# Three-dimensional stability of an old dam

## La stabilité tri-dimensionnelle d'un ancien barrage

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SYNOPSIS This paper presents an example of an old earth dam for which three-dimensional (3-D) behavior has a major effect on its seismic stability and shows that under certain conditions which are often encountered in practice, 3-D effects can be very significant and should be considered when evaluating the stability of earth dams. Due to the restraint that the sides of the embankment impose on the dam's center section underlain by weak materials, 3-D stability analyses result in minimum factors of safety which are about 120 percent greater than those computed by conventional 2-D analyses. Neglecting such effects would result in an over-conservative assessment of stability.

## INTRODUCTION

The effects of three-dimensional (3-D) behavior on the stability of earth and rockfill dams have long been recognized and several methods of 3-D stability analysis have been proposed within the past 25 years (Sherard et al., 1963; Anagnosti, 1968; Baligh and Azzouz, 1975; Chen and Chameau, 1982; Dennhardt and Forster, 1985). Previously published results show that the effects of 3-D behavior on the stability of a slope depend mainly on the ratio L/H between the length L and height H of the slope. Typical results for slopes in homogeneous cohesive soils indicate that 3-D effects increase the factor of safety over that computed assuming two-dimensional (2-D) conditions by about 10 to 30 percent for L/H ratios between about 2 and 6. Thus, it would appear from these results that 3-D effects are typically not very significant. The purpose of this paper is to present an example of an old earth dam for which 3-D behavior has a major effect on its seismic stability and to show that under certain circumstances which are often encountered in practice, 3-D effects can be very significant and should be considered when evaluating the stability of earth dams.

## DESCRIPTION OF DAM

The dam is located in Northern California approximately one mile west of a major active fault capable of generating a maximum credible earthquake of magnitude 6 3/4. Such an event could result in earthquake shaking at the dam site with a peak horizontal acceleration on bedrock of up to 0.7g.

The dam is an earthfill of clayey and sandy soils approximately 27 meters high with upstream and downstream slopes of 2 1/2:1 and 2:1, respectively. The crest is about 5 meters wide and 200 meters long. Fig. 1 shows a section along the longitudinal axis of the dam looking upstream. Although thin portions of the embankment extend over the abutments, the main body of the dam sits in a narrow canyon about 12 meters wide at the bottom and with side slopes of about 1 3/4:1. Thus, the effective length over height ratio L/H of the dam is about 4.

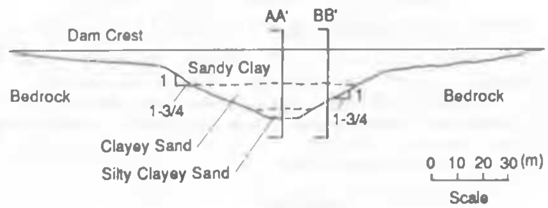


Fig. 1 Section along longitudinal axis looking downstream

Built in 1908, few records of the dam's construction remain. Exploration of the dam materials and research of the construction records revealed that while most of the embankment consists of stiff sandy clays and clayey sands, a 5-to 10-ft thick layer of loose silty clayey sand was placed at the bottom of the canyon below a portion of the downstream shell as shown in Fig. 2. These materials however do not

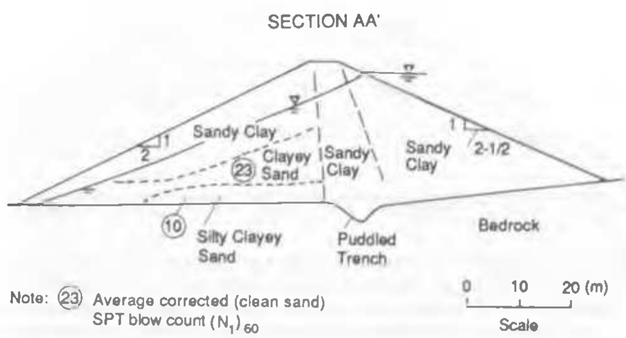


Fig. 2 Maximum cross section of dam

appear to extend up to the abutment sections of the dam as shown in Fig. 3.

