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SYNOPSIS Analyses are presented of the anticipated earthquake performance of three generally similar earth dams, situated in a seismically active area of northern California. The series of analyses illustrates a cost-effective approach which involved full-scale finite element analyses of one dam, and the use of limited dynamic analysis techniques and correlations to evaluate the other two. The simplified techniques were applied only after testing them against the finite element analyses, information is also presented on how the results of simplified and full-scale dynamic analysis procedures correlate.

INTRODUCTION

Evaluation of the seismic adequacy of earth dams using modern sophisticated techniques can be a very expensive process. On the other hand, overly-simplified methods, such as the "pseudo-static" analyses in vogue before the advent of sophisticated technology. while sometimes informative, are not very helpful in evaluating adequacy where seismic performance is seriously questioned. Simplified dynamic analysis techniques have been developed which are often amenable to meaningful interpretation and which do not entail the expeditures of time and money required for full-scale analyses. However, there are many situations which still necessitate full-scale progams utilizing finite element techniques, at least until sufficient correlative information is available to demonstrate that more limited techniques will suffice. Therefore, it is beneficial to review the results of studies which provide comparisons of seismic performance calculated by both full-scale and simplified analyses.

This paper describes the investigation of adequacy of three earth dams situated in a highly seismic area of northern California. Because of the many similarities between these dams, a program was devised which involved intensive investigations and analyses of one dam, and use of the results as a basis for correlative evaluations of the other two. Simplified dynamic analysis techniques were applied to the latter two dams, after first testing these limited techniques against the results of the more rigorous analyses.

DESCRIPTIONS OF DAMS

The Santa Clara Valley Water District owns and operates several dams in Santa Clara County, California, generally south of San Francisco Bay. Because of the highly seismic nature of this region, and in recognition of advances in earthquake engineering technology, the District has undertaken a program of seismic reevaluation of its dams. Three of these dams--Almaden, Calero and Guadalupe--are situated in the same general area, in proximity to the great San Andreas fault (Fig. 1). All three dams were constructed in the mid-1930's, designed by the same engineer to similar standards with similar zoning, and founded on the same geologic complex. Embankment materials were obtained from soil deposits that were generally similar in nature, although investigation did reveal there were some differences in average materials characteristics and densities among the three dams. The dams

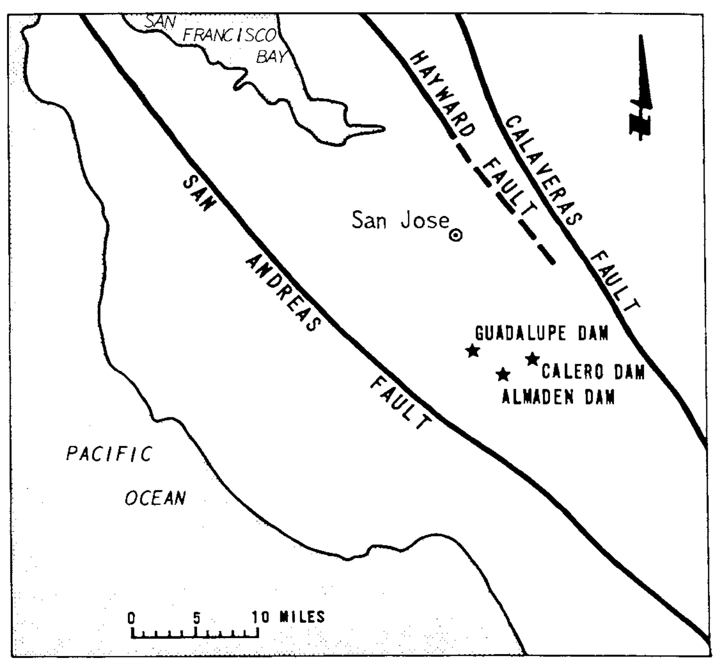

Fig. 1. Vicinity Map

range from 100 to 140 feet in height and have identical slopes, except that a berm was added at the upstream face of Guadalupe Dam in 1972 following a drawdown slope failure.

