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CONTENTS

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DISCUSSION

Proc. Paper 11534

TECHNICAL NOTES

Proc. Paper 11538

[a]Discussion period closed for this paper. Any other discussion received during this discussion period will be published in subsequent Journals.

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JOURNAL OF THE GEOTECHNICAL ENGINEERING DIVISION

Dynamic Analysis of the Slide in the Lower San Fernando Dam during the Earthquake of February 9, 1971

By H. Bolton Seed,[1] Izzat M. Idriss,[2] Kenneth L. Lee,[3] Members, ASCE, and Faiz I. Makdisi,[4] A. M. ASCE

Introduction

During the San Fernando, California, earthquake of February 9, 1971 (magnitude 6.6) a major slide occurred in the upstream slope of the Lower San Fernando Dam (Fig. 1), requiring, as a safety precaution, the temporary evacuation of 80,000 people living downstream. In the same earthquake, slide movements also caused a downstream movement of about 5 ft (1.5 m) in the embankment of the Upper San Fernando Dam. Both dams were within a few miles of the main zone of energy release by the earthquake. Detailed descriptions of these events and the studies conducted to establish the mechanism of sliding and the applicability of pseudostatic procedures for analyzing the stability of the embankments have been presented previously (Seed, et al., 1975) together with the results of a comprehensive study of the characteristics of the soils comprising the embankments and foundations of the two dams (Lee, et al., 1975).

It was concluded from these investigations that the primary cause of the upstream slide in the Lower Dam was the development, towards the end of the earthquake shaking, of very high pore-water pressures in an extensive zone of hydraulic fill near the base of the embankment and upstream of the clay core so that much of this soil was in a liquefied or very low strength condition.

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[1] Prof. of Civ. Engrg., Univ. of California, Berkeley, Calif.

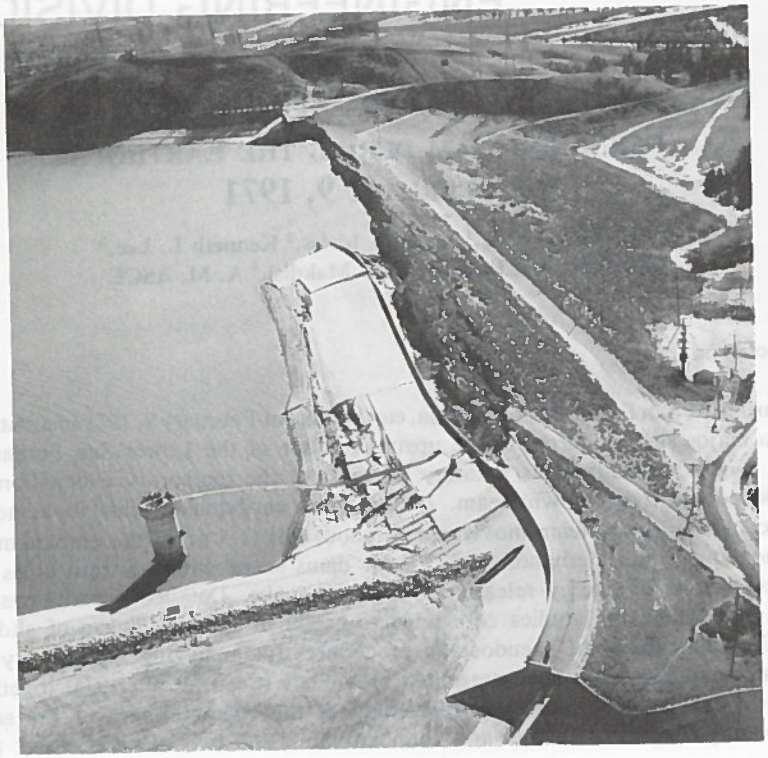
[2] Assoc., Woodward-Lundgren & Assocs., Oakland, Calif., and Asst. Research Engr., Univ. of California, Berkeley, Calif.

[3] Assoc. Prof., School of Engrg. & Applied Sci., Univ. of California, Los Angeles, Calif.

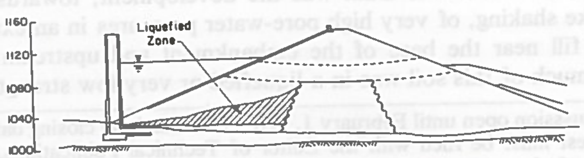
[4] Grad. Research Asst., Dept. of Civ. Engrg., Univ. of California, Berkeley, Calif.

The location and extent of this zone, as determined by a reconstruction of the slide mechanism is shown in Fig. 1(*b*).

In the Upper Dam, there was similar evidence of development of high pore water pressures in some zones of the embankment but the extent of these zones was not sufficient to cause a major slide.



(a) View of Dam after Earthquake
(Photograph by U.S. Geologic Survey)



(b) Cross section Showing Zone of Liquefaction

FIG. 1.—Slide in Lower San Fernando Dam

Both the Upper and Lower San Fernando Dams were of the hydraulic fill type and because there are 30 other hydraulic fill dams in California, some of which are in areas where seismic activity may be at least as severe as in San Fernando, it was considered to be of major importance to utilize the San Fernando experience to evaluate: (1) The ability of current design and analysis

procedures to determine the seismic safety of these types of structures and others with generally similar characteristics; and (2) whether it would be desirable to adopt new design procedures and criteria for evaluating the seismic stability of embankments of this type.

Both preearthquake and postearthquake evaluations of the anticipated behavior of the dam based on pseudostatic analysis procedures using a seismic coefficient of 0.15 indicated substantial factors of safety. For the Upper Dam, such analyses led to a computed factor of safety of 2-2.5 while for the Lower Dam the computed factors of safety ranged from 1.22-1.6 depending on the details of the computational procedure used (Seed, et al., 1975). Thus, conventional practice in the use of this method would not have indicated any potential for slide movements in either dam.

To compute a factor of safety of 1.0 using the pseudostatic method of analysis, it would have been necessary to use seismic coefficients in the range of 0.25-0.35 for the Lower Dam or 0.43-0.55 for the Upper Dam, depending on how the test data is utilized in the computation procedure. These results pose a number of difficult problems for design engineers. If seismic coefficients of the order of 0.25-0.5 are required to adequately assess the stability of these types of dams against shaking of the intensity developed in San Fernando, should values of comparable magnitude be used for similar dams that may be subjected to comparable levels of shaking—or even higher values for more severe shaking intensities? The use of such values would lead to very much flatter slopes than have conventionally been used for earth dams leading to unnecessary expense in many cases and providing little additional benefit in others. However, considerable difficulty would be encountered in knowing under which conditions these situations would apply or, as in San Fernando, whether the use of higher values of seismic coefficients might be justified. In view of the many other limitations of pseudostatic analysis procedures (Seed et al., 1969), it was considered desirable to explore the applicability of dynamic analysis procedures for evaluating the stability of the embankments. The results of such an analysis for the Lower Dam are presented herein.

