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# Seismic analysis of Melado dam

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ABSTRACT: Melado dam is a 90 m - high earthfill structure composed of coarse-gravel shoulders and a clayey-gravel core, located in Chile, about 320 km South of Santiago. A cast-in-place concrete wall, 1.0 m thick and embedded 0.50 m into bedrock, was employed as seepage cutoff in the foundation soil. The purpose of the present analysis is to predict the behavior of this dam, during the ocurrence of a seismic event of high magnitude.
A two-dimensional finite element analysis using the FEADAM and QUAD-4 computer programs was carried out. Non-linear stress-strain relationships for the dam materials were obtained from triaxial tests.
Numerical results show that a good seismic performance of this dam can be expected. In this paper special emphasis is placed in the examination of seismic amplification aspects.

## 1 INTRODUCTION

The construction of Melado dam was finished at the end of 1990.

This dam has a maximum height of 90 m above the Melado river bed and a length of 310 m between abutments at its crest, i.e., the crest length to height ratio is only 3.44. In addition, the topography of the bedrock at the Melado-dam canyon presents some longitudinal irregularities. However, for simplifying purposes, no three-dimensional effect was herein considered.

Thus, a two-dimensional finite element model was carried out. The analyzed cross section of the dam is located at 180 m of the left abutment (see Figures 1 and 2). The stress field in the dam and in its foundation for a full reservoir condition, computed using the FEADAM program (Duncan et al. 1980), was published by Rodríguez-Roa et al. (1990). This stress condition was herein considered as the existing one just before the seismic event (see Figure 3).

The results presented in this paper correspond to the analysis performed with the QUAD-4 program (Idriss et al. 1973). Since this dam has been adequately instrumented future measurements will obviously allow to check the accuracy of this numerical prediction.

The earthquake considered in this analysis was the N70E component of the accelerogram record obtained in rock at the Universidad Técnica Federico Santa María, Valparaíso, during the March 3, 1985 earthquake. Following the recommendations of MN-HARZA (1982) the values recorded were scaled to a maximum ground acceleration of 0.25 g. The accelerogram used has a strong motion with a duration of 44.17 sec and a predominant period of 0.15 sec, which is approximately in agreement with the data compiled by Araya and Saragoni (1980).

## 2 DYNAMIC PROPERTIES OF THE DAM MATERIALS

Figure 1 includes a brief description of the different materials that compose the dam. Index properties of the soils considered in the finite alement analysis are summarized in Table 1.

Table 1. Index properties (see figure 1)

| Material | Max size (cm) | Fine content (%) | L.L. | P.L. | P.I. | Specific grav. of solids |
|---|---|---|---|---|---|---|
| 1 | 7.6 | 12.0 | 26.7 | 18.6 | 8.1 | 2.77 |
| 5 | 61.0 | 2.0 | N.P. | N.P. | N.P. | 2.86 |
| 1A | 0.2 | 92.0 | 44.5 | 20.3 | 24.2 | 2.75 |
| 8 | >61.0 | 2.0 | 23.8 | 17.9 | 5.9 | 2.77 |

Note:
L.L.= Liquid Limit; P.L. = Plastic Limit;
P.I. = Plasticity Index; N.P. = Non plastic

A field relative density larger than 85 per cent was required for the shoulders of the dam, and a dry density of 95 per cent of the Modified Proctor maximum density was considered appropriate for the core.

Furthermore, a dry density equal to 90 per cent of Standard Proctor density was specified for material 1A (see Figure 1) in order to get a high compressibility soil surrounding the top of the concrete cutoff wall. The placement water content used for this material was about the optimum water content plus 3 per cent.