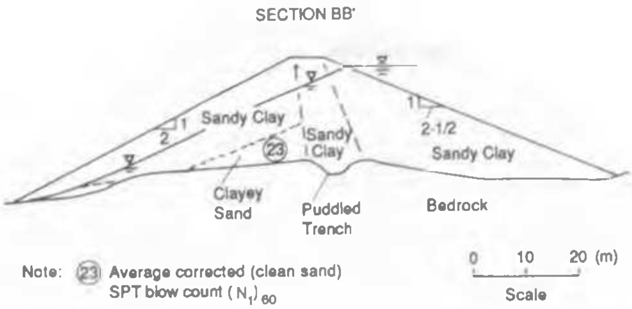


Fig. 3 Typical abutment cross section

## EMBANKMENT MATERIAL PROPERTIES

A summary of material properties used in the stability analyses described below is presented in Table I. In general, the sandy clays in the downstream shell have a liquid limit between 30 and 35 and a plasticity index between 10 and 13. The clays in the upstream shell and core are slightly more plastic and finer grained than those downstream. The clayey sands have about 40 to 50 percent fines and a plasticity index of about 10. These materials have an average corrected "clean sand" (Seed, 1987) Standard Penetration Test (SPT) blow count $(N_1)_{60}$ of about 23 (see Fig. 2). The silty clayey sand at the bottom of the canyon has about 30 to 45 percent fines, a plasticity index of about 7, and an average corrected "clean sand" blow count $(N_1)_{60}$ of about 9 to 10.

TABLE I

Summary of Material Properties Used in Stability Analyses

| Material | Effective Stress Parameters $\phi'$ | Effective Stress Parameters c'(kPa) | Undrained Shear Strength (kPa) |
|---|---|---|---|
| Sandy Clay-Upstream | $32^o$ | 10 | $17.5+0.31\sigma_v'$(1) |
| Sandy Clay-Core | $32^o$ | 10 | $17.5+0.31\sigma_v'$ |
| Sandy Clay-Downstream | $38^o$ | 0 | $0.52\sigma_v'$ |
| Clayey Sand | $40^o$ | 0 | $0.55\sigma_v'$ |
| Silty Clayey Sand | $35^o$ | 0 | $0.45\sigma_v'$ |

Note: (1) $\sigma_v'$ denotes vertical effective stress

Cyclic triaxial tests indicated that the embankment sandy clays have a very low liquefaction potential and will not lose significant strength during cyclic loading. The tests also indicated that the clayey sands are unlikely to liquefy but may generate significant excess pore pressures during cyclic loading. Monotonic shearing after the tests indicated that the undrained strength of these materials after cyclic loading may be approximated by the following expression:

$$S_{upc} = S_u\ (1-r_u)^{0.16} \qquad (1)$$

where $S_{upc}$ is the undrained strength after cyclic loading, $S_u$ is the undrained static shear strength, and $r_u$ is the excess pore pressure ratio.

## LIQUEFACTION POTENTIAL

The potential for generation of excess pore pressures and liquefaction of the clayey and silty sands during the maximum credible earthquake which could affect the dam, was evaluated using the procedures proposed by Seed (1983) and Seed et al. (1984). The dynamic response of the dam during the earthquake was determined from the response of individual soil columns. Dynamic properties used in the analyses were evaluated from the results of downhole shear and compression wave velocity measurements. Dynamic shear stresses calculated from these one-dimensional analyses were then modified to account for 3-D effects based on the results of Mejia and Seed (1983).

Factors of safety against liquefaction of the clayey and silty sands were computed by comparing the dynamic shear stresses induced by the earthquake to the shear stresses required to induce liquefaction based on the SPT blow count of the materials as proposed by Seed et al. (1984). The computed factors of safety together with the results of the cyclic triaxial tests were used to evaluate excess pore pressure ratios immediately after the earthquake as shown in Fig. 4. It can be seen that the silty clayey