Dynamic Analyses of Stability of Lower Dam during San Fernando Earthquake

Procedures for dynamic analysis of embankment stability have only recently been developed (Newmark, 1965; Seed, 1966). In dealing with saturated cohesionless materials for which pore pressures may vary during an earthquake, it has been found most convenient to utilize the procedure proposed by the senior writer involving the following steps: (1) Determine the initial stresses in the embankment before the earthquake; (2) determine the characteristics of the motions developed in rock underlying the embankment and its soil foundation during the earthquake; (3) evaluate the response of the embankment to the base rock excitation and compute the dynamic stresses induced in representative elements of the embankment; and (4) by subjecting representative samples of soil to the combinations of preearthquake stress conditions and superimposed dynamic stress applications, determine by test the effects of the earthquake-induced stress on soil elements in the embankment—these effects will include any evidence of soil liquefaction and the magnitude of the deformations. This procedure has been found to provide a satisfactory evaluation of the failure of the Sheffield Dam during the Santa Barbara earthquake of 1925 (Seed, et al., 1969) and it has been used for design studies of a number of other embankment dams. Accordingly, it was adopted for analysis of the San Fernando Dams in the 1971 earthquake. Details of the procedures following in implementing the various steps in the analysis are described in the following sections.

Static Analysis.—The dynamic test results presented by Lee, et al. (1975)

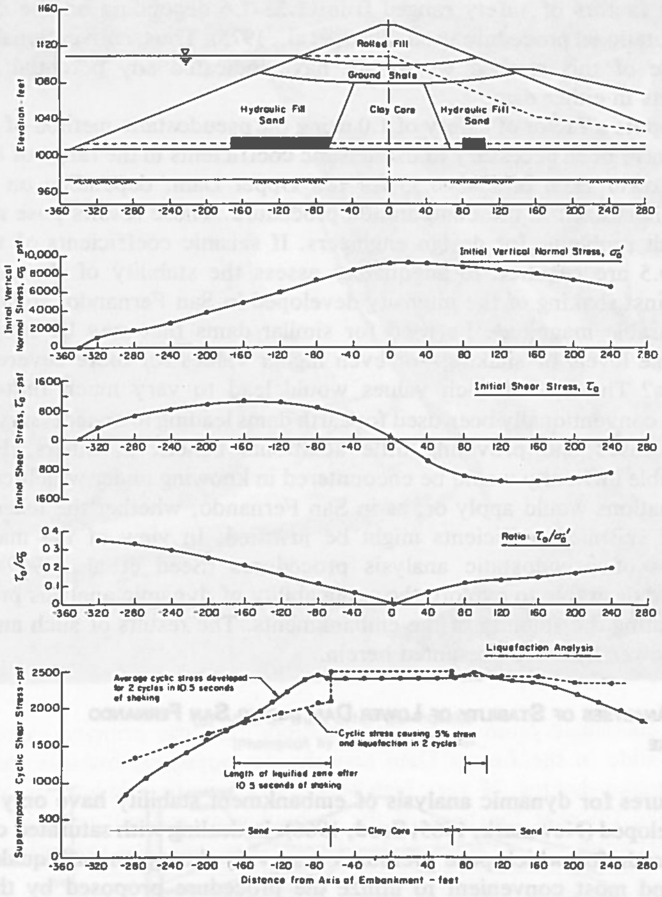


FIG. 2.—Analysis of Soil Stability along Base of Embankment [El. 1,014 (309 m)] after 10.5 sec of Shaking, Using Base Motions Determined from Seismoscope Record—Lower San Fernando Dam (1 ft = 0.305 m; 1 psf = 47.9 N/m²)

clearly indicate that the effects of cyclic loading on the behavior of the embankment soils are considerably influenced by the stresses existing in the soil before the cyclic stresses are applied. Of particular importance are the initial effective normal stresses, σ'_{fc}, and the ratio, τ_{fc}/σ'_{fc}, along the potential failure plane (for which τ_{fc} = the initial shear stress).

These initial stresses can be evaluated most conveniently by static finite element

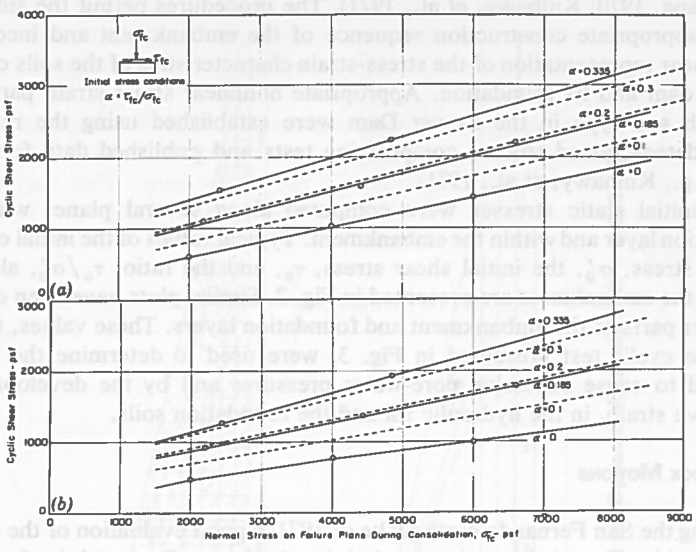


FIG. 3.—Cyclic Stress Conditions Causing 5% Strain and Liquefaction for Hydraulic Sand Fill—Lower San Fernando Dam: (*a*) Two Cycles; (*b*) Five Cycles (1 psf = 47.9 N/m²)

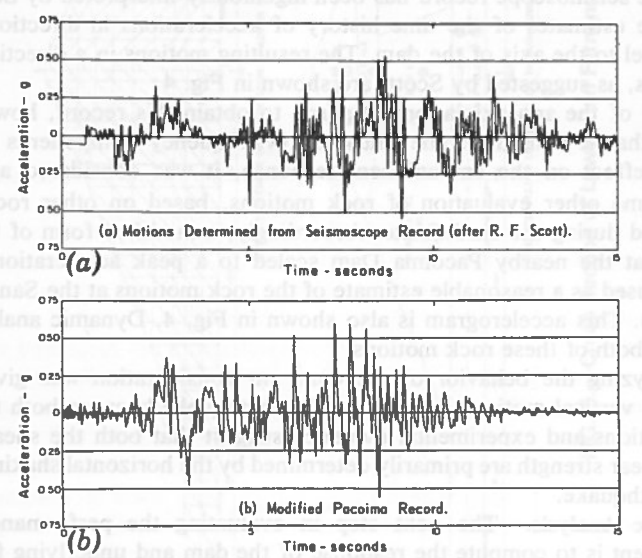


FIG. 4.—Time Histories of Acceleration in Base Rock

procedures. The use of these procedures for computing static stresses in soil structures is described in detail in several recent publications (e.g., Duncan and Chang, 1970; Kulhawy, et al., 1971). The procedures permit the simulation of the appropriate construction sequence of the embankment and incorporate a nonlinear representation of the stress-strain characteristics of the soils comprising the dam and its foundation. Appropriate nonlinear stress-strain parameters for each soil type in the Lower Dam were established using the results of consolidated-drained triaxial compression tests and published data for similar soils (e.g., Kulhawy, et al., 1971).

The initial static stresses were computed along several planes within the foundation layer and within the embankment. Typical values of the initial effective normal stress, σ'_0, the initial shear stress, τ_0, and the ratio, τ_0/σ'_0, along the base of the embankment are presented in Fig. 2. Similar plots have been obtained for other parts of the embankment and foundation layers. These values, together with the cyclic test presented in Fig. 3, were used to determine the stresses required to cause excessive pore-water pressures and by the development of excessive strain, in the hydraulic fill and the foundation soils.

