DYNAMIC RESPONSE ANALYSES OF COGSWELL DAM DURING THE 1991 SIERRA MADRE AND 1987 WHITTIER NARROWS EARTHQUAKES

R. W. Boulanger [1], J. D. Bray [2], S. M. Merry [3] and L. H. Mejia [3]

ABSTRACT

The recorded strong motions at Cogswell Dam during the 1991 Sierra Madre and 1987 Whittier Narrows earthquakes provide a valuable opportunity to investigate and evaluate the accuracy and reliability of conventional geotechnical procedures for evaluation of the dynamic response characteristics of rockfill dams and dams with highly three-dimensional geometries. The Sierra Madre Earthquake (M_L = 5.8) produced a significantly stronger level of shaking than the more distant 1987 Whittier Narrows Earthquake (M_L = 5.9) and thus the recorded accelerograms provide an excellent opportunity to investigate the dynamic properties of the rockfill over the imposed range of earthquake loads.

INTRODUCTION

Cogswell Dam, formerly San Gabriel Dam No. 2, is a 280-foot high concrete-faced, dumped rockfill dam founded on bedrock in a narrow notch-shaped canyon (Fig. 1). The dam is located approximately 20 miles north of Whittier, California, in the San Gabriel Mountains on the West Fork of the San Gabriel River and is owned and operated by the Los Angeles County Flood Control District (LACFCD). The dam retains Cogswell Reservoir, with a capacity of 8850 acre-feet, for the purposes of flood control and water conservation. The spillway and outlet works are both located on the right abutment. Construction of the dam and related facilities was partially completed in April, 1934, and fully completed in 1948 when the permanent concrete facing was installed.

The dam was recently shaken by two significant earthquakes - the 1987 Whittier Narrows and 1991 Sierra Madre earthquakes. The epicenter of the 1987 Whittier Narrows Earthquake (M_L=5.9) was 18 miles southwest of the dam and the epicenter of the 1991 Sierra Madre Earthquake (M_L=5.8) was 2.3 miles northwest of the dam. Maximum horizontal accelerations at the crest of the dam were about 0.15 g and 0.49 g for these two events, respectively. Records of the earthquake motions were obtained by seismographs located at the center and right side of the dam crest, and on the right abutment above the dam crest.

These recorded motions provide an excellent opportunity to: (1) investigate the accuracy of two- and three-dimensional dynamic response analyses for predicting the response of highly three-dimensional dams; and (2) complement the limited data available on dynamic properties of rockfill materials by back-calculating the dynamic properties of the rockfill over a range of earthquake-induced shear strains. This paper presents the findings of the two-dimensional analyses and describes the three-dimensional analyses in progress.

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[1] Department of Civil Engineering, University of California at Davis

[2] Department of Civil Engineering, University of California at Berkeley

[3] Woodward-Clyde Consultants, Oakland, California

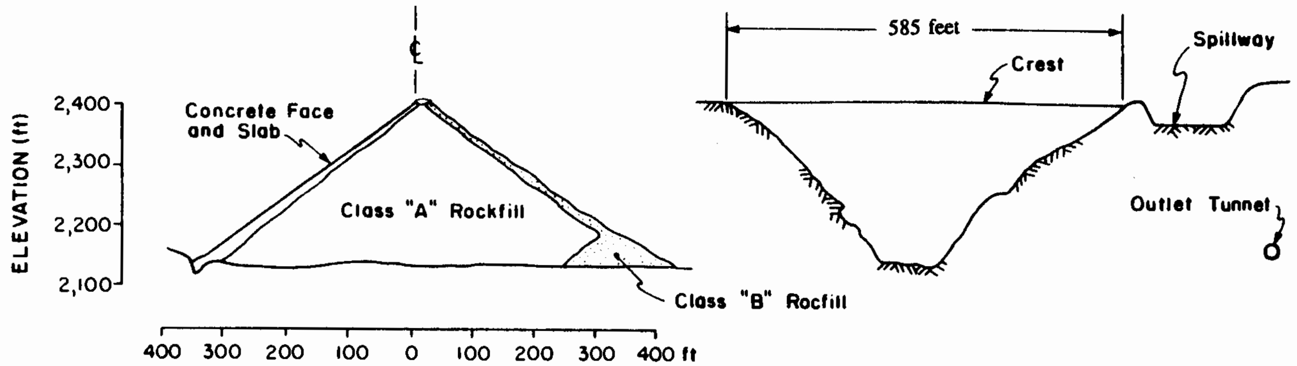


Fig. 1. Transverse and longitudinal cross-sections of Cogswell Dam

EMBANKMENT AND FOUNDATION MATERIALS

Information and data on the embankment and foundation materials were obtained from previous engineering studies, safety reviews, and construction records for the dam as maintained in the files of the Division of Safety of Dams, State of California. Recent summaries of these data are contained in reports by the Los Angeles County Flood Control District (1980) and the State of California (1986).

Cogswell Dam was constructed as a largely homogenous dumped-rockfill dam with an upstream concrete facing slab and concrete cutoff wall. Three classes of rockfill were specified in construction of the dam, as shown in Fig. 1. Class "A" rockfill comprises the main body of the dam and was a well graded mixture with the following specifications by weight: 40% from quarry chips to 1,000 lbs; 30% between 1,000 lbs and 3,000 lbs; 30% between 3,000 lbs and 14,000 lbs; and no more than 3% quarry dust. Class "B" rockfill was used to place both a 50-foot high downstream toe and a downstream facing layer varying from 8 feet thick at the crest to 12 feet thick at the toe. Class "B" rockfill was a heavier specification, with one-half to exceed 14,000 lbs in weight. Class "C" rockfill was used to place an upstream facing layer varying from 6 feet thick at the crest to 15 feet thick at the toe. Class "C" rockfill ranged from quarry chips to 14,000 lbs and was to be derrick placed to the maximum possible density.