As shown on Fig. 2, the zoning of all three dams is quite simple--impervious upstream zones were achieved by routing finer-grained materials to those areas, while pervious zones received the coarser materials. The designation "pervious" is relative only, since the materials as compacted generally contained fairly significant amounts of fines. Consequently, locations of the phreatic surfaces were important variables to consider in making the assessments of earthquake adequacy. This was one of the key questions addressed by the field program at each dam. It was also necessary that field and laboratory investigations be sufficient in scope to define, within reasonable limits, the response and strength characteristics of embankment materials.

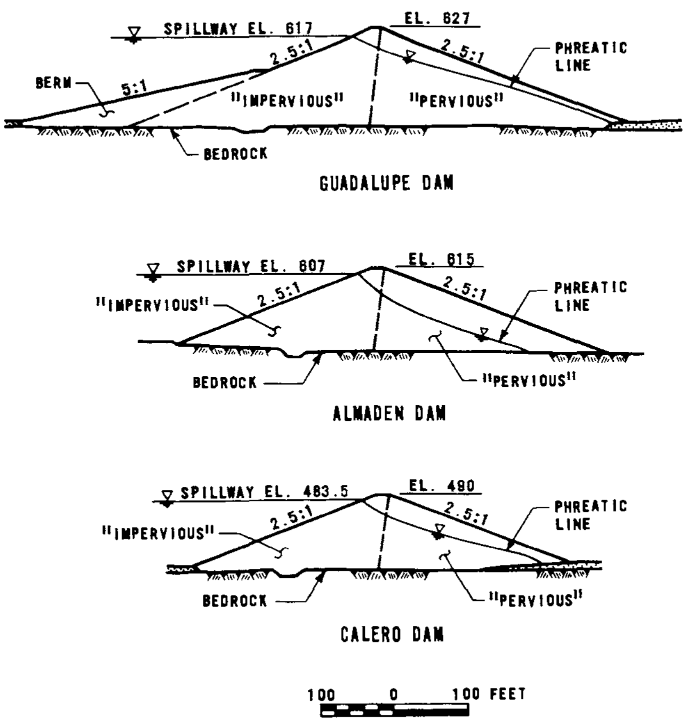

Fig. 2. Typical Sections of Guadalupe, Almaden and Calero Dams

SEISMICITY

The three major active fault structures in the region are the San Andreas, Calaveras and Hayward fault zones (Fig. 1). Postulated maximum credible earthquakes on each, and their respective distances from the three dams, are shown in Table I. Although other local faults exist in the area, these three faults have the potential of generating earthquakes that would produce the most severe ground shaking at all three damsites. The characteristics were compared of earthquake motions likely to be experienced at the damsites due to maximum credible earthquakes occurring on the three faults, and it was concluded that the San Andreas event would produce by far the most severe effects on all three dams.

TABLE I. Maximum Credible Earthquake Magnitudes (M) and Distances from Dams

Dam	Distances from Dams (in Miles) San Andreas Fault (M = 8½)	Hayward Fault (M = 7½)	Calaveras Fault (M = 7½)
Almaden	6	9½	10½
Calero	8½	7	8½
Guadalupe	6	10	12

A synthetic accelerogram proposed by Seed and Idriss (1969) was chosen as a basis for representing the time history of motions of a maximum credible earthquake on the San Andreas fault. For purposes of the analyses, this accelerogram, which has a duration of 75 seconds, was scaled to produce peak bedrock accelerations of 0.58g at Almaden and Guadalupe Dams, and 0.52g at Calero Dam.

FIELD AND LABORATORY INVESTIGATIONS

Field Program

The field investigation included fifteen borings at the three dams to identify embankment materials and zoning, recover 6-inch diameter undisturbed samples and, in the case of Guadalupe Dam, provide for in-situ measurement of shear wave velocities. Pneumatic piezometers were selectively installed in the bore holes to permit locating the phreatic lines within the dams.

The field investigation revealed that the embankments were basically homogeneous and mainly consisted of well compacted clayey sand with gravel. Piezometer readings indicated that the phreatic line in Guadalupe Dam was the highest. The phreatic line was lowest in Almaden Dam, probably because the materials in the downstream zone were somewhat coarser than those in the downstream zones of the other two dams.