Base Rock Motions

During the San Fernando earthquake of 1971, a good evaluation of the motions developed in the rock formations underlying the Lower Dam and the foundation alluvium was provided by a seismoscope record obtained on the abutment. The rock formation at the recording station is generally similar to that underlying the embankment and the motion characteristics should therefore be about the same. The seismoscope record has been ingeniously interpreted by Scott (1973) to provide estimates of the time history of accelerations in directions normal and parallel to the axis of the dam. The resulting motions in a direction normal to the axis, as suggested by Scott, are shown in Fig. 4.

In view of the approximations required to obtain this record, however, and the fact that it contains some unusual low-frequency components that have a strong effect on the embankment response, it was considered appropriate to use some other evaluation of rock motions, based on other rock records determined during the earthquake. Accordingly, a modified form of the record obtained at the nearby Pacoima Dam scaled to a peak acceleration of 0.6 g was also used as a reasonable estimate of the rock motions at the San Fernando Dam sites. This accelerogram is also shown in Fig. 4. Dynamic analyses were made for both of these rock motions.

In analyzing the behavior of the dam, no consideration was given to the effects of vertical motions induced by the earthquake because both theoretical considerations and experimental evidence suggest that both the shear stresses and the shear strength are primarily determined by the horizontal shaking induced by an earthquake.

Dynamic Analysis.—The next step in evaluating the performance of the embankment is to compute the response of the dam and underlying foundation during the earthquake (Seed, et al., 1969) to provide values of cyclic stresses that are likely to be induced in the soils.

This analysis is most conveniently performed using nonlinear dynamic finite element procedures. The nonlinear dynamic material properties are incorporated

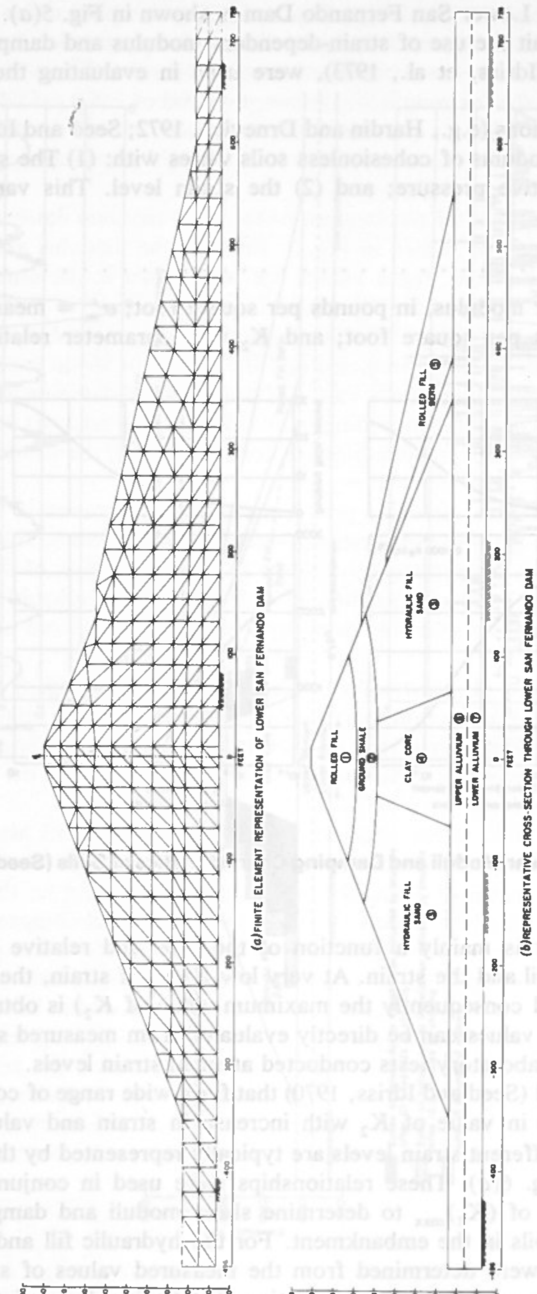


FIG. 5.—Cross Section of Lower San Fernando Dam Used for Dynamic Analysis (1 ft = 0.305 m)

in the analysis by using strain-dependent modulus and damping values as subsequently described. The finite element representation used in evaluating the response of the Lower San Fernando Dam is shown in Fig. 5(*a*). Computer programs that permit the use of strain-dependent modulus and damping values for each element (Idriss, et al., 1973), were used in evaluating the response of the dam.

Recent investigations (e.g., Hardin and Drnevich, 1972; Seed and Idriss, 1970) indicate that the modulus of cohesionless soils varies with: (1) The square root of the mean effective pressure; and (2) the strain level. This variation can be expressed by

$$G = 1{,}000\, K_2\, \sigma_m'^{1/2} \quad (1)$$

in which G = shear modulus, in pounds per square foot; σ_m' = mean effective pressure in pounds per square foot; and K_2 = a parameter relating G and

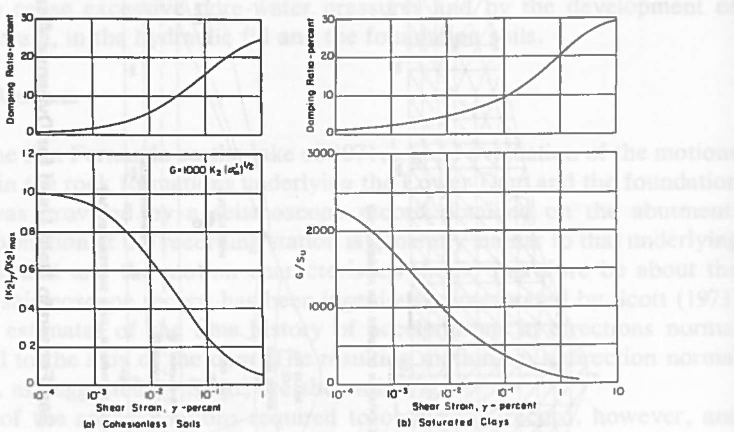


FIG. 6.—Average Shear Moduli and Damping Characteristics of Soils (Seed and Idriss, 1970)

σ_m'; this parameter is mainly a function of the type and relative density of the cohesionless soil and the strain. At very low levels of strain, the maximum modulus value (and consequently the maximum value of K_2) is obtained. The maximum modulus values can be directly evaluated from measured shear wave velocities or from laboratory tests conducted at small strain levels.

It has been found (Seed and Idriss, 1970) that for a wide range of cohesionless soils, the decrease in value of K_2 with increase in strain and values of the damping ratio at different strain levels are typically represented by the relationships shown in Fig. 6(*a*). These relationships were used in conjunction with appropriate values of $(K_2)_{max}$ to determine shear moduli and damping ratios for cohesionless soils in the embankment. For the hydraulic fill and alluvium, values of $(K_2)_{max}$ were determined from the measured values of shear wave velocity. For other soils in the cross section that have a lesser influence on the response, it was considered that estimates of $(K_2)_{max}$ based on previous test data for similar materials would provide a sufficient degree of accuracy.