Cyclic undrained triaxial tests were performed on specimens prepared from the fraction of the shoulder material which was finer than 19.1 mm. This portion of the material was separated and recombined to obtain a grain size distribution curve parallel to the gradation

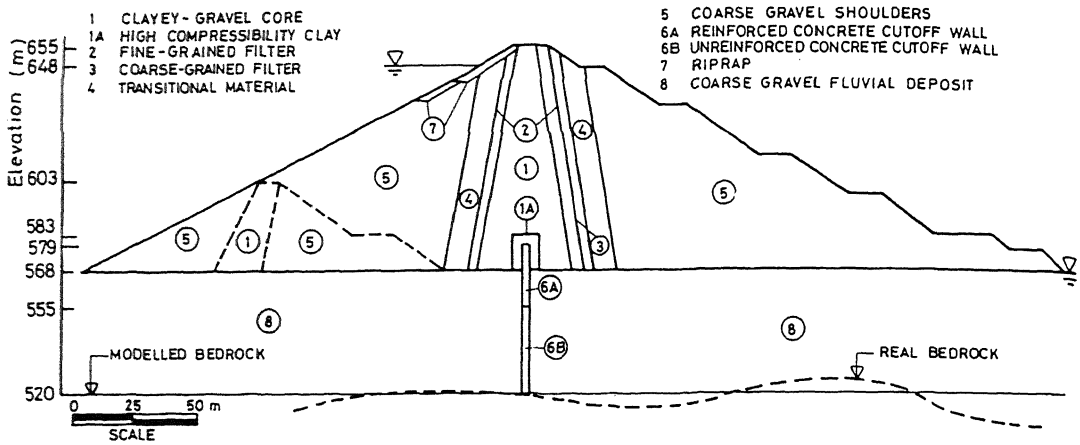


Figure 1. Analyzed cross section of Melado dam

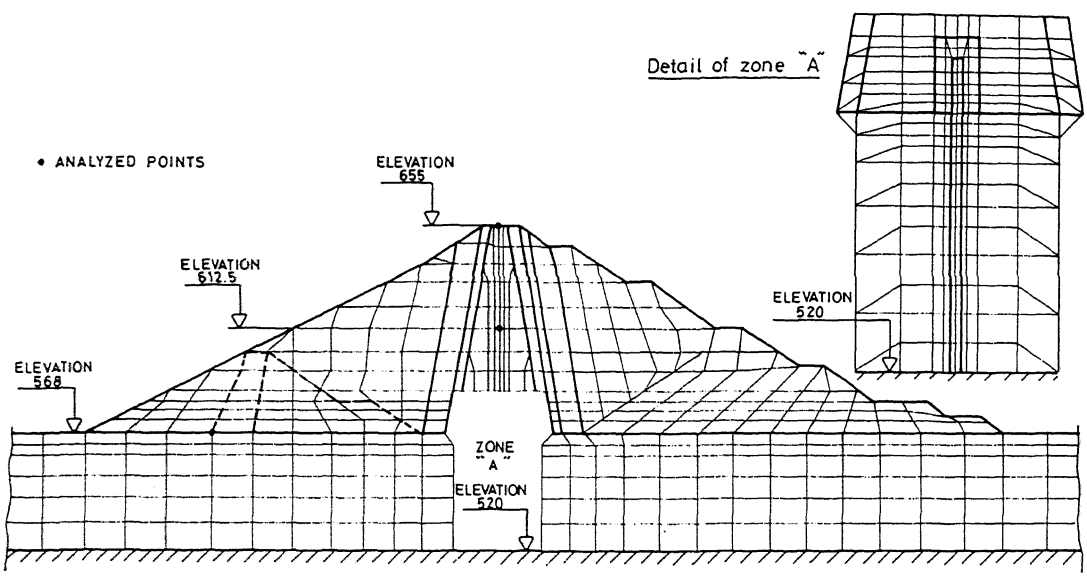


Figure 2. Finite element mesh

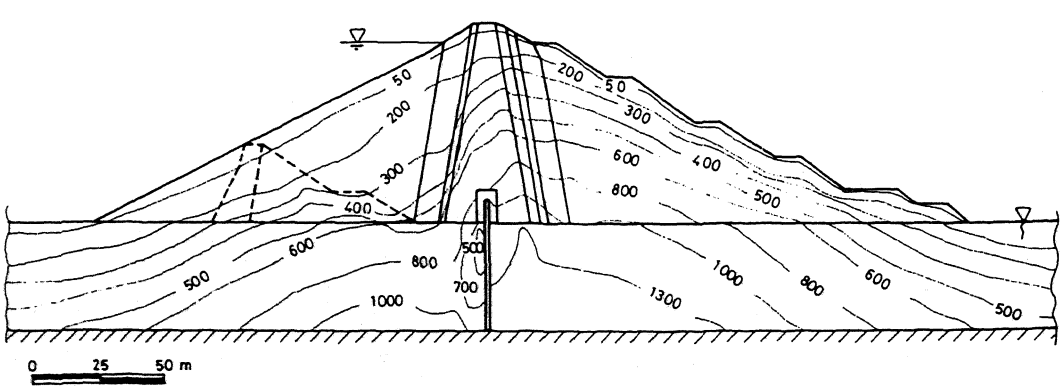


Figure 3. Initial stress field (curves of equal mean principal effective stress, in kPa)

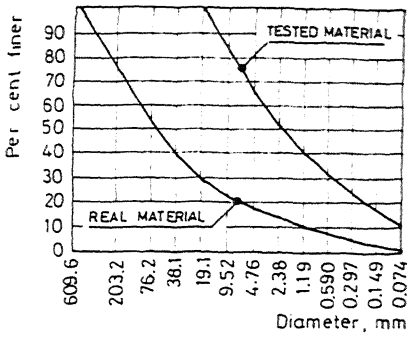


Figure 4. Grain size distributions for the shoulder material

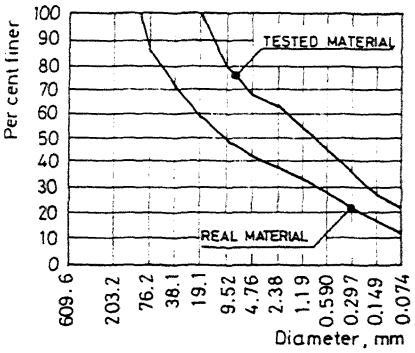


Figure 5. Grain size distributions for the core material

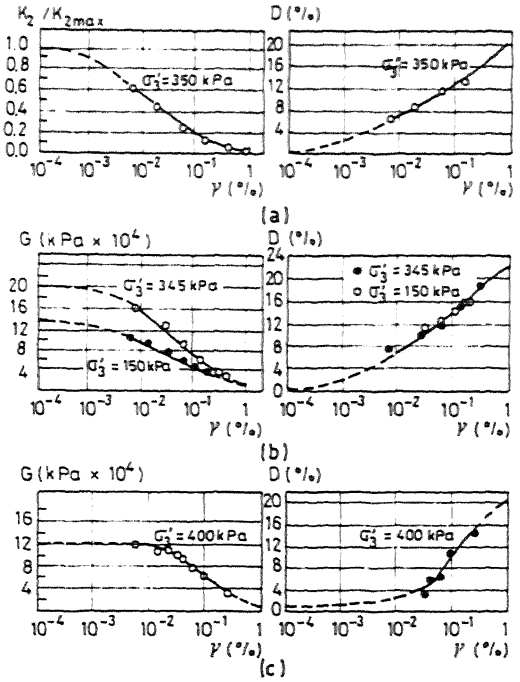


Figure 6. Dynamic properties of the soils:
(a) $k_2/k_{2max}$ vs $\gamma$ and D vs $\gamma$ curves for the shoulder material
(b) G vs $\gamma$ and D vs $\gamma$ curves for the core material
(c) G vs $\gamma$ and D vs $\gamma$ curves for the compressible clay

curve of the real material (see Figure 4). In order to reduce the testing effort the same dynamic properties obtained for this material were used to model the materials 2, 3, 4 and 7 (see Figure 1).

Specimens prepared from the portion of the material 1 which was finer than 19.1 mm were used to determine the properties of the core material (see Figure 5).

The relationship between the shear modulus, G, and the confining pressure, $\sigma'_m$, proposed by Seed and Idriss (1970) for a granular soil is:

$$G = 1{,}000\ K_2(\sigma'_m)^{1/2} \qquad (1)$$

in psf units, so that the variation of G with the strain amplitude can be expressed through its influence on the soil modulus coefficient, $K_2$.