All rockfill material consisted of granitic rock obtained from a quarry located in Devil's Canyon, which is approximately 1.5 miles upstream of the left abutment. The quarried rock was to be sound, hard, durable, and unaffected by air and moisture. Quality control tests indicated an average compressive strength of 6,629 psi, an average unit weight of 174.7 pcf, and a 5.04% breakdown by a Rock Drop test developed for the project.

The main body of the embankment was placed by dry dumping of 25-foot lifts with no compaction or sluicing. The conventional practice of sluicing the rockfill was omitted due to the scarcity of water at the dam site. Following completion of the entire rockfill section in the Fall of 1933, construction began on the concrete facing with the intention of completing this work by the Spring of 1934. Heavy rains in December 1933 through March 1934 wetted the fill and led to large settlements which disrupted the facing already constructed and caused significant deformations of the dam. During one particularly severe rainstorm of December 31, 1933 the crest of the dam settled approximately 5.8 feet, and throughout the following four months of rain, the total settlement of the crest was as much as 13.6 feet. This led to a need to reshape the dam and reconstruct the upstream facing. A temporary timber facing was constructed and left in place for about 10 years until settlements had essentially ceased, at which time it was replaced by a reinforced concrete panel facing.

The foundation rock of Cogswell Dam is predominantly light-colored (augen) gneiss intruded by numerous dikes of andesite porphyry and hornblende amphibolite and dike-like masses of granophyric granite. Surficial weathering has reduced exposed portions of these rocks to relatively incompetent materials. During construction preparation of the dam foundation, the weakest rock was removed. The rocks within the prepared dam foundation are described as significantly jointed and sheared, moderately to strongly weathered, moderately hard, and moderately strong.

OBSERVED RESPONSE OF COGSWELL DAM

Instrumentation data for the dam includes piezometer data, drainage system discharge data, survey monument data, and strong-motion accelerograph data. The piezometer and drainage system discharge data indicate no significant effects as a result of the Whittier Narrows or Sierra Madre earthquakes. However, the reservoir was nearly empty for silt removal at the time of the Sierra Madre Earthquake and was only at about elevation 2266 feet (139 feet below the crest) at the time of the Whittier Narrows Earthquake. Survey monument data indicates that deformations of the crest of the dam were insignificant as a result of the Whittier Narrows Earthquake and attained maximum values of 1.61-inch vertically and 0.63-inch horizontally (downstream) as a result of the Sierra Madre Earthquake.

The Sierra Madre Earthquake was initially reported to have only caused minor transverse hairline cracks in the crest pavement and cracks along vertical joints in the parapet wall at three locations. Subsequent detailed inspections indicated that the earthquake caused cracking of the upstream concrete facing near its juncture with the concrete cutoff walls along both abutments. The observed zones of cracking ranged from 2 to 8 feet wide and extended from just below the parapet wall to about 35 feet down (vertically) on the right abutment and 15 feet down on the left abutment. Maximum crack widths of 0.5 inch were reported while more typical crack widths were less than about 0.25 inch. The Whittier Narrows Earthquake was not reported to have caused any damage to the dam.

A total of 3 strong motion accelerographs were installed at three locations on and near the dam as shown in Fig. 2. It should be noted that the accelerographs were temporarily removed during recent construction of a reinforced concrete parapet wall along the dam crest and upon re-installation (prior to the Sierra Madre Earthquake), were renumbered differently from the time of the Whittier Narrows Earthquake. At each location, motions were recorded in three orthogonal directions: transverse to the dam's axis, parallel to the dam's axis, and vertical. This paper will concentrate on the transverse motions as these are the motions generally considered to be of primary engineering interest. The acceleration time histories for the motions transverse to the dam's axis are presented in Fig. 3 for the Whittier Narrows and Sierra Madre earthquakes, respectively. The corresponding response spectra are shown in Figs. 4 and 6, and the corresponding Fourier amplification ratios between the center crest and abutment recordings are shown in Figs. 5 and 7. A summary of recorded motion characteristics during these events is provided in Table 1.

The two earthquakes excited Cogswell Dam at significantly different levels. The Whittier Narrows Earthquake produced transverse peak ground accelerations at the right abutment, right crest and center crest of 0.06 g, 0.10 g, and 0.15 g, respectively. Conversely, the Sierra Madre Earthquake produced higher transverse peak ground accelerations at these locations of 0.26 g, 0.32 g and 0.42 g, respectively. The observed crest to abutment amplification ratio for the Whittier narrows event was roughly 2.4; whereas, for