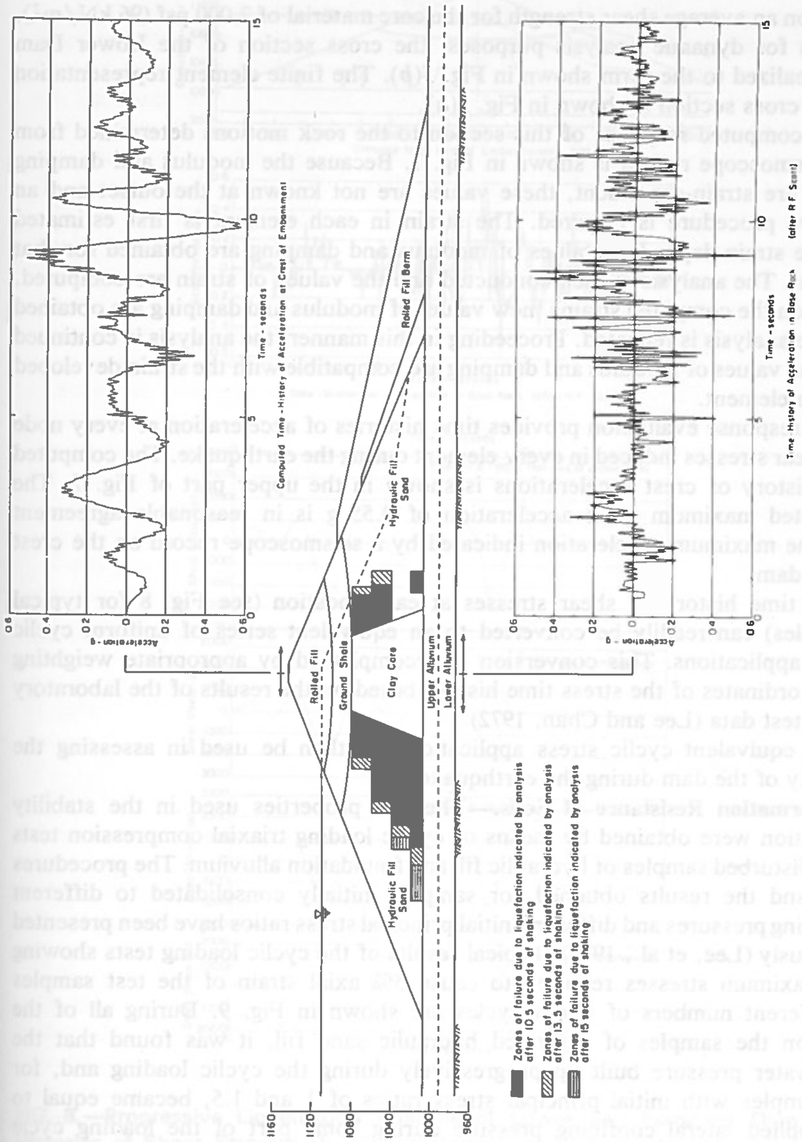


FIG. 7.—Analysis of Response of Lower Dam during San Fernando Earthquake to Base Motions Determined from Seismoscope Record (1 ft = 0.305 m)

Shear modulus values for saturated cohesive soils have been found to vary with the undrained shear strength and the strain level approximately as shown in Fig. 6(*b*) (Seed and Idriss, 1970). Representative damping ratios for these soils are shown in this figure. Shear moduli and damping ratios for the clay core of the Lower Dam were determined from the results shown in Fig. 6(*b*), based on an average shear strength for the core material of 2,000 psf (96 kN/m^2).

Thus for dynamic analysis purposes, the cross section of the Lower Dam was idealized to the form shown in Fig. 5(*b*). The finite element representation of this cross section is shown in Fig. 5(*a*).

The computed response of this section to the rock motions determined from the seismoscope record is shown in Fig. 7. Because the modulus and damping values are strain-dependent, these values are not known at the outset and an iterative procedure is required. The strain in each element is first estimated and the strain-dependent values of modulus and damping are obtained for that element. The analysis is then conducted and the values of strain are computed. Based on the computed strains, new values of modulus and damping are obtained and the analysis is repeated. Proceeding in this manner, the analysis is continued until the values of modulus and damping are compatible with the strain developed in each element.

The response evaluation provides time histories of acceleration at every node and shear stresses induced in every element during the earthquake. The computed time history of crest accelerations is shown in the upper part of Fig. 7. The computed maximum crest acceleration of 0.55 g is in reasonable agreement with the maximum acceleration indicated by a seismoscope record on the crest of the dam.

The time history of shear stresses at each location (see Fig. 8 for typical examples) can readily be converted to an equivalent series of uniform cyclic stress applications. This conversion is accomplished by appropriate weighting of the ordinates of the stress time history based on the results of the laboratory cyclic test data (Lee and Chan, 1972).

The equivalent cyclic stress application can then be used in assessing the stability of the dam during the earthquake.

Deformation Resistance of Soils.—The soil properties used in the stability evaluation were obtained by means of cyclic loading triaxial compression tests on undisturbed samples of hydraulic fill and foundation alluvium. The procedures used and the results obtained for samples initially consolidated to different confining pressures and different initial principal stress ratios have been presented previously (Lee, et al., 1975). Typical results of the cyclic loading tests showing the maximum stresses required to cause 5% axial strain of the test samples in different numbers of stress cycles are shown in Fig. 9. During all of the tests on the samples of saturated hydraulic sand fill, it was found that the pore-water pressure built up progressively during the cyclic loading and, for the samples with initial principal stress ratios of 1 and 1.5, became equal to the applied lateral confining pressure during some part of the loading cycle by the time the axial strain had increased to 5%. In this sense, the samples could be considered to have liquefied in that they would undergo some degree of strain with no significant resistance to deformation. However, under the largest stress in any load cycle, the pore pressure was reduced, effective confining pressures were developed, and the samples were again able to support the applied

load. With increasing numbers of cycles the strain developed before the sample stabilized in this way was progressively increased. In all tests of this type where

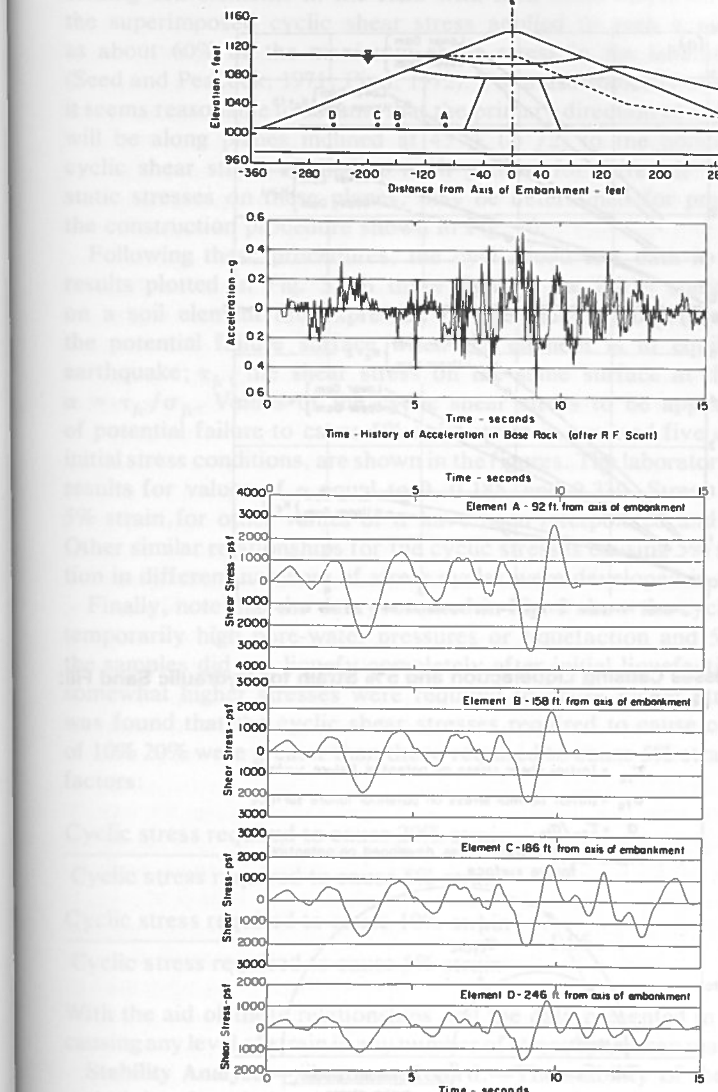


FIG. 8.—Progressive Liquefaction Analysis of Lower San Fernando Dam: Time Histories of Shear Stresses Developed along Base of Embankment (1 ft = 0.305 m; 1 psf = 47.9 N/m^2)

the strain exceeded 5%, the residual pore-water pressure on completion of a load cycle was equal to the applied confining pressure. Thus, the data in Fig.