Figure 6a shows the results obtained for the ratio $K_2/(K_2)_{max}$ and for the damping factor, D, in the cyclic triaxial tests performed using 4" diameter-samples composed of the material representative of the dam shoulders. These results were also employed to model the dynamic behaviour of the foundation soil. According to the data presented by Seed et al. (1986), the coefficient $(K_2)_{max}$ was roughly taken equal to 180 for the shoulder material. On the other hand, due to the lack of experimental information concerning the dynamic properties of the dense coarse gravel of the fluvial deposit, a $(K_2)_{max}$ value ranging between 100 and 150 was considered for the foundation soil.

Figure 6b illustrates the measured variation of the shear modulus and damping ratio with shear strain for the core material. From such results it was shown that equation (1) is also valid in this case. The computed $(K_2)_{max}$ value was equal to 49.1.

The relationships indicated in Figure 6c, were used in the model to represent the dynamic behavior of the material 1A (see Figure 1).

Below elevation 555 m, the concrete cutoff wall is unreinforced ($f'_c$ = 6,800 kPa), so this material 6B (see Figure 1) could undergo some structural damage during the earthquake depending on the magnitude of the shear strains. Consequently, it is necessary to consider a ($G/G_{max}$ vs $\gamma$) curve for this material. Such relationship was obtained following the procedure proposed by Rodríguez-Roa et al (1990) (see Figure 7). Only a rough estimate of the damping ratio-shear strain curve was used for the material 6B since the accuracy of this relationship does not significantly influence the numerical results of the model.

A constant shear modulus equal to $10.97 \times 10^6$ kPa, measured with ultrasonic tests, was assumed for the reinforced concrete cuttoff wall (material 6A: $f'_c$ = 19,125 kPa).

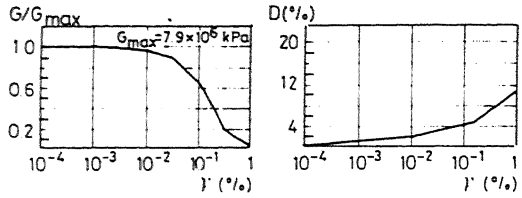


Figure 7. Unreinforced concrete ($f'_c$ = 6,800kPa) (material 6B)

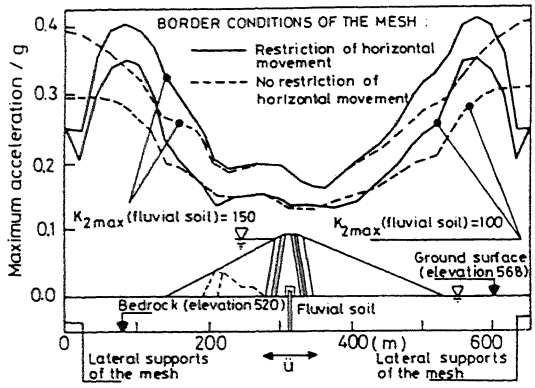


Figure 8. Maximum accelerations at elevation 568 m

## 3 RESULTS OBTAINED

The maximum accelerations induced on the surface of the fluvial deposit (elevation 568, see Figure 8) show a significant variation. The smallest values occur at points under the dam and the highest ones in the "free field" zone, away from the influence of the weight of the dam (Figure 8). If the finite element mesh has lateral supports, restraining the horizontal movement, the maximum acceleration induced at the ground surface reduces to 0.25 g at the borders of the mesh, value that obviously coincides with the maximum acceleration of the earthquake record, because the sides of the mesh move as a rigid body with the bedrock. Figure 8 also shows that the total width of 650 m used for the mesh, based on an estimated length of the seismic waves propagated through the rock (Dibaj and Penzien 1969), was adequate because the accelerations induced under the dam did not vary significantly wichever was the type of lateral support used for the mesh.

Figure 9 shows the maximum accelerations induced on the central axis of the dam, depending on the height or elevation of the point considered. It is seen that the fundamental period of the dam-foundation system, To, decreases as the fluvial deposit becomes more rigid, getting closer to the predominant period of the bedrock motion. Hence, the seismic amplification increases accordingly.