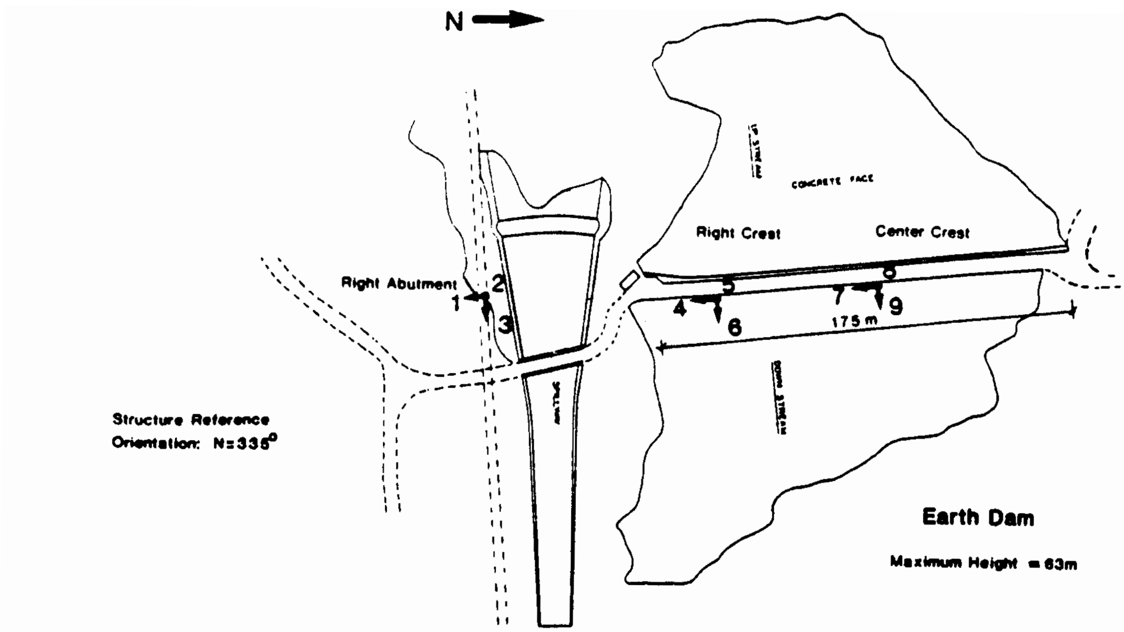
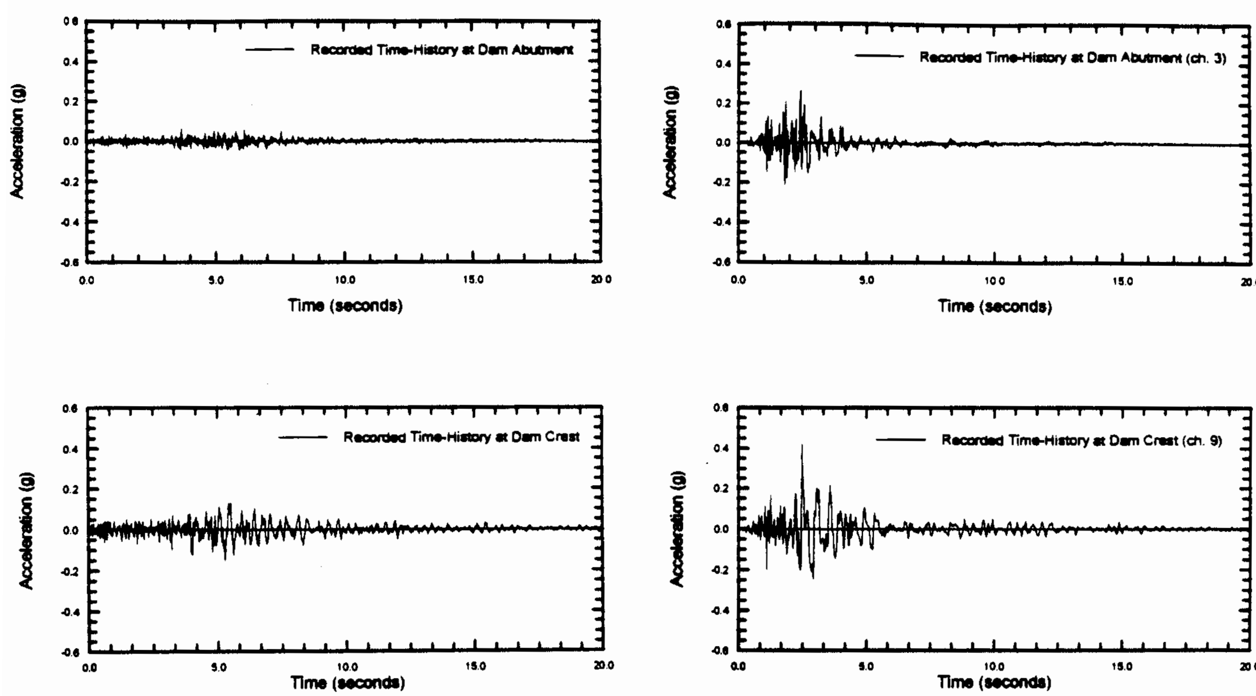


Fig. 2. Plan View of Cogswell Dam Showing Sensor Locations (CSMIP Station No. 23210)