9 show the stresses required to cause 5% strain and liquefaction, defined in this way.

In utilizing the results of cyclic load tests in a dynamic analysis it is convenient

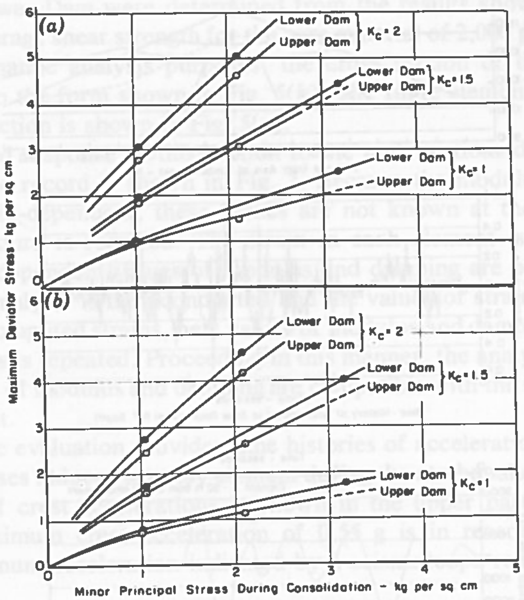


FIG. 9.—Cyclic Stresses Causing Liquefaction and 5% Strain for Hydraulic Sand Fill: (a) Two Cycles; (b) Five Cycles

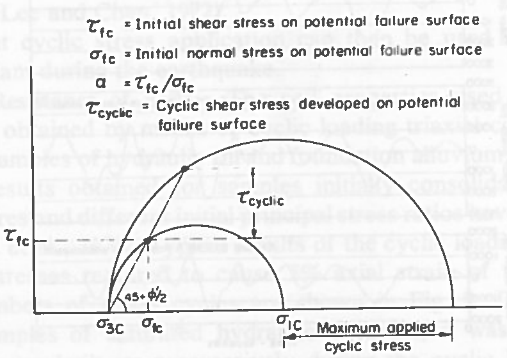


FIG. 10.—Procedure for Interpretating Cyclic Load Triaxial Test Data to Determine Cyclic Shear Stress on Potential Failure Surface

to determine the deformations produced by known values of cyclic shear stress superimposed on the initial (preearthquake) static stress conditions. For this purpose, the cyclic load test data presented in Fig. 9 must be interpreted in

terms of the cyclic shear stress developed in the primary direction of potential failure.

For samples initially consolidated under an isotropic stress condition, representing soil elements in the field with zero shear stress on horizontal planes, the superimposed cyclic shear stress applied to such a plane may be taken as about 60% of the maximum shear stress in the laboratory test specimen (Seed and Peacock, 1971; Finn, 1972). For anisotropically-consolidated samples, it seems reasonable to assume that the primary direction of failure and movement will be along planes inclined at $45 + (\phi'/2)$ to the horizontal and thus the cyclic shear stress applied to such planes, for different values of the initial static stresses on these planes, may be determined for practical purposes by the construction procedure shown in Fig. 10.

Following these procedures, the cyclic load test data in Fig. 9 lead to the results plotted in Fig. 3. In these figures, the initial static stress conditions on a soil element are expressed by the value of σ_{fc}, the normal stress on the potential failure surface when the element is in equilibrium before the earthquake; τ_{fc}, the shear stress on the same surface at the same time; and $\alpha = \tau_{fc}/\sigma_{fc}$. Values of the cyclic shear stress to be applied in the direction of potential failure to cause 5% axial strain in two and five cycles for different initial stress conditions, are shown in the figures. The laboratory test data provides results for values of α equal to 0, 0.185, and 0.330. Stress conditions causing 5% strain for other values of α have been interpolated and plotted as shown. Other similar relationships for the cyclic stresses causing 5% strain and liquefaction in different numbers of stress cycles were developed in the same way.

Finally, note that the data presented in Fig. 3 show the cyclic stresses causing temporarily high pore-water pressures or liquefaction and 5% strain. Because the samples did not liquefy completely after initial liquefaction had developed, somewhat higher stresses were required to cause higher strains. Generally, it was found that the cyclic shear stresses required to cause compression strains of 10% 20% were greater than those required to cause 5% strain by the following factors:

$$\frac{\text{Cyclic stress required to cause 20\% strain}}{\text{Cyclic stress required to cause 5\% strain}} = 1.15 \quad (2)$$

$$\frac{\text{Cyclic stress required to cause 10\% strain}}{\text{Cyclic stress required to cause 5\% strain}} = 1.06 \quad (3)$$

With the aid of these relationships and the data presented in Fig. 3 the stresses causing any level of strain in any number of stress cycles can readily be determined.

Stability Analysis—Abutment Record.—The stability of the embankment and underlying foundation is assessed by comparing the stresses induced by the earthquake (using the abutment record as input rock motion) and the stresses required to cause liquefaction or prescribed limits of strain, or both. The locations for which the induced stresses exceed the stresses required to cause liquefaction or acceptable strain limits determine the extent of liquefaction or unacceptable performance.

The stability of the embankment was evaluated by investigating the liquefaction and strain potential within four potential zones of failure located as follows:

(1) A zone along the base of the embankment extending from El. 1,008-El. 1,020 (307 m-311 m)—the stress conditions along the plane at El. 1,014 (309 m) (average elevation for the zone) were used for evaluating the possibility of liquefaction and the strain potential in this zone; (2) a zone extending from El. 1,020-El. 1,040 (307 m-317 m); (3) a zone extending from El. 1,040-El. 1,060 (317 m-323 m); and (4) a zone extending from El., 1,060-El. 1,080 (323 m-329 m).

In making these evaluations, consideration was given to the tendency for liquefaction in the hydraulic fill to develop progressively. Previous studies (Seed, et al., 1969) have indicated that the progressive development of liquefaction, starting with elements near the center of an embankment and progressing toward the face of the dam, can markedly influence the potential behavior of embankments during earthquakes.

