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SYNOPSIS: The 1987 Whittier Narrows earthquake (M_L = 5.9) shook two dams, the Puddingstone and Cogswell dams, which were instrumented as part of the California Strong Motion Instrumentation Program (CSMIP). The resulting recorded accelerograms provided a valuable opportunity to investigate and evaluate the accuracy and reliability of conventional geotechnical procedures for evaluation of dynamic response characteristics of earth and rockfill dams. This paper presents the results of these studies, which provide insight regarding current techniques for dynamic soil property evaluation and the applicability of one- and two-dimensional analytical procedures to evaluation of the dynamic response of these types of dams.

THE PUDDINGSTONE DAM

The Puddingstone dam, located approximately 16 miles northeast of Whittier, California, actually consists of three earth dams. Figure 1 shows a schematic plan view of the main dam (Dam No. 1), and Figure 2 shows a cross section through the maximum height embankment section of the main dam. The main dam is a rolled earth fill embankment with a maximum height of 148 feet and a crest length of 1,085 feet. Two smaller saddle dams (Dams No. 2 and 3) with heights of 49.5 and 60 feet also serve to retain the reservoir. This study concerns only the main earth dam (Dam No. 1).

The Puddingstone dam was constructed during 1926 and 1927 of locally available crushed weathered shale. The resulting compacted material, which comprises the main portions of all three homogeneous earth embankment dams, is a sandy clayey silt with weathered shale fragments. Typically, the soil is composed of 60 to 90% fines of medium to high plasticity (MH-CH), with LL ≈ 55 to 70 and PI ≈ 26 to 32, and 10 to 40% sand and gravel sized particles. As shown in Figure 2, the toe of the main dam is drained with a triangular toe drain section composed of large boulders and gravel. The bedrock underlying the dam is primarily unweathered shale.

Seismic Instrumentation and the Recorded Motions

A total of 18 strong motion accelerographs were installed at six locations on and near the main dam, as shown in Figure 1. At most locations, motions were recorded in three orthogonal directions: vertically, parallel to the main dam axis and transverse to the main dam axis. This paper will concentrate on the transverse motions, as these are the motions of primary engineering interest. Sensors 1-6 and 13-18 were sited to record "bedrock" motions. These sensors were actually installed on shallow, stiff soil

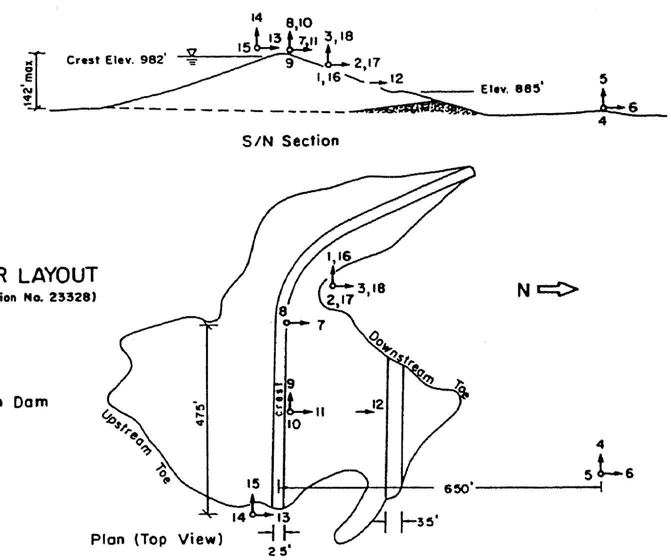

Fig. 1. Plan View of Puddingstone Main Dam Showing Sensor Locations

deposits or on protrusions of low-grade rock, so that they do not record true rock motions. They will be referred to as "near" rock sites. Sensors 1, 2, 3 and 16, 17, 18 were co-located, and produced nearly identical records for the 1987 Whittier Narrows earthquake. Sensors 7-12 were sited on the main dam's crest and downstream face.

The Whittier Narrows earthquake of October 1, 1987 provided an excellent record of the seismic response and performance of Puddingstone dam. This local magnitude 5.9 earthquake on the newly discovered Whittier Fault located approximately 16 miles from the dam site produced strong motions with peak ground accelerations of the "near" rock sites ranging from 0.04 g to 0.08 g. Puddingstone dam suffered no significant damage as a

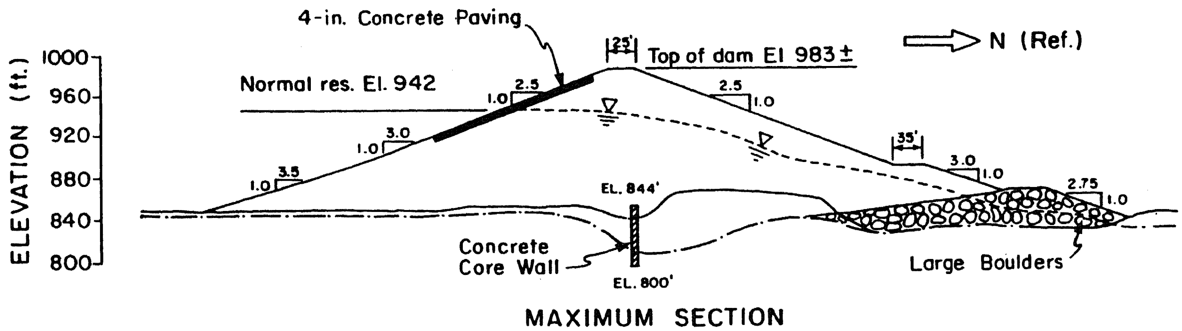

Fig. 2. Cross Section Through the Maximum Height Section of Puddingstone Dam

result of the earthquake shaking. Unfortunately, one of the sensors (Station 7) did not operate. Hence, the variation of strong motions in the transverse direction along the crest of the dam cannot be studied. On the other hand, Sensors 11 and 12 in conjunction with the recorded "near" rock motions provide an excellent opportunity to study the variation of strong motions transverse to the dam at the center of the crest and at the mid-height downstream slope of the dam at its maximum cross-section, and thus to study the dam's response characteristics of principal engineering interest.

Figure 3 shows the response spectra for the transverse components of the motions recorded at the three "near" rock sites (Channels 3, 6 and 13). All three motions are largely similar, and the peak accelerations recorded at all three stations were on the order of $a_{max} \approx 0.07$ g. A closer inspection, however, showed a higher concentration of energy at higher frequencies in the motion recorded on Channel 3, corresponding to the most "rock-like" recording among the three, so this motion was taken as the apparent rock motion for these studies. Subsequent comparative studies involving analyses using each of the three "near" rock transverse recordings confirmed that the best overall dam response could be achieved with the use of the Channel 3 recording as an input motion.

Figure 4 shows the response spectra for the transverse motions recorded on "near" rock, at the middle of the downstream face, and at the center of the crest. This figure clearly shows the response amplification as the dam was excited. The peak accelerations of these three recorded motions are 0.07 g, 0.18 g and 0.19 g, respectively.