So, the ratio of the maximum crown acceleration over the maximum earthquake acceleration varies from 1.08 to 1.35 as $(K_2)_{max}$ for the fluvial deposit changes from 100 to 150. In other terms, this means that a 50% increase in the stiffness of the foundation soil produces an increase, in this case, of about 25% in the maximum acceleration at the crest of the dam.

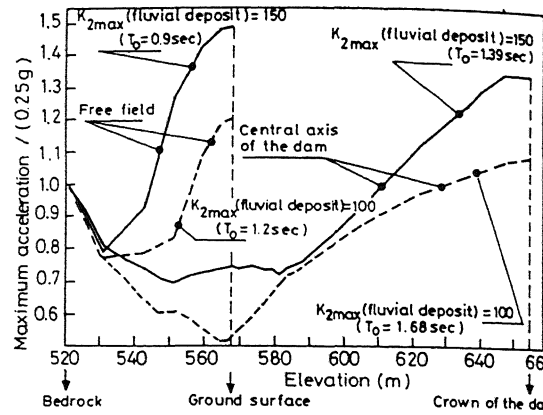


Figure 9. Seismic amplification ratio vs elevation

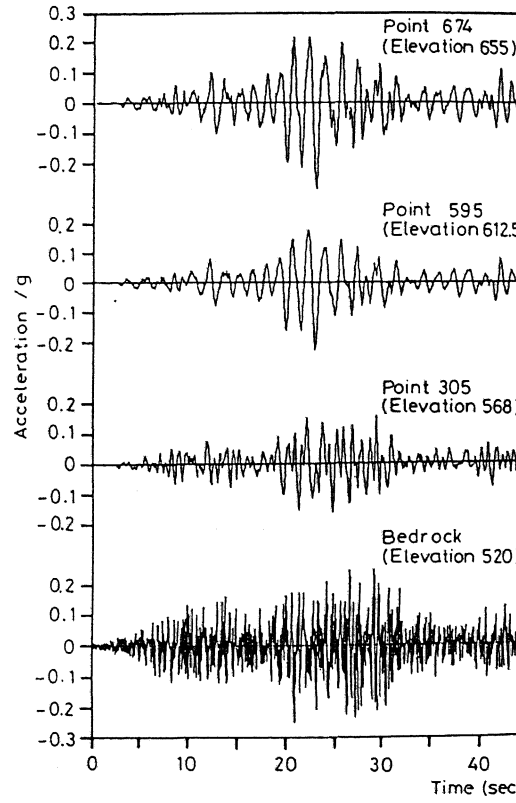


Figure 10. Time histories of accelerations

The study of the site conditions before building the dam, that is, the analysis of the "free field" condition, was also carried out with the QUAD-4 program. From such analysis it was found that the natural period of the

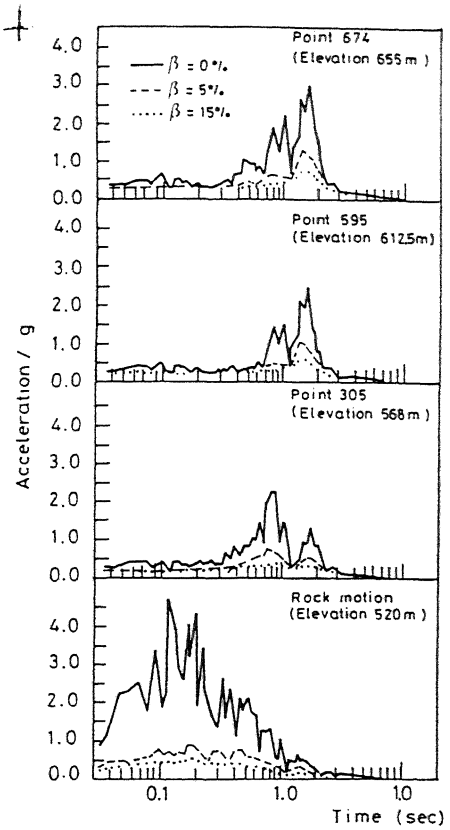


Figure 11. Response spectra for a value of $K_{2max}$=100 for the fluvial deposit