As can be noted from Fig. 7, the peak accelerations (and therefore the peak stresses) occur approx 10 sec after the start of the motions. Accordingly, the liquefaction potential was initially investigated for the first 10.5 sec of the motion. This evaluation along the base of the embankment is presented in Fig. 2. The stresses induced in this section during the first 10.5 sec of shaking were represented by two cycles of an equivalent uniform cyclic stress. The stress required to cause liquefaction and 5% strain in two cycles were obtained from Fig. 3(*a*), using the values of initial normal stress, σ_0', and the ratio, τ_0/σ_0', shown in Fig. 2(*a*). The stresses developed for two cycles in 10.5 sec of shaking along the base of the embankment are compared to the stresses required to cause liquefaction in two cycles in Fig. 2(*b*). A very small zone of liquefaction (i.e., where the stresses developed during shaking exceed the stresses required to cause transient liquefaction and 5% strain) appears to develop in the downstream part of the dam. However, an extensive zone of liquefaction, extending over a distance of approx 100 ft (30 m), develops in the hydraulic fill upstream of the clay core. Similar analyses were made for the stress conditions developed along planes at El. 1,030, 1,050, and 1,070 (314 m, 320 m, and 326 m) leading to similar determination of zones of liquefaction.

The zones where transient liquefaction and strains exceeding 5% can be expected to develop within the embankment after 10.5 sec of shaking in the upstream and downstream sections of the dam as determined by these analyses are summarized in Fig. 7. Note that such liquefaction is indicated over a considerable part of the hydraulic fill upstream of the clay core; a much smaller zone of liquefaction is indicated in the hydraulic fill downstream of the clay core. However, the upstream extent of the zone of high pore pressures or liquefaction does not appear to be sufficiently extensive at this stage to permit the upstream slide that took place during the earthquake.

The effects of additional shaking on the embankment can be studied by continuing the analysis beyond the first 10.5 sec of shaking. Because a considerable part of the embankment has liquefied after 10.5 sec, redistribution of both static and dynamic stresses in the adjacent nonliquefied elements would take place during the remainder of the earthquake. The redistribution of dynamic stresses can be taken into account by reducing the shear moduli in the liquefied zones close to zero for the ensuing period of shaking. The analysis is then continued with the previously computed response values after 10.5 sec as initial values. Note that the modulus and damping values for elements in the nonliquefied zone are established based on the strain developed in each element during the ensuing period of motion.

The increase in liquefaction potential in the hydraulic fill during the ensuing 3 sec of motion (i.e., from 10.5 sec-13.5 sec) was evaluated in this way. The time histories of stresses developed in elements along the base of the embankment after 13.5 sec of shaking are shown in Fig. 8. The time history of stresses in elements that had not liquefied after 10.5 sec of shaking were converted to three equivalent uniform cycles using the procedures previously outlined, and compared with the stresses required to cause transient liquefaction and 5% strain in three cycles. This comparison showed that an additional zone of liquefaction extending approx 25 ft (7.6 m) upstream along the base of the embankment would develop after 13.5 sec of shaking. However, no additional zone of liquefaction appears likely to develop in the downstream part.

Similar evaluations for the other parts of the embankments were made and the extent of liquefaction in the hydraulic fill after 13.5 sec of shaking is summarized in Fig. 7.

Finally, the procedure was repeated for the remainder of the motion (i.e., from 13.5 sec-15 sec). The analysis of the embankment was continued with the previously computed response values after 13.5 sec as initial values, and the shear moduli in the liquefied zones assigned values close to zero. The time history of stresses in the elements that had not liquefied in 13.5 sec of shaking were converted to four cycles of equivalent cyclic stresses. Comparing the stresses developed along the base of the embankment for four cycles after 15 sec of shaking and the stresses required to cause transient liquefaction and 5% strain in four cycles led to an additional zone of liquefaction extending approx 50 ft (15 m) upstream after 15 sec of shaking. Similar evaluations for the other parts of the embankment were made and the extent of liquefaction in the hydraulic fill after 15 sec of shaking was determined. The zones of liquefaction indicated by the analysis for the entire duration of motion are summarized in Fig. 7. Note that these zones are in reasonable agreement with the areas where liquefaction was indicated to have occurred in the embankment by the field observations (see Fig. 1).

The zone of liquefaction in the hydraulic fill upstream of the clay core, indicated by analysis after 15 sec of shaking as shown in Fig. 7, appears to be extensive enough to result in the upstream slide that took place during the earthquake. The extent of the zone of liquefaction downstream of the clay core, however, appears to be too small to result in any significant downstream movement during the earthquake.

The same procedure was used to evaluate the liquefaction and strain potential in the foundation layer at El. 1,000 (33 m) (which represents the average depth within the upper alluvium). This analysis indicated that there was ample margin of safety against development of liquefaction or 5% strain in the upper alluvium during the San Fernando earthquake. Because the lower alluvium is denser than the upper alluvium and has even higher cyclic strength characteristics, it would have an even higher margin of safety against deformation and failure during this earthquake. Note that there was no field evidence of failure in the alluvium during the earthquake. Thus the results of this part of the analysis also correlate well with the observed field performance.

The stability of the dam was also evaluated considering the base motions developed in the rock underlying the dam to be similar to the modified Pacoima record shown in Fig. 4(*b*). This analysis led to essentially the same result as that previously described.

Quantitative Evaluation of Slope Displacements

While the preceding analyses provide a qualitative evaluation of the embankment performance which is in good agreement with the observed behavior, quantitative assessments of the extent of slope movements will often be required. Preliminary assessments of the potential deformations can be obtained by using the cyclic shear stresses computed by the dynamic response analyses in conjunction with the cyclic load test data to determine the potential compressive strains of individual elements of the embankment; i.e., the compressive strain the element would develop if it were subjected to the dynamic stresses induced by the earthquake and were not constrained by the deformations in the surrounding soil. For example, for a soil element near the base of the embankment and just upstream from the core, the data in Fig. 2 show that the computed stresses considerably exceed those required to cause transient liquefaction and 5% strain. Using the results presented previously for the stresses required to cause larger strains than 5%, it can readily be estimated that after 10.5 sec of shaking, elements of hydraulic fill in this zone of the embankment would develop a strain on the order of 20%–30% if they were subjected to the computed stresses but were not constrained by the adjacent soil. This strain may be considered as the strain potential after 10.5 sec of shaking for soil elements in this vicinity. It would, of course, be increased to some extent by further deformations occurring in the last 5 sec of earthquake shaking.

Values of strain potential for all elements of hydraulic fill, as indicated by the computed stresses for the seismograph record of base motion and the test data presented previously, are shown in Fig. 11. Contours of equal strain potential, based on these results, are shown in Fig. 12. It may be seen that very large strains, on the order of 20%–50% tend to develop in zones of the embankment located in the upstream shell near the base and near the center of the embankment adjacent to the clay core. On the other hand, the soil in the outer part of the upstream shell and in most of the downstream shell tends to develop relatively small potential strains ranging from 0%–10%.

