of static stresses has been obtained using hiperbolic models with the Finite Element method. The dynamic behaviour of the dam has been analysed assuming linear equivalent models for the different materials.

The QUAD IV program has been used to estimate the acceleration and shear stress histories induced by the two seismic inputs.

APPLICATION OF THE NEWMARK METHOD

General Approach

Various procedures have been published in technical reviews, and they all attempt to go a stage further in using the "potential strains" values, determined from laboratory tests together with the results of Finite Elements calculations (Serff 1976).

The procedure developed in this article, applies theoretical concepts outlined by Newmark in the 1965 Rankine Lecture. In this work, Newmark hypothesis are applied to all the acceleration time histories obtained, for each and every one of the finite elements that form the grid that defines the cross-section calculated.

Thus, instead of assuming that a rigid solid affected by one single accelerogram is being dealt with as was suggested by Newmark in his original text, each one of the finite elements is being taken to behave individually.

The theoretical sliding surfaces would be represented by every one of the acceleration time histories for each of the finite elements.

Integration for Several Yield Accelerations

The double integration of the acceleration time histories has been carried out for different yield acceleration values. To give an idea of the range of the yield values considered, the Table below shows the yield accelerations obtained for different sliding surface heights using the pseudostatic method to carry out limit equilibriums analysis.

Height of sliding slope	Yield acceleration
30 m	0.28 g
60 m	0.26 g
150 m	0.25 g

By way of example, Fig. 8 shows the acceleration time history at element nº 4, which was obtained from calculation using the QUAD IV program, for the 55 seconds that the far field earthquake lasted. This element nº 4 lies close to the dam crest.

Fig. 9 shows the velocity time history for a yield acceleration of 0.20 g, once the first integration was carried out. Based on this, the evolution of the displacement time history for the same element nº 4 is obtained through a new integration. The displacement time history can be seen in Fig. 10.

In this specific case, the final displacement is 85.7 cm.

Note: The value 0.20 g is an extreme and theoretical one, not related directly with Canales slope satbility results. This most pesimistic hypothesis has been considered only for illustration purposes.

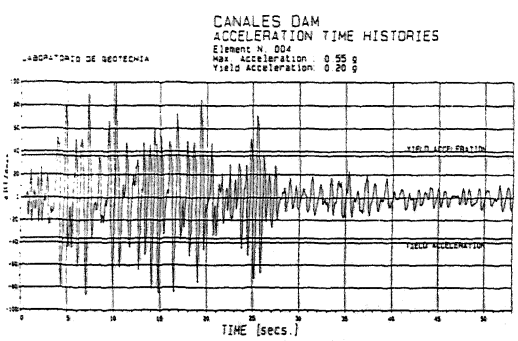


Figure 8

Permanent Deformation Maps

Fig. 11, shows the isolines of the permanent displacements map obtained for a yield acceleration value of 0.30 g with the far field ground motion, by using the same procedure for all the elements.

Likewise, Figures 12 and 13 show the permanent deformation value maps for yield accelerations of 0.26 g and 0.25 g, respectively.

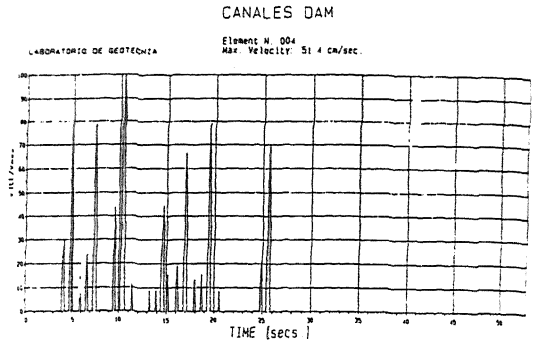


Figure 9

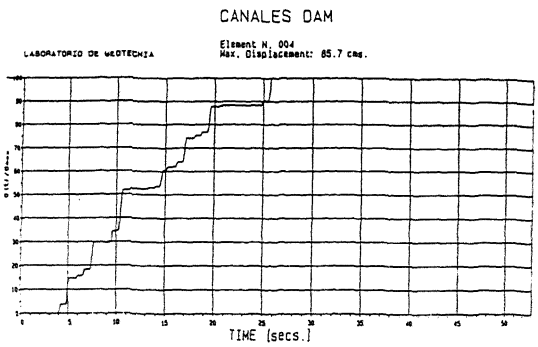


Figure 10

An analysis of the effects of the near field earthquake has been carried out using the same procedure.

Interpretation of the Results

These residual displacement maps serve to show the (irreversible) movements that would take place for all sliding surface heights, each of these being represented by the value of the yield acceleration that identifies it.

So, by way of example, for the 25 m slope, it's yield acceleration value is 0.30 g. The maximum values to be deduced are about 20-25 cm for the accelerograms associated with the finite elements close to the crest. However, they would be about 7.5 cm for the accelerograms calculated for elements lying near to the toe of the slope.

Given that an accelerogram which is representative of the whole sliding mass, of a 25 m sliding surface, would be between both extremes, it can be assumed that the permanent displacement of a sliding surface with a height of 25 m would be about 15 cm. (This value has been selected among others obtained by different procedures; average value of the area involved, average weighted value, etc.).

In the same way, it can be shown that for sliding surface depths of 60 and 150 m residual displacements 20 cm and 15 cm respectively are pertinent.

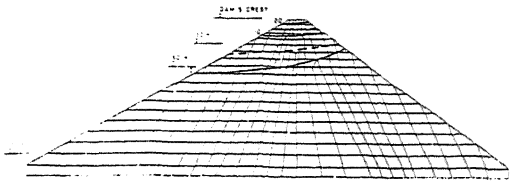


Figure 11 Estimation of the permanent crest displacement, under a yield acceleration value of 0.30 g.

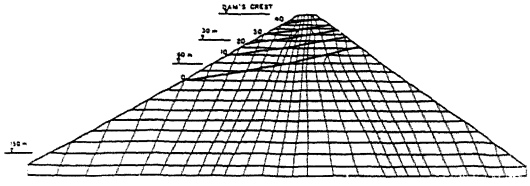


Figure 12 Estimation of the permanent crest displacement, under a yield acceleration value of 0.25 g.

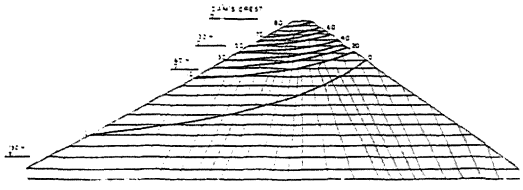


Figure 13 Estimation of the permanent crest displacement, under a yield acceleration value of 0.20 g.

CONCLUSIONS

In the light of the results so far commented, it seems that the application of Newmark's methodology to the results of the analysis of the dynamic behaviour of an earthdam by the Finite Elements method gives residual displacements that can be regarded as reasonable and correct.

When this particular method is used for the dynamic calculation of the Canales Dam, the results show that, under certain hypothesis, permanent displacement ranges of between 15 and 20 cm can be expected in the dam crest.

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