To determine dynamic moduli of the embankment materials at low strain level, cross-hole shear wave velocity measurements were made in borings drilled from the crest at Guadalupe Dam. The shear modulus parameters, K_{2max}, computed from the measured shear wave velocities, were relatively uniform throughout the entire depth, and averaged approximately 100.

Laboratory Testing

Laboratory testing for all three dams included classification, static and dynamic strength and resonant column tests. With the exception of cyclic shear strength data, the material properties determined from these tests are

TABLE II - Summary of Material Properties

Property	Guadalupe Upstream	Guadalupe Downstream	Almaden Upstream	Almaden Downstream	Calero Upstream	Calero Downstream
Avg. Dry Unit Weight (pcf)	125	122	118	126	123	131
Gradation (%)						
> No. 4	17 - 32	17 - 40	6 - 44	25 - 62	3 - 33	17 - 43
< No. 200	30 - 44	20 - 46	17 -.55	6 - 35	24 - 60	15 - 56
Plasticity						
L.L.	30 - 43	32 - 39	35 - 49	-	30 - 35	32 - 41
P.I.	12 - 25	12 - 19	14 - 30	-	11 - 17	15 - 22
Predominant Material Classification	SC	SC	SC	SC-GP	SC	SC
Permeability (cm/sec)	5×10^{-9}	5×10^{-9}	6×10^{-9}	9×10^{-8}	1×10^{-7}	6×10^{-7} to 3×10^{-9}
Compaction Test						
Max. dry density (pcf)	126	123	121	126	119	135
Opt. moisture content (%)	11.0	12.1	13.2	12.7	13.9	9.3
Relative Compaction (%)	99	99	97	100	100+	97
Strength						
ICU c' (ksf)	0	0	0.2	0.4	0.5	0.5
ϕ' (deg.)	30	37	30	38	31	31
c (ksf)	0.1	0.7	0.3	0.5	0.4	0.4
ϕ (deg.)	19	16	15	17	16	16
UU Su (ksf)	0.5 -1.1	0.5 - 0.8	1.6 - 2	1.0 - 2.7	2.0	2.0
K_{2max}	94 -116	67 - 97	69 - 77	122 - 193	96 -131	114 - 168

summarized in Table II. The data presented in this table clearly indicate that, although some minor variations exist, the embankment material properties for all three dams are generally very similar. The exception is Alamaden Dam, where material in the downstream zone is noticeably coarser.

Cyclic triaxial tests were performed on samples recovered from all three dams, consolidated under both isotropic and anisotropic stress conditions, with lateral consolidation pressures ranging from 20 to 80 psi. The results of these tests, presented in terms of cyclic shear stress required to cause 10 percent strain in 30 cycles, are shown on Fig. 3. As is typical for well-compacted clayey materials, the samples did not show any noticeable change in straining pattern during cyclic loading after pore water pressure reached 100 percent of the confining pressure.

Post-cyclic, static undrained triaxial tests were also performed on samples from all three dams to determine the extent of strength reduction of the embankment materials due to cyclic straining. The test results (presented in Table III) show no appreciable change in static shear strength after cyclic loading.

To assess the shear modulus parameters of the embankment materials, resonant column tests were performed on samples from all three dams. The K_{2max} values obtained for Guadalupe Dam are in remarkable agreement with those computed from the shear wave velocities measured in the field. This agreement has been common in our experience with other dams in this locality.

ANALYSES

Both rigorous and simplified methods were utilized to determine the earthquake-induced deformations in Guadalupe Dam. The results of analyses by both methods were compared and used as a basis for correlative evaluations of the other two dams.