In reality, the zones developing lower strains will tend to restrain the movements of zones of higher strain potential so that the overall displacements in the embankment will be a representative average of the behavior of all the elements. The movements of the soil in the embankment will depend to some extent on the distribution of strain potential, but failure might also be induced by the dead weight of the embankment after a zone of hydraulic fill adjacent to the core has lost its shear resistance as a result of the earthquake shaking. Fig. 13 shows the zone near the clay core where the strain potential is 10% or higher and for which the test data showed high residual pore pressures to have developed. If it is considered that the soil in this zone would provide no effective resistance to slide movements in the embankment, a stability analysis can be made to assess the stability of the slope against static failure, as shown in Fig. 13. Using average consolidated-undrained strength parameters because of the sudden change in stress distribution, the computed factor of safety along

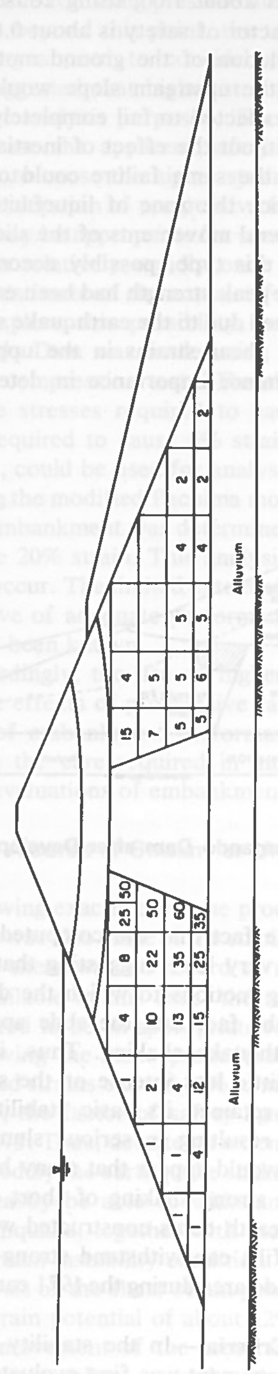


FIG. 11.—Strain Potential in Hydraulic Fill—Lower San Fernando Dam

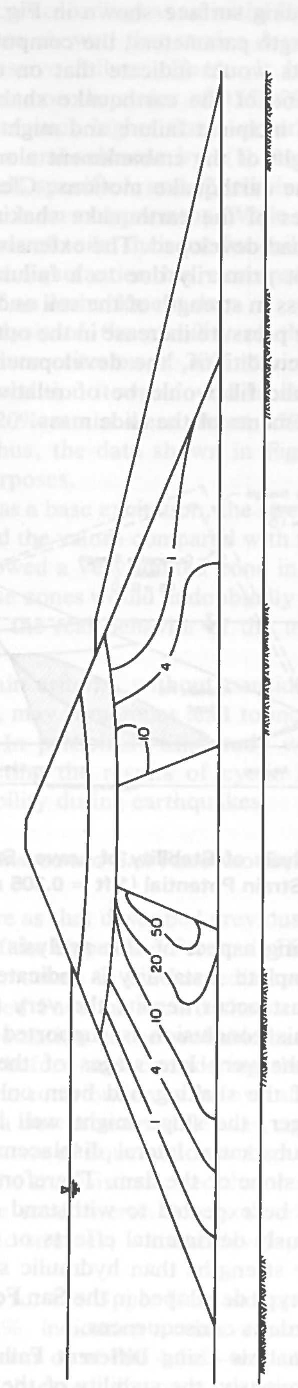


FIG. 12.—Contours of Strain Potential in Hydraulic Fill—Lower San Fernando Dam

the critical sliding surface shown in Fig. 13 is about 1.06; using conservative values of strength parameters, the computed factor of safety is about 0.8.

These results would indicate that on completion of the ground motions or towards the end of the earthquake shaking, the upstream slope would be in a condition of incipient failure and might be expected to fail completely under the static weight of the embankment alone without the effect of inertia forces induced by the earthquake motions. Clearly, the same failure could occur in the later stages of the earthquake shaking once the zone of liquefaction and strength loss had developed. The extensive lateral movements of the slide mass were no doubt primarily due to a failure of this type, possibly accompanied by a further loss in strength of the soil once the peak strength had been exceeded and some pore pressure increase in the outer shell due to the earthquake shaking. Under these conditions, the development of shear strains in the upper part of the hydraulic fill would be of relatively minor importance in determining the total movements of the slide mass.

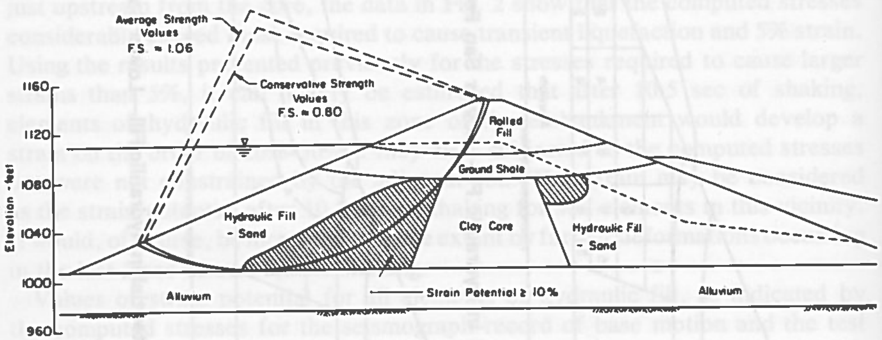


FIG. 13.—Analysis of Stability of Lower San Fernando Dam after Development of Zone of High Strain Potential (1 ft = 0.305 m)

An interesting aspect of this analysis is the fact that the computed margin by which complete instability is indicated is very low, suggesting that failure would only just occur despite the very strong motions to which the dam was subjected. This conclusion is supported by the fact that the slide apparently occurred in the very late stages of the earthquake shaking. Thus, it would appear that if the shaking had been only a little less intense or the soil only a little stronger, the slope might well have retained its basic stability while undergoing substantial lateral displacements resulting in serious slumping of the upstream slope of the dam. Therefore, it would appear that many hydraulic fill dams can be expected to withstand quite strong shaking of short duration without seriously detrimental effects or that earth dams constructed with soils having higher strengths than hydraulic sand fills can withstand strong shaking similar to the type developed in the San Fernando area during the 1971 earthquake without hazardous consequences.

Stability Analysis Using Different Failure Criteria.—In the stability analysis described previously, the stability of the embankment was first evaluated using the stresses required to cause 5% strain and liquefaction (pore pressure equal

to effective confining pressure during cyclic loading) as an initial failure criterion, because beyond this point, the test data showed that strains developed rapidly indicating a need to consider progressive failure effects. It has sometimes been suggested that higher strain levels, on the order of 15%-20%, should be used for analysis purposes. The use of such higher strain criteria would tend to increase the apparent safety of an embankment by: (1) Requiring higher stresses to cause the higher strain levels specified; and (2) eliminating the need for consideration of progressive failure in an embankment. While such considerations may be appropriate for some types of soils, their use for soils that increase in strain relatively rapidly once initial liquefaction has occurred may lead to an overestimate of the actual safety of an embankment dam.

To explore this possibility, an analysis of the stability of the Lower San Fernando Dam was made using a failure criterion of 20% strain in the cyclic triaxial compression tests. For this condition it was found from the test data that the stresses required to cause 20% strain were about 15% higher than those required to cause 5% strain. Thus, the data shown in Fig. 3, increased by 15%, could be used for analysis purposes.

Using the modified Pacoima motion as a base excitation, the stress distribution in the embankment was determined and the values compared with those required to cause 20% strain. This analysis showed a very limited zone in which failure would occur. The limited extent of these zones would undoubtedly be considered indicative of adequate performance if the real behavior of the upstream slope had not been known.

Accordingly, the use of higher strain criteria, without consideration of the possible effects of progressive failure, may sometimes lead to incorrect assessments of embankment performance. In particular, this study would seem to indicate the care required in interpreting the results of cyclic load tests for use in evaluations of embankment stability during earthquakes.