(a) Whittier Narrows Earthquake

(b) Sierra Madre Earthquake

Fig. 3. Recorded Transverse Acceleration Time Histories

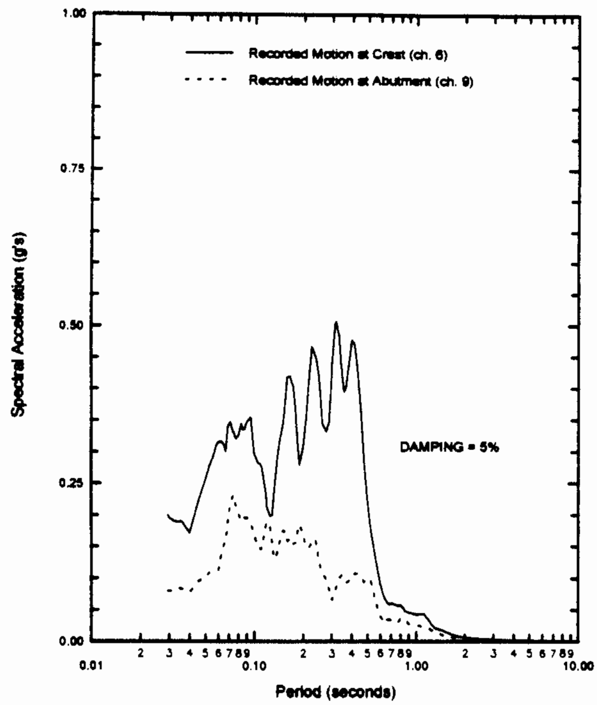


Fig. 4. Recorded Response Spectra: Whittier Narrow Earthquake

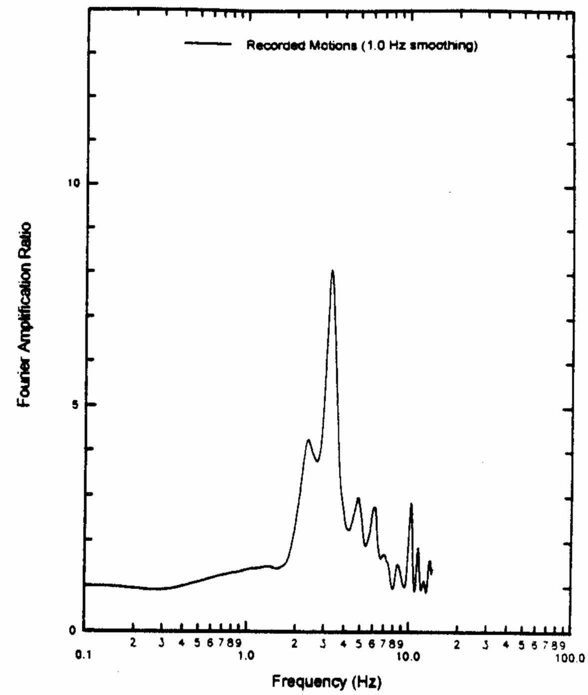


Fig. 5. Fourier Amplification Ratios: Whittier Narrow Earthquake

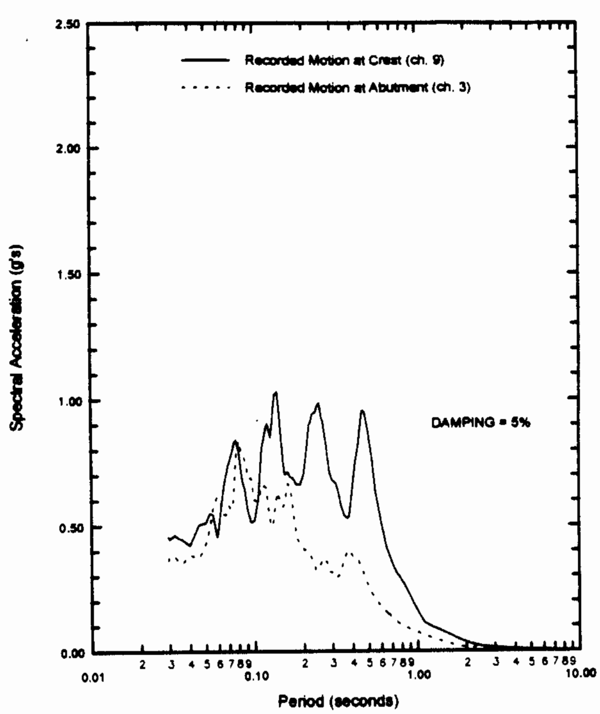


Fig. 6. Recorded Response Spectra: Sierra Madre Earthquake

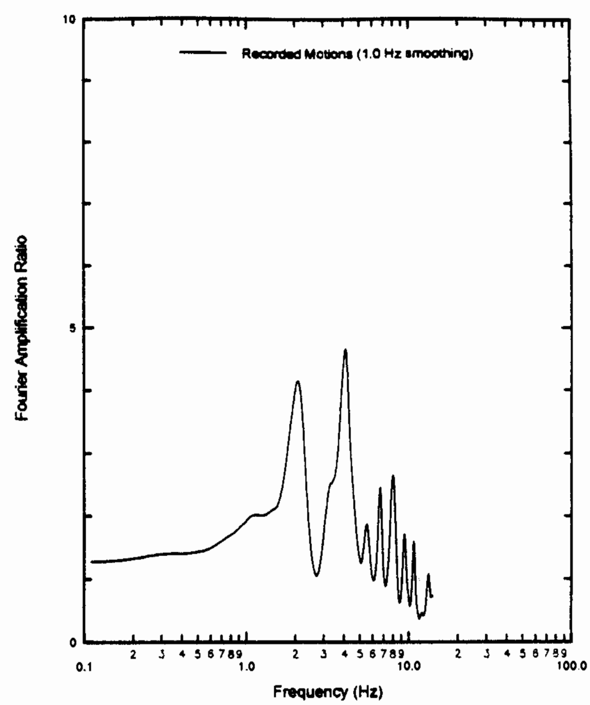


Fig. 7. Fourier Amplification Ratios: Sierra Madre Earthquake

TABLE 1. Characteristics of Earthquake Motions

		Whittier Narrows			Sierra Madre		
Accelerograph	Component	Peak Ground Accel.(1) (g)	Max. Spectral Accel.(1) (g)	Period at Max. Spectral Accel. (sec)	Peak Ground Accel.(1) (g)	Max. Spectral Accel.(1) (g)	Period at Max. Spectral Accel. (sec)
Abutment	Transverse	0.064	0.200	0.075	0.264	0.838	0.080
Right Crest	Transverse	0.100	0.367	0.190	0.318	1.082	0.227
Center Crest	Transverse	0.151	0.507	0.320	0.421	1.031	0.138
Abutment	Longitudinal	0.061	0.175	0.225	0.302	0.980	0.250
Right Crest	Longtiduinal	0.087	0.391	0.325	0.376	1.775	0.420
Center Crest	Longitudinal	0.137	0.385	0.320	0.486	1.647	0.435
Abutment	Vertical	0.06	--	--	0.227	--	--
Right Crest	Vertical	0.11	--	--	0.386	--	--
Center Crest	Vertical	0.14	--	--	0.484	--	--

(1)After baseline and instrument correction of accelerogram records.

the Sierra Madre event this ratio was 1.6. This decrease in the crest to abutment amplification ratio resulting from stronger earthquake shaking agrees with the findings of previous case studies (e.g., Harder 1991).