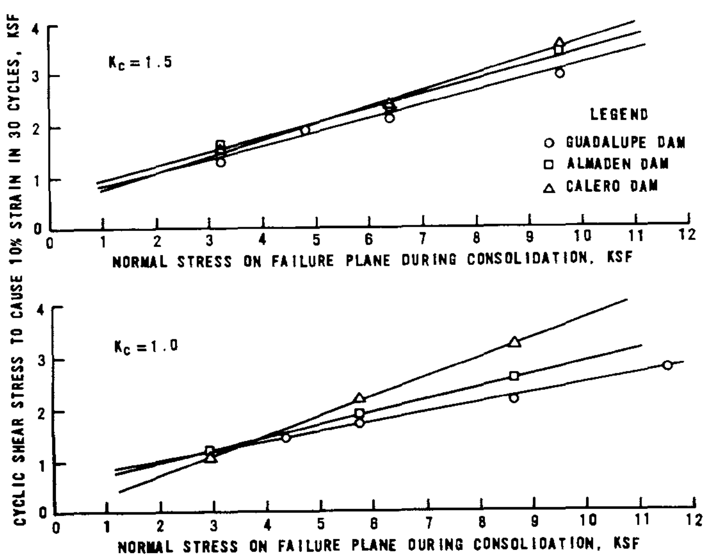

Fig. 3. Cyclic Shear Strength of Embankment Materials for Guadalupe, Almaden and Calero Dams

TABLE III. Shear Strength Prior to and After Cyclic Loading*

	Guadalupe	Almaden U/S	Almaden D/S	Calero
Prior to Cyclic Loading:				
c' (ksf)	0	0.18	0.39	0.48
ϕ' (deg)	37	30	38	31
c (ksf)	0.10	0.26	0.45	0.40
ϕ (deg)	19	15	17	16
After Cyclic Loading:				
c' (ksf)	0	0.43	0.39	0
ϕ' (deg)	36	30	38	40
c (ksf)	0.30	0.72	0.45	0.22
ϕ (deg)	24	15	24	23

*Cyclic Strains Ranged from 5 to 10%

Finite Element Analysis

The response of Guadalupe Dam was analyzed using the computer program QUAD-4 (Idriss, et al. 1973). The computed maximum acceleration at the crest was 0.77g, or an amplification of about 1.3 times the maximum bedrock acceleration.

The induced shear stresses obtained from the response analysis were then converted to equivalent uniform cyclic shear stresses, and the "strain potential" for every element was calculated by comparing the stress required to cause certain specified strains with the induced equivalent uniform cyclic shear stress.

However, strain potential is physically meaningless because any development of compressive or shear strains will be constrained by the surrounding elements. To integrate the potential strains in the embankment into a compatible deformation pattern, the finite element program DEFORM (Serff, et al. 1976) was used. The results of the computation are plotted on Fig. 4. The computed vertical deformation at the dam crest was about 7.2 feet and the maximum lateral displacement, occurring on the upstream slope, was about 8.7 feet.

Correlative Study

Most simplified methods are extensions and refinements of various forms of the method proposed by Newmark (1964). The concept involved in this type of analysis is that movements in the embankment would begin to occur if the earthquake-induced inertia forces on a potential sliding mass were greater than the yield resistance, and that movements would stop when the inertia forces were reversed. Therefore, the acceleration at which the inertia forces become sufficiently high to cause yielding to begin (yield acceleration) is first computed. The displacements are then computed by double integration of the effective acceleration on the sliding mass in excess of this yield acceleration. Subsequent studies conducted by Seed and Martin (1966) and Ambraseys and Sarma (1967) have greatly improved the understanding of the dynamic response of embankments to seismic loads.

Two simplified procedures for assessing earthquake-induced embankment deformations have been developed in recent years. The method proposed by Makdisi and Seed (1978) utilizes the concept developed by Newmark (1965), but considers the deformable nature of an earth structure when subjected to earthquake excitation. The method proposed by Sarma (1975, 1979) adopts a similar approach, but uses the assumption of a rigid block on an inclined plane, rather than a deformable body.

Both the Makdisi-Seed and the Sarma method require determination of crest acceleration, variation of maximum acceleration with depth, fundamental period and yield acceleration. To determine yield acceleration, slope stability analyses were conducted for the critical sliding surfaces at various depths in the embankment. Consolidated-undrained shear strength was used and various seismic coefficient values were applied in the analysis. For any potential sliding surface, the yield acceleration was considered to equal the value of the seismic coefficient at which the factor of safety is unity.