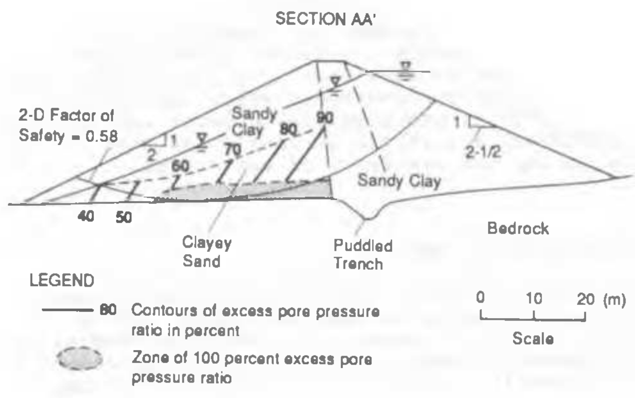


Fig. 4 Excess pore pressures in maximum cross section immediately after the earthquake

sand at the bottom of the canyon is expected to develop an excess pore pressure ratio of 100 percent and liquefy. Furthermore, based on its SPT blow count the residual strength of the material (Seed, 1987) is expected to be less than about 10 kPa. Fig. 4 also shows that although the clayey sand underlying most of the downstream shell is not expected to liquefy, it may develop relatively high pore pressure ratios during the earthquake.

## STABILITY EVALUATION

### Two-dimensional Analysis

The potential for instability of the downstream slope as a result of excess pore pressures generated during the earthquake was evaluated using conventional 2-D stability analyses based on Bishop's modified method of slices for circular failure surfaces. This method is implemented in the computer program STABR coded at the University of California at Berkeley. The analyses were performed using the undrained shear strengths shown in Table I with the following exceptions: a) shear strengths for the clayey sand were evaluated using the calculated excess pore pressure ratios shown in Fig. 4 and equation (1), and b) a very small strength was assigned to the liquefied silty sand reflecting the fact that this material has a low residual strength which will develop only after significant strain.

Assuming 2-D conditions for the maximum section of the dam (see Fig. 1), the minimum computed factor of safety against sliding of the downstream slope immediately after the earthquake is 0.58. The corresponding failure surface is shown in Fig. 4. It can be seen that the 2-D stability of the slope is controlled by the presence of the liquefied layer at the bottom of the canyon. Thus, based on conventional 2-D stability analyses of the maximum section alone, it would be concluded that major sliding of the downstream slope would occur as a result of the earthquake.

However, 2-D analyses neglect the fact that the liquefied materials are confined to the bottom of the canyon and that other sections of the dam are significantly more stable. For example, as shown in Fig. 5, 2-D stability analyses of the abutment section shown in Fig. 1 (section BB') result in a minimum factor of safety of 1.36. These side sections will restrain movement of the center section and improve its stability. Thus, it can be concluded that 3-D behavior is likely to have a significant effect on the dam's seismic stability.

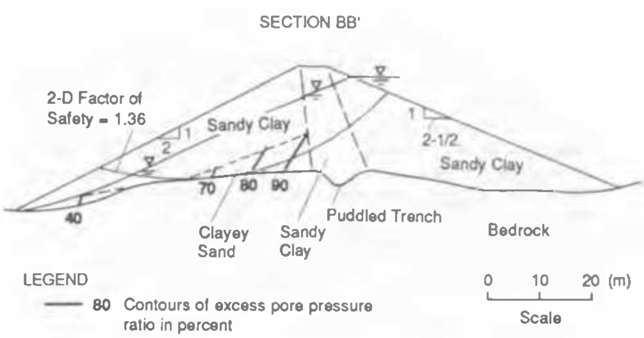


Fig. 5 Stability of typical abutment section immediately after the earthquake

### Three-dimensional Analysis

Three-dimensional stability analyses of the downstream slope immediately after the earthquake were performed using the method proposed by Baligh and Azzouz (1975). This method assumes that the 3-D failure surface is a solid of revolution. The method was implemented using a modified version of the computer program STABR. A commercially available spread-sheet program was used to evaluate the length of the critical failure surface based on the output from STABR.

The 3-D failure surface was assumed to consist of a central cylinder with ellipsoidal ends. A plane of symmetry was assumed at the maximum cross section (section AA') since the dam is nearly symmetrical with respect to this section. The central cylindrical surface was assumed to be as wide as the bottom of the canyon (about 15 ms). A search for the critical 3-D failure surface was conducted by varying the total surface length along the longitudinal axis of the dam. The computed 3-D factors of safety immediately after the earthquake for vaiious assumed lengths are shown in Fig. 6 by the solid line. It can be seen that the 3-D factor of safety varies from about 1.6 to about 1.25 depending on the length of the failure surface.