Three approaches were taken to evaluate the observed 3-D fundamental period of the dam during these earthquake events. For the Whittier Narrows Earthquake, the dam's fundamental period was estimated to be between 0.37 and 0.42 seconds based on: (1) the response spectra calculated for select sections of the crest accelerogram representing the initial period of decay of strong shaking suggested that the motion's predominant period was about 0.37 seconds; and (2) Fourier amplification ratios from the abutment to crest (Fig. 5) indicated that the recorded motion's predominant period was about 0.42 seconds.

For the Sierra Madre Earthquake, the dam's fundamental period was estimated to be between 0.45 and 0.48 seconds based on: (1) the response spectra calculated for select sections of the crest accelerogram representing the initial period of decay of strong shaking suggested that the motion's predominant period was about 0.45 seconds; and (2) Fourier amplification ratios from the abutment to crest (Fig. 5) indicated that the recorded motion's predominant period was about 0.48 seconds. Since the rockfill's stiffness degrades under stronger earthquake excitation, the fundamental period of Cogswell Dam is higher for the Sierra Madre Earthquake.

FINITE ELEMENT MODELS

Analyses of the initial static stresses in the dam (required for determining dynamic properties) were performed using the finite element method (FEM) program SSCOMPPC (Boulanger et al. 1991). SSCOMPPC employs the Duncan et al. (1980) hyperbolic soil model to represent the nonlinear stress-dependant stress-strain and volumetric strain response of the rockfill. The finite element mesh for the two-dimensional analyses of the maximum cross-section through the dam is shown in Fig. 8.

The two-dimensional (2-D) dynamic FEM analyses were performed using the computer program FLUSH (Lysmer et al. 1975). FLUSH is a 2-D FEM program for the dynamic response analysis of earth structures using the method of complex response to solve the equations of motion of a soil-structure system in the frequency domain. The nonlinear dynamic behavior of soils is modeled using the equivalent-linear method as proposed by Seed and Idriss (1970). After node and element renumbering, the same finite element mesh used for the initial static stress analyses was utilized for the 2-D dynamic response analyses.

The three-dimensional (3-D) dynamic FEM analyses were performed using a modified version of the computer program TLUSH (Kagawa et al. 1981). The original TLUSH program was developed to run on the now obsolete CDC 7600 main frame computer. Considerable effort has been put forth to develop a PC-compatible version of TLUSH which is currently being validated. The fully 3-D program TLUSH is similar to FLUSH in that it uses the method of complex response in the frequency domain and models soil behavior by the equivalent-linear method. The finite element mesh for the 3-D dynamic response analyses as shown in Fig. 9 has the same maximum cross-sectional geometry as in the 2-D mesh. A full mesh was used due to several asymmetrical features in the dam.

DYNAMIC ROCKFILL PROPERTIES

The dynamic properties of rockfill materials are not well documented or understood. It is customary to assume that rockfill behaves similar to cohesionless soils such as gravels or sands in terms of modulus degradation and damping characteristics. Thus, the dynamic shear modulus degradation relationships (G/G_{max} vs. shear strain relationship) and the damping ratio versus shear strain relationships as recommended by Seed et al. (1984) for gravelly soils were utilized to model the rockfill. It then remains to select a value for the parameter K_{2max} which establishes the maximum shear modulus (G_{max}) as:

$$G_{max} = 1000\ K_{2max}\ (\sigma_m')^{1/2} \tag{1}$$

where G_{max} and the mean effective confining stress (σ_m') are in units of psf. The value of σ_m' was taken as the average of the three principal stresses, of which two are obtained from the initial static stress analyses described in the next section. Since the 2-D static stress analyses do not provide the intermediate principal stress (σ_2'), it was estimated for each element as:

$$\sigma_2' = 0.35\ (\sigma_1' + \sigma_3') \geq \sigma_3' \tag{2}$$

The value of K_{2max} for the Class "B" and "C" rockfill zones were taken as 1/3 greater than the value assigned to the body of the dam (Class "A") in all analyses; and thus, only the value for the body of the dam is referred to hereafter. A Poisson's ratio of 0.35 was used for the rockfill.

INITIAL STATIC STRESS ANALYSES

The initial static stress analyses were performed in steps to incrementally model the placement of the rockfill and the subsequent loads produced against the upstream face by the reservoir (elevation set to the value reported at the time of each earthquake). Parameter studies showed that the calculated mean confining stresses were not sensitive to the model parameters, which is reasonable given the homogenous cross-section of the dam. The results of the 2-D static stress analysis were then corrected to account for the effects of canyon shape using data presented by Lefebvre and Duncan (1971). For a triangular shaped canyon, Lefebvre and Duncan showed that a dam with a crest length (L) to height (H) ratio of 2:1, a 2-D analysis of the maximum cross-section can overestimate the static stresses in the lower third of the dam by