For the Makdisi-Seed procedure, the approximate crest acceleration and fundamental period were calculated using the computer program SHAKE (Schnabel et al. 1972). The variation of maximum acceleration with depth was then determined using the summary curve prepared by Makdisi and Seed.

For the Sarma procedure, the maximum accelerations at various depths were determined using a plot (developed by Sarma) which represents the upper bound of the response amplification curves for an embankment-foundation model subjected to nine strong motion accelerograms. The fundamental periods were computed using a procedure also presented by Sarma (1979).

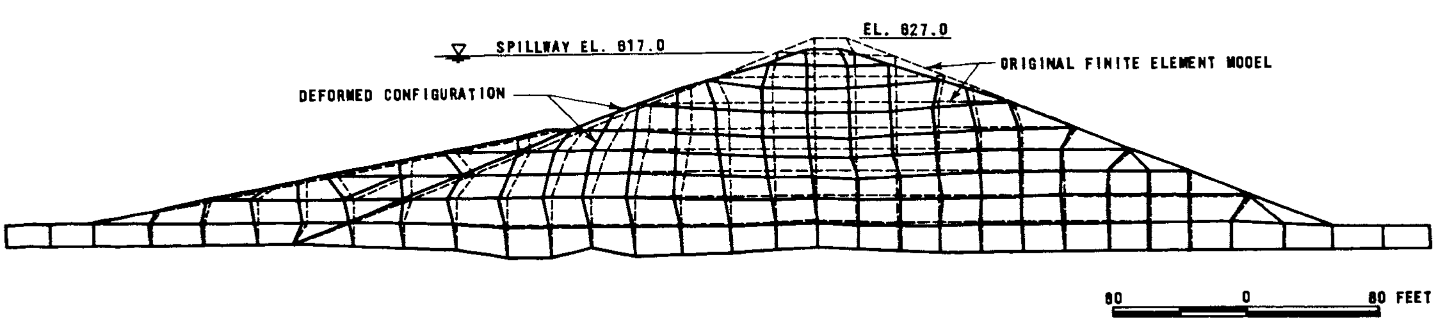

Fig. 4. Computed Earthquake-Induced Deformation in Guadalupe Dam

Both these simplified methods were used to compute embankment deformations for, Guadalupe Dam, and the computed displacements were compared with those obtained by the finite element analysis. The results of the comparison were then used as a basis for predicting the earthquake-induced deformations for Almaden and Calero Dams.

RESULTS

The results of analyses using both the finite element analysis and simplified methods are summarized in Table IV. The agreement between computed displacements for Guadalupe Dam using the finite element method and the Makdisi-Seed method is remarkable. The maximum horizontal displacement computed by the latter method was 7.2 feet, occurring about 50 feet below the dam crest. As shown on Fig. 4, the maximum horizontal displacement computed by the finite element method also took place at a depth about 50 feet below the crest, and the computed displacements at this depth ranged from 6.2 to 8.7 feet with an average value of about 7.5 feet. It should be noted that, since SHAKE is a one-dimensional analysis which assumes level ground and free field wave propagation, the maximum acceleration and fundamental period thus computed would not be the same as those obtained from the finite element analysis. A comparison of the results obtained by SHAKE and QUAD-4 analyses indicated that the maximum crest acceleration and fundamental period computed by SHAKE were somewhat lower. Nevertheless, in spite of the approximations exercised in determination of parameters and some simplifying assumptions involved in the development of the analytical procedure, the maximum displacement computed by the Makdisi-Seed method was practically the same as that obtained from the finite element analysis.

TABLE IV. Computed Embankment Deformations*

Method	Guadalupe	Almaden	Calero
QUAD-4 and DEFORM:			
Crest Settlement	7.3		
Max. Horiz. Displacement	6.2-8.7		
at Depth (Below Crest)	50		
Makdisi & Seed (1978):			
Max. Displacement	7.2	1.7	1.5
at Depth (Below Crest)	50	55	40
Sarma (1979):			
Max. Displacement	12.3	2.8	3.9
at Depth (Below Crest)	50	55	80