Dynamic Analysis of Stability of Upper Dam during San Fernando Earthquake

Following exactly the same procedure as that described previously, a dynamic analysis was also made of the stability of the Upper San Fernando Dam. However, because there was no record of rock motions in the immediate vicintiy of the dam available in this case, the modified Pacoima record shown in Fig. 4 was considered to be an appropriate representation of the probable base excitation.

Following the same procedures as before and considering the zone where computed strains exceed 10% to make no contribution to the overall embankment stability, the factor of safety against a downstream slide was computed to be about 1.75. Thus, despite an extensive zone of liquefaction or high pore pressure developed by the earthquake shaking, the analysis indicated that the embankment would easily be able to withstand the small inertia forces developed later in the earthquake, together with the static stresses, without developing a residual downstream instability condition.

Analysis of the shear strain potential for the Upper Dam indicated an average shear strain potential of about 12%-16%, indicating a relative horizontal downstream movement of the crest and berm of about 4.5 ft-6 ft (1.4 m-1.8 m). This is in excellent accord with the observed downstream movement of the crest of about 5 ft (1.5 m). Clearly, this high degree of agreement is fortuitous but it does indicate the potential of the analysis procedure for evaluating the stability and deformation of dams of this type.

Analysis of Stability of Lower Dam for Magnitude 6.5 Earthquake 20 miles Away

During the San Fernando earthquake, several other hydraulic fill dams (Fairmont, Lower Franklin, and Silver Lake Dams) were subjected to strong shaking effects. These dams were located about 20 miles (32 km) from the epicentral region of the earthquake and records of ground motions indicate that the maximum accelerations in rock at this distance were probably reduced to about 0.2 g. However, despite this shaking intensity, none of these dams suffered any significant damage.

Accordingly, it was considered of interest to perform a dynamic analysis of the Lower San Fernando Dam to determine how it might have behaved if the earthquake of February 9, 1971 had been centered 20 miles (32 km) away. Appropriate rock motions having a maximum acceleration of 0.2 g were used for this analysis and it was found that although the computed crest acceleration for the dam was about 0.3 g, at no point in the embankment would the soil have developed a condition approaching liquefaction or 5% strain.

Thus, the analysis procedures would show that no damage would be expected to the Lower Dam from a magnitude 6-1/2 earthquake occurring at a distance of about 20 miles (32 km) and similar results would be expected for other dams of similar construction; this was in fact borne out by the performance of Fairmount, Lower Franklin, and Silver Lake Dams.

Note that both field performance and analytical studies show that the seismic stability of a hydraulic fill dam is not determined only by its composition and configuration, but also by the intensity of shaking to which it is subjected. Thus, a dam that may be unsuitable for a highly seismic region may be entirely safe if it is located in a region of more moderate seismicity. In this respect the level of the water in the reservoir also has a major effect on the stability and increases in stability can be effected by limiting the reservoir level.

Conclusions

The events associated with the performance of the Upper and Lower San Fernando Dams during the earthquake of February 9, 1971 indicate that a major catastrophe was narrowly missed. Accordingly, the study previously described was undertaken to determine the adequacy of existing analytical procedures to predict slide movements of this type and whether new methods and criteria are required for evaluating the seismic stability of earth dams.

In an effort to keep the investigation procedures as close as possible to those that might be employed in standard design practice, the boring and sampling were performed by the personnel of the State of California Department of Water Resources and the soil testing was performed in the laboratories of the Los Angeles Department of Water and Power and the State of California Department of Water Resources. A limited program of cyclic load testing was conducted in a university laboratory to expedite the testing program, but check tests were performed to ascertain that the results obtained were similar to those that might have been determined in the other laboratories.

Thus, the test data used in the analyses was that provided by responsible design agencies in accordance with their normal engineering procedures. Using this information, analyses were made of the stability of the Upper and Lower San Fernando Dams as described previously. It is believed that the following conclusions are warranted by the results of the investigation:

1. Dynamic analyses of the response of the San Fernando dams appear to provide a satisfactory basis for assessing the stability and deformations of the embankments during the earthquake.

2. Analysis of the stability of the Lower San Fernando Dam for the motions resulting from a magnitude 6.5 earthquake occuring at a distance of 20 miles (32 km) shows that no damage would be expected. This latter result is in good accord with the observed performance of the Fairmont, Lower Franklin, and Silver Lake dams during the February 9th earthquake.

3. Both the analytical results and the field performance of hydraulic fill dams during the February 9th earthquake show that the seismic stability of hydraulic fills is not determined only by their void ratios or relative density, but also by the nature, intensity, and duration of the seismic shaking.

4. The fact that the dynamic response analysis procedure was in the investigation gives reasonable evaluations of the performance of three different hydraulic fill dam conditions: (a) Lower San Fernando Dam (major slide in upstream slope); (b) Upper San Fernando Dam (significant movement downstream); and (c) Fairmont, Lower Franklin, and Silver Lake Dams (no significant damage) gives some degree of confidence in the ability of the method to anticipate the performance of other structures subjected to different shaking intensities. Because of the analytical simplifications required, however, good engineering judgment must be exercised in the selection of soil characteristics for use in the analyses, in the detailed steps followed to conduct the analyses, and in the evaluation of the results obtained. Hopefully, the degree of judgment required will be reduced as further developments in soil testing and analysis procedures are introduced. In the meantime, the detailed procedures used in this study should provide a useful guide for other evaluations. However, it is emphasized that other procedures may well be applied and give acceptable results in many cases. It would seem desirable, however, that any analytical procedures used to evaluate the seismic stability of embankment dams should have the capability of correctly predicting the performance of hydraulic fill dams in the San Fernando earthquake.

On the basis of the good results obtained in applying dynamic analysis procedures to the aforementioned dams, as well as other dams affected by earthquakes (Sheffield Dam, Seed, et al., 1971; Dry Canyon Dam, Lee and Walters, 1974), it seems reasonable to conclude that dynamic analysis methods provide the engineer with a new tool to add to his present collection and thereby make improved evaluations of the anticipated performance of critical structures meriting this type of treatment.

At the same time it should be recognized that even new and more sophisticated tools must be used with skill if they are to produce the desired results. If the components making up the tool are weak in any respect, the results can be grossly misleading. The proper use of dynamic analysis procedures for earth structures where major changes in pore-water pressures may occur during earthquake shaking involves a number of essential components: (1) A sufficiently intensive investigation of the embankment to establish its composition and the recognition of representative soil types; (2) a good analysis of the preearthquake static stresses in the embankment; (3) a good analysis of the dynamic stresses induced in the embankment by the design earthquake; (4) a series of cyclic load tests on representative samples to determine the changes in pore-water pressure and the deformations likely to occur in the soils; (5) an acceptable interpretation of the analytical results and test data to assess the extent of deformations likely to occur in the embankment; and (6) the exercise of judgment at all stages of the study and in evaluating the final results of the analyses, before a final judgment on the safety of the dam is made.

If the dynamic analysis is deficient in any of these respects, it may well be misleading and thereby of little or no value. Thus, if any of the steps cannot be performed appropriately, or replaced by appropriate judgment, it may often be better not to perform the analysis at all.

On the other hand, performed by knowledgable engineers, there seems to be no reason why all of the steps cannot be performed with a sufficient degree of accuracy at the present time that the overall results are extremely useful in the final assessment of seismic stability.

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