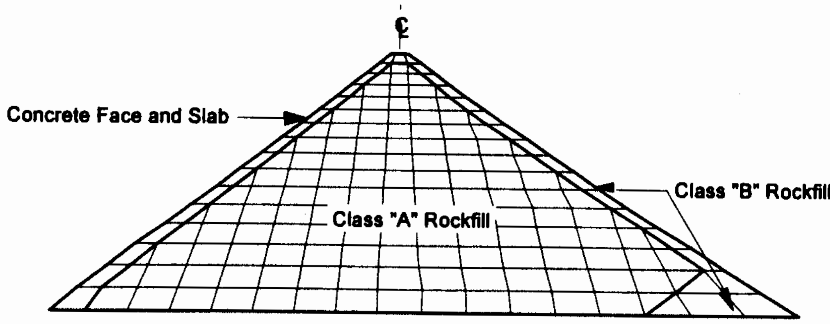


Fig. 8. Two-Dimensional FEM Mesh

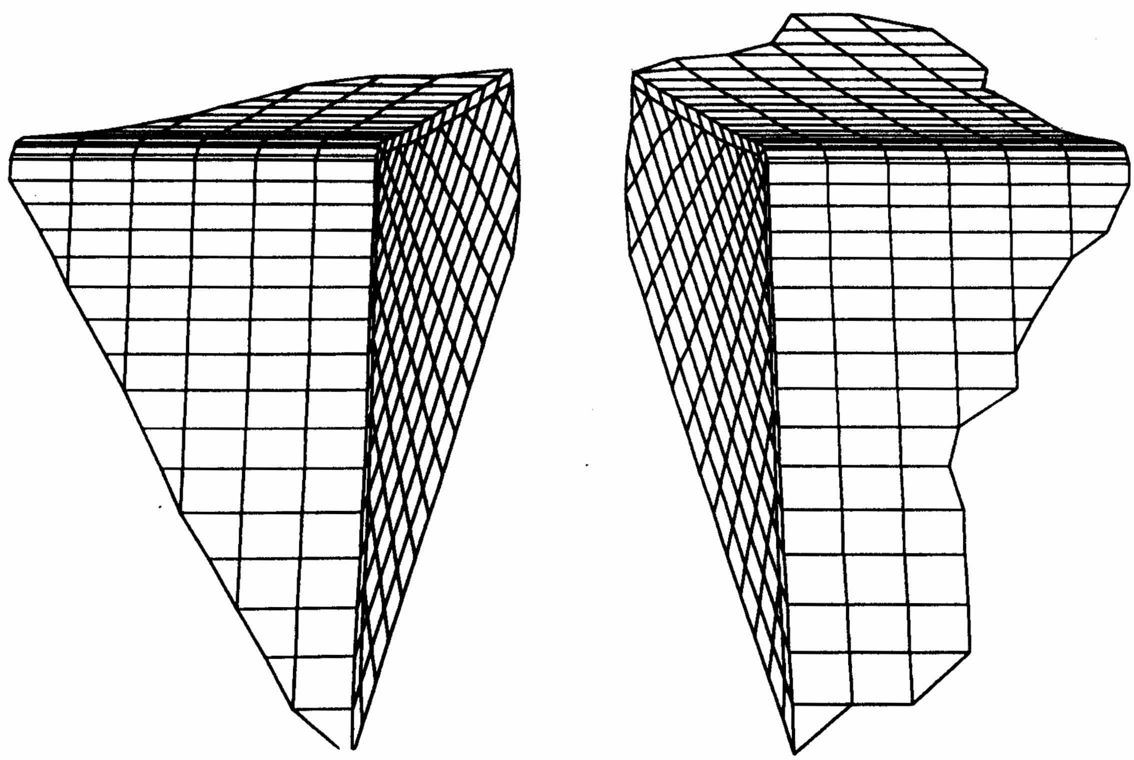


Fig. 9. Three-Dimensional FEM Mesh

as much as 40%. Consequently, the mean confining stresses obtained from the 2-D analyses were reduced by an amount that varied linearly with elevation from 0% at the lower 1/3-point of the dam to 30% at the base of the dam. Since the same maximum cross-sectional mesh geometry was used for all three FEM analyses, the calculated mean confining stresses corresponded directly to individual elements in the 2-D dynamic mesh and were projected longitudinally into the 3-D dynamic mesh.

TWO-DIMENSIONAL DYNAMIC RESPONSE ANALYSES

The 2-D dynamic response of the dam was computed using the recorded transverse abutment motions as the input motion to the rigid base. A maximum frequency of 12 Hz was used in the analyses. Analyses were performed with values for the soil model parameter K_{2max} of 60, 80, 100, 120, and 140. Additional sensitivity analyses included the use of an upper bound relationship for the damping ratio versus shear strain relationship. The computed response of the dam was compared to the recorded response in terms of acceleration response spectra, Fourier amplification ratios, and acceleration time histories at the dam crest. The results of selected analyses are summarized in Table 2. The response spectra and Fourier amplification ratios calculated for a K_{2max} value of 100 and both earthquake motions are presented in Figs. 10 through 13.

In general, the 2-D dynamic response analyses were not able to capture a couple of aspects of the recorded responses. The calculated Fourier amplification ratios exhibit their greatest value at their longest period, which corresponds to the first mode of vibration for the 2-D model and progressively smaller amplification ratios at higher modes of vibration. In contrast, the Fourier amplification ratios for the recorded motions exhibit a lower peak ratio at their first mode of vibration than is produced at the second mode of vibration. The difference in Fourier amplification ratios is reflected in the differences between the calculated and recorded acceleration response spectra. The calculated response spectra generally overpredict the recorded response spectra and do not accurately reproduce the same "shape" because the 2-D model is amplifying the input motions in a significantly different way than the recorded motions indicate was the case. The use of the upper bound damping relationship did improve the agreement between the calculated and recorded motions in terms of the magnitude of response but did not improve the general shape of the response spectra or Fourier amplification ratios.

THREE-DIMENSIONAL DYNAMIC RESPONSE ANALYSES

The 3-D dynamic response analyses are currently in progress and thus only preliminary results are currently available; each analysis requires about 1 day to run on a PC and 2 to 3 days on the VAX and thus the parameter studies are time consuming. The 3-D dynamic response of the dam is also being computed using the recorded transverse abutment motions as the input motion to the rigid base.

The 3-D dynamic response analyses appear to be able to capture some of the principal features of the recorded dam response better than was achieved with the 2-D response analyses. For example, the calculated Fourier amplification ratios exhibit the same trend of a lower amplification ratio at the first mode of vibration with a greater amplification ratio for the second mode.