*in feet

The maximum horizontal displacement for Guadalupe Dam computed by the Sarma method was significantly higher than that computed by the finite element method. The maximum displacements computed for all three dams by the Sarma method were also consistently higher than those computed by the Makdisi-Seed method. The probable explanations are that the Sarma method uses the assumption of a rigid block on an inclined plane (rather than a deformable body), and that the maximum crest acceleration used in the analysis was obtained from the upper bound of the response amplification curves for a group of nine strong motion records. However, because the deviation was consistent, the results obtained from this method could still be used as a guide to predict the maximum displacements of the other two dams. It was possible to compute the ratio of the maximum diplacement for Guadalupe Dam determined by this method to that determined by the finite element method and the same ratio was used to estimate the maximum displacements for the other two dams.

Based on the results of both the Makdisi-Seed and Sarma methods, the maximum horizontal displacements at both Almaden and Calero Dams were estimated to be about 2 feet. Since the finite element analysis for Guadalupe Dam showed that the crest settlement would be of the same order as the maximum horizontal displacement, crest deformations at Almaden and Calero Dams were estimated to be 2 feet.

Although strain potential calculations were not required for the simplified methods, it was of interest to determine if the anticipated strain potentials were within the range where a normal stress-strain relationship holds. Since the seismic resistance depends to a great extent on the effective normal stress on any failure plane, it was essential to assess realistically the normal stresses in the embankment.

In a finite element analysis, such as QUAD-4, stress-strain compatibility of all adjacent elements is satisfied during computation. In addition, because of the two-dimensional nature of the analysis, it is possible to account for the effects of anisotropy and seepage forces. However, in a one-dimensional analysis, (such as SHAKE), only effective vertical overburden pressures can be used in the computation.

To overcome this deficiency, the results of finite element analyses on several dams were reviewed. This review indicated that the increase in effective vertical stress due to seepage forces generally ranged from 20% to 40%, with an average increase of about 30%. Furthermore, the ratio of shear stress to vertical stress (α-value) was generally found to range from 0 to 0.2 in the upstream portion of the embankment, instead of zero as is assumed in the one-dimensional analysis. Without taking these factors into account, the one-dimensional analysis would greatly underestimate seismic resistance of an embankment.

Based on the above considerations, adjustments were made to the effective vertical stresses in the Almaden and Calero Dam embankments to compensate for the effects of anisotropy and seepage forces. The results of the one-dimensional analysis with the above-mentioned adjustments showed that the average strain potentials were on the order of 10 percent for Almaden Dam and less than 10 percent for Calero Dam.

CONCLUSIONS

The results of the study indicate that simplified methods show promise of having widespread application to the prediction of earthquake-induced deformations of dams, provided they are used judiciously. Such techniques are particularly appropriate, and cost-effective, for the evaluation of groups of dams having somewhat similar configurations and characteristics, where there is an opportunity to test these methods against more rigorous analyses.

In the particular case studied, the horizontal deformations calculated using procedures developed by Makdisi and Seed were in remarkable agreement with those computed by finite element analyses.

Deformations calculated by the Sarma method were consistently higher than those calculated by the Makdisi-Seed method, and were also higher than those obtained by finite element analyses. This consistent deviation is probably due to the rigid body assumption utilized in development of the method and the selection of high response amplification for the determination of crest acceleration. However, because the deviation was consistent, there is reason to believe that results obtained by the Sarma method can be used as a guide to predict maximum displacements of dam embankments which exhibit reasonably similar site conditions and material properties, provided the method is tested against more rigorous analyses and appropriate adjustments are made.

Both simplified methods produced results which are generally in accord with what would be expected, on the basis of what is known about the three dams and experience with analysis of other dams. The numerical relationships of computed deformations for the three dams appear to be of the proper order, considering locations, heights, internal conditions and material strengths.

In a simplified, one-dimensional determination of strain potential, it is important to account for two-dimensional effects, such as seepage forces and anisotropic stress conditions, when computing normal stresses or assessing dynamic strength. Otherwise, this type of analysis will greatly underestimate seismic resistance of an embankment.

The collection, comparison and publication of data showing results of case-history studies using both rigorous and simplified techniques would be most helpful in hastening the time when cost-effective, simplified methods can be broadly applied with reasonable confidence.

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