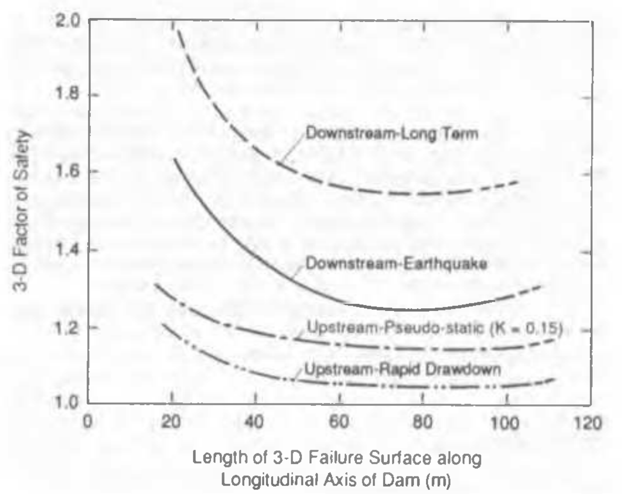


Fig. 6 Variation of 3-D factor of safety with length of failure surface

The minimum computed 3-D factor of safety is 1.25. This represents an increase of almost 120 percent over the 2-D factor of safety. This increase is due to the 3-D restraint that the sides of the embankment impose on the central section underlain by the liquefied materials. Thus, 3-D analyses would indicate that the potential for major sliding of the downstream slope is significantly lower than indicated by 2-D analyses of the maximum section alone. Such a wide difference in conclusions regarding the safety of the dam points out the need for considering 3-D effects in cases such as this.
It is interesting to note that the computed critical 3-D failure surface is significantly wider than the section of embankment underlain by the liquefied materials. Although intuitively the failure surface might be expected to be confined to this center section, the analyses

indicate that there is a tendency for this section to drag the side sections along with it resulting in a wider failure surface.

## DISCUSSION

The effects of 3-D behavior on the stability of the downstream slope immediately after the earthquake are significantly larger than those which might be expected based on previously published results for homogeneous cohesive soils. This large difference is due to the presence of the weak layer of liquefied soil at the bottom of the canyon. Interestingly, when more homogeneous embankment conditions prevail, such as under pre-earthquake static loading, the computed 3-D effects are smaller.

The variation of 3-D factor of safety with failure surface length for long term loading of the downstream slope and rapid drawdown loading of the upstream slope is shown in Fig. 6. Also shown in this figure are factors of safety for pseudo-static loading of the upstream slope (for a seismic coefficient of 0.15). The minimum computed 3-D factors of safety are compared with the results of 2-D analyses in Table II. It can be seen that under pre-earthquake static loading the differences between minimum 2-D and 3-D factors of safety are less than 10 percent. These results are in good agreement with those from previous studies (Baligh and Azzouz, 1975; Chen and Chameau, 1982; Gens et al. 1988).

Previous studies indicate that for homogeneous cohesive slopes 3-D effects become smaller as the length of the failure surface increases. This trend is also evident from the results shown in Fig. 6 for static loading. On this basis, it can be concluded that for relatively homogeneous dams in narrow canyons the critical failure surface will extend to the canyon walls. However, for zoned dams the critical surface will depend on the distribution of material zones within the dam.

## CONCLUSIONS

This paper has presented an example of an old earth dam for which 3-D behavior has a major effect on the dam's seismic stability. Important conclusions from this study include: 1) three-dimensional behavior can have a significant effect on the stability of earth and rockfill dams under conditions which are often encountered in practice, 2) neglecting these effects can result in an over-conservative assessment of stability, 3) for dams in narrow canyons with length to height ratios L/H up to about 4 and weak soils at the bottom of the embankment or within the foundation, 3-D factors of safety may be up to 120 percent larger than those computed assuming 2-D behavior, and 4) for relatively homogeneous dams in narrow canyons the critical 3-D failure surface will extend to the canyon walls. For zoned dams the extent of the critical surface will depend on zoning of the dam.

TABLE II

Comparison Between 2-D and 3-D Factors of Safety

| Loading Conditions | Factor of Safety 2-D | 3-D | Percent Difference |
|---|---|---|---|
| Downstream Long Term | 1.46 | 1.56 | 7% |
| Downstream Earthquake | 0.58 | 1.25 | 116% |
| Upstream Rapid Drawdown | 0.96 | 1.05 | 9% |
| Upstream Pseudo-Static (K=0.15) | 1.05 | 1.15 | 10% |

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