The fundamental period of a dam calculated by 2-D response analyses is expected to differ significantly from values recorded for highly 3-D dam geometries. Mejia and Seed (1983) proposed a relationship between the fundamental frequency of a 3-D dam in a V-shaped or rectangular-shaped canyon versus an infinitely long dam with the same maximum cross-section and properties. For a dam with a L/H ratio of 2.1:1 in a V-shaped canyon, their relationship suggests that a 2-D response analysis using the true material properties should calculate a fundamental period which is about 1.65 times greater than the recorded

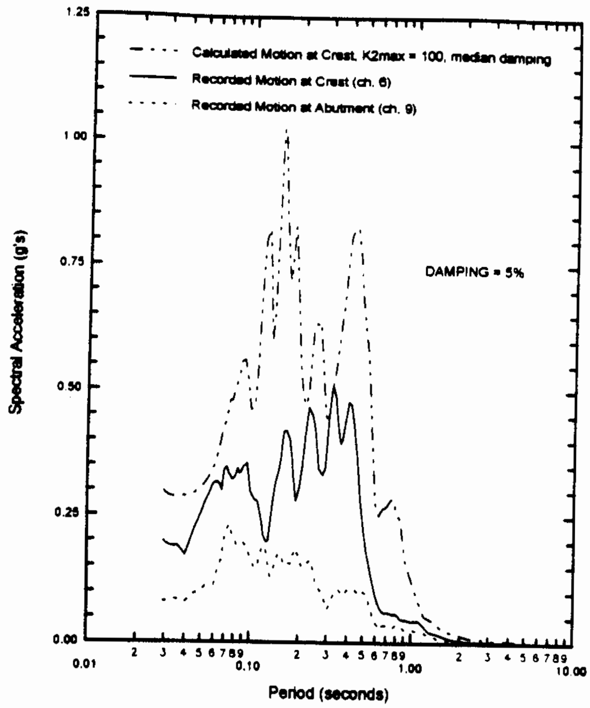


Fig. 10. Calculated 2-D Dynamic Response Spectra: Whittier Narrows Earthquake

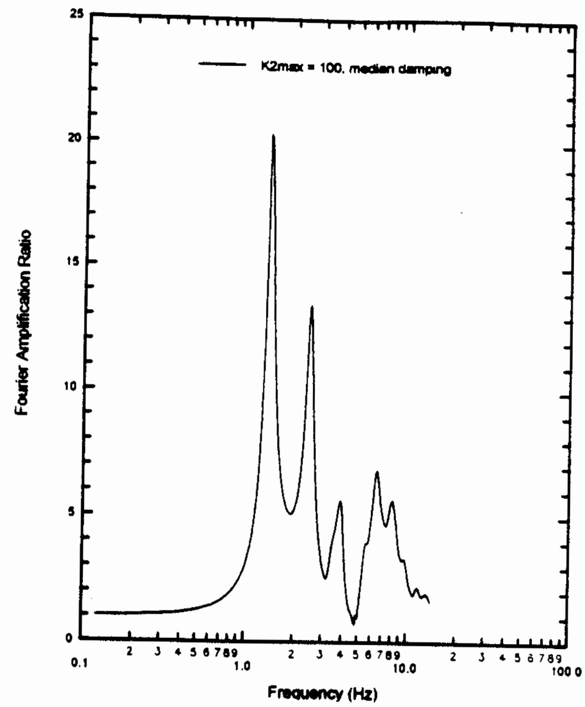


Fig. 11. Calculated 2-D Fourier Amplification Ratios: Whittier Narrows Earthquake

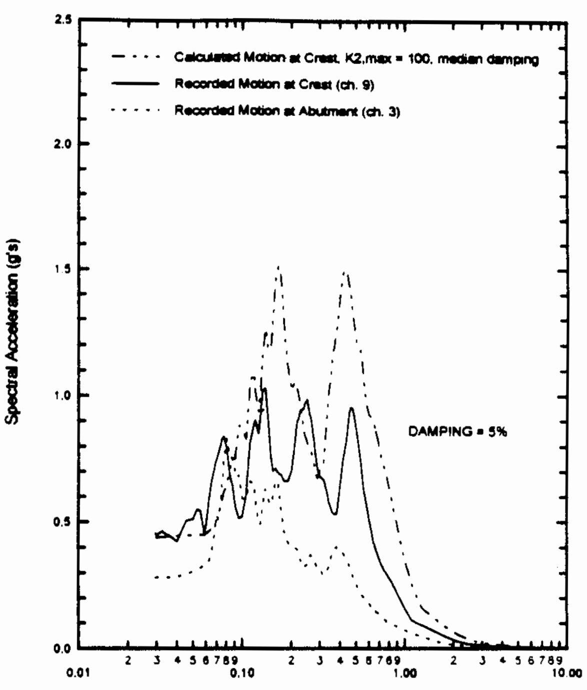


Fig. 12. Calculated 2-D Dynamic Response Spectra: Sierra Madre Earthquake

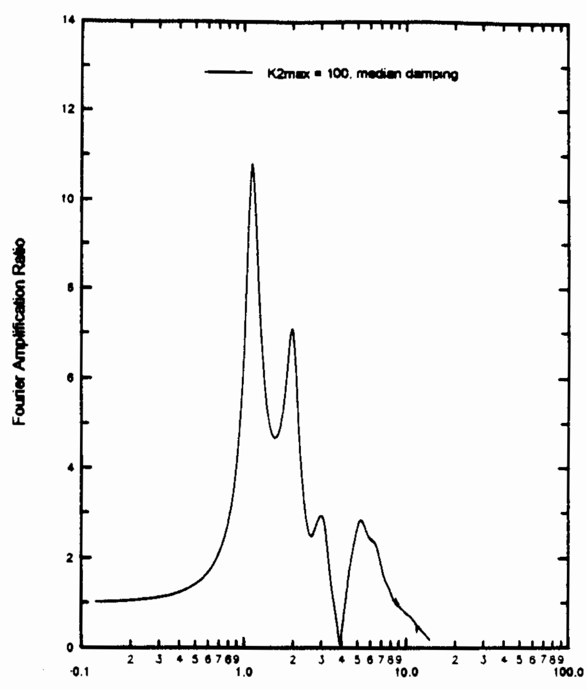


Fig. 13. Calculated 2-D Fourier Amplification Ratios: Sierra Madre Earthquake

TABLE 2(a). Results of 2-D Dynamic Response Analyses for Whittier Narrows Earthquake