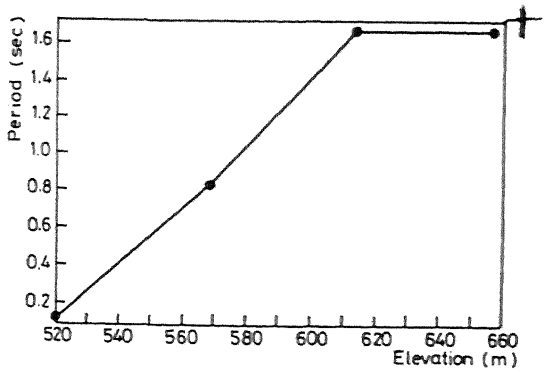


Figure 12. Predominant periods of acceleration time histories

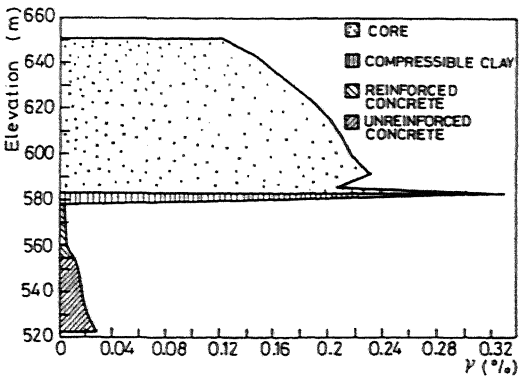


Figure 13. Maximum shear strains at the central axis of the dam

ground is 1.2 and 0.9 sec for values of $(K_2)_{max}$ for the fluvial deposit of 100 and 150 respectively.

The maximum ground surface accelerations produced in these free field conditions varied from 0.30 g to 0.37 g depending on the stiffness of the soil. Such values represent a seismic amplification of 20% and 48%, respectively, of the maximum bedrock acceleration.

Figure 10 shows the acceleration time histories obtained at different heights in the dam, for a value of $(K_2)_{max}$ equal to 100 for the fluvial deposit, and Figure 11 shows their corresponding response spectra for 0.0, 5.0 and 15% of critical damping.

The acceleration time histories generated within the dam clearly show the filtering of the highest frequencies from the applied earthquake. It is seen that the accelerations computed at mid height and in the crest of the dam have almost the same predominant period, which is very close to the fundamental period of 1.68 sec obtained for the dam-foundation system in this case (see Figure 12).

It was also observed that the absolute horizontal displacements computed in different points of the dam increase with their elevation, reaching a maximum value of 19 cm at the crown. Such value remained almost constant for an increase in 50% of the ground stiffness.

The concrete diaphragm registered at its head a maximum horizontal displacement relative to the bedrock of 5.48 cm. The angular distortions induced in the diaphragm increase with depth, reaching a maximum value of 3 x $10^{-2}$ (%) (see Figure 13). Such value represents approximately 23% of the maximum angular distortion produced in a cylindrical sample made of a material with the properties of the concrete below elevation 555 m, and subjected in the laboratory to a loading equal to the unconfined compressive strength "$q_u$" (Rodríguez-Roa et al. 1990).

## 4 CONCLUSIONS

The results obtained confirm, once more, the great influence that the magnitude of the predominant period of the earthquake considered has in the phenomena of seismic amplification. Also, the results show the inapplicability of using seismic records measured on the ground surface, in the free field condition, to determine the seismic response of large dams. For this kind of studies it is necessary to incorporate the foundation soil in the dynamic analysis and to use design earthquakes applied to the bedrock.

With regard to the most convenient lateral support for the finite element mesh, it may be said that the type of support does not significantly affect the results

obtained if the lateral extent of the mesh that includes the foundation soil is large enough. In any case, it is recommended to use lateral supports without restriction of the horizontal displacements in order to carry out the analysis of horizontal seismic movements because, as we have seen, this condition is closer to the physical reality.

The dynamic displacements experienced by the Melado dam are relatively small and they are not very sensitive to the $(K_2)_{max}$ value assumed for the foundation soil.

Also, the maximum value of the angular distortions experienced by the concrete diaphragm would be within the allowable limits. With regard to this point, it is noteworthy to emphasize the damping role played by the high compressibility clay surrounding the top of the diaphragm.

## ACKNOWLEDGEMENTS

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