$k_{2,max}$	Modulus Degradation Relationship	Damping Relationship	Average Effective Shear Strain (%) (range)	G/G_{max}	Fraction of Critical Damping (%)	Fundamental Period[(1)] (sec)	Max. Crest Accel. (g)	Response Spectra at Crest: Max. Spectral Accel. (g)	Response Spectra at Crest: Period at Max. Spectral Accel. (s)
60	Gravel	Median	0.013 (0.010-0.020)	0.51	6.71	0.98	0.20	0.75	0.19
80	Gravel	Median	0.012 (0.009-0.017)	0.52	6.47	0.84	0.21	0.88	0.18
100	Gravel	Median	0.010 (0.005-0.015)	0.56	5.85	0.73	0.27	1.03	0.15
100	Gravel	Upper-Bound	0.007 (0.004-0.011)	0.60	5.10	0.71	0.19	0.63	0.44
120	Gravel	Median	0.008 (0.005-0.012)	0.59	5.36	0.65	0.29	1.11	0.36
140	Gravel	Median	0.008 (0.006-0.011)	0.59	5.32	0.60	0.26	1.06	0.12
Recorded	N/A	N/A	N/A	N/A	N/A	0.37-0.42	0.15	0.51	0.32

TABLE 2(b). Results of 2-D Dynamic Response Analyses for Sierra Madre Earthquake

$k_{2,max}$	Modulus Degradation Relationship	Damping Relationship	Average Effective Shear Strain (%) (range)	G/G_{max}	Fraction of Critical Damping (%)	Fundamental Period[(1)] (sec)	Max. Crest Accel. (g)	Response Spectra at Crest: Max. Spectral Accel. (g)	Response Spectra at Crest: Period at Max. Spectral Accel. (s)
60	Gravel	Median	0.042 (0.030-0.058)	0.33	11.4	1.21	0.35	0.86	0.28
80	Gravel	Median	0.037 (0.026-0.054)	0.35	10.9	1.02	0.42	1.26	0.38
100	Gravel	Median	0.033 (0.024-0.052)	0.37	10.4	0.89	0.44	1.51	0.17
100	Gravel	Upper-Bound	0.027 (0.021-0.034)	0.40	9.5	0.87	0.36	1.18	0.42
120	Gravel	Median	0.032 (0.020-0.052)	0.37	10.2	0.80	0.56	1.96	0.42
140	Gravel	Median	0.033 (0.020-0.055)	0.37	10.3	0.73	0.68	2.30	0.42
Recorded	N/A	N/A	N/A	N/A	N/A	0.45-0.48	0.42	1.03	0.14

(1)Period for maximum amplification at dam crest from amplification function between crest and base of model.

fundamental period. For these preliminary 2-D and 3-D analyses of Cogswell dam, a comparison of the results for a K_{2max} value of 100 suggests that the ratio of the 2-D to 3-D calculated fundamental periods is about 2.0. The difference may be attributable to a greater stiffening effect produced by the somewhat asymmetric geometry of the canyon walls, and this possibility will be explore further as the 3-D analyses progress.

SEISMICALLY-INDUCED PERMANENT DEFORMATION ANALYSES

The presently available engineering methodologies for estimating seismically-induced permanent defomations in rockfill dams are not well refined and need to be viewed with considerable engineering judgement. It is interesting, however, to perfom a defomation analysis using the parameters adopted in previous engineering studies of the dam prior to the Sierra Madre Earthquake. As part of a geotechnical investigation of Cogswell Dam, the LACFCD (1980) estimated that the pseudostatic yield acceleration for potential slip surfaces through the dam is 0.21 g. For the Sierra Madre Earthquake, the maximum average acceleration (k_{max}) for potential sliding masses is estimated to be about 0.3 g based on the recorded motions and the relationships between peak crest accelerations and maximum average accelerations presented by Makdisi and Seed (1978). Thus, the ratio of k_y/k_{max} was 0.5 for which the Makdisi-Seed (1978) procedure for estimating dam and embankment earthquake-induced defomations predicts permanent defomations for a M = 6.5 earthquake to be less than 1.5 inches. Since the 1991 Sierra Madre event was only a $M_L = 5.8$ event, the predicted seismically-induced permanent defomations would be less than 1 inch. A Newmark (1965) double-integration of the recorded crest acceleration-time history scaled to a peak acceleration of 0.3 g was also performed. This approach predicted permanent deformations due to the Sierra Madre event to also be less than 1 inch. These estimates are in good agreement with the observed maximum defomations of 1.61-inch vertical and 0.63-inch horizontal at the dam crest, and with previous experiences regarding the perfomance of rockfill dams during earthquake loading.

SUMMARY

Cogswell Dam, a 280-foot high concrete-faced, loosely dumped rockfill dam, experienced a peak transverse crest acceleration of 0.42 g during the 1991 Sierra Madre Earthquake. The dam perfomed very well with a maximum deformation of about 1.6-inch at the crest and with relatively minor cracking in limited portions of the upstream concrete facing. The good perfomance of the dam is consistent with previous experiences and with simplified seismically-induced permanent defomation analyses.

Preliminary results of FEM dynamic response analyses of Cogswell dam during the Whittier Narrows and Sierra Madre earthquakes were presented. An initial assessment of the results indicate that the two-dimensional dynamic response analyses were limited in their ability to accurately model the observed dam response and, in general, tended to overpredicted the recorded dam response. Three-dimensional response analyses are presently being perfomed to take full advantage of this valuable set of strong motion records. Together, the two- and three-dimensional dynamic response analyses are expected to enable a more reliable estimation of the dynamic properties of the loosely dumped rockfill comprising the body of Cogswell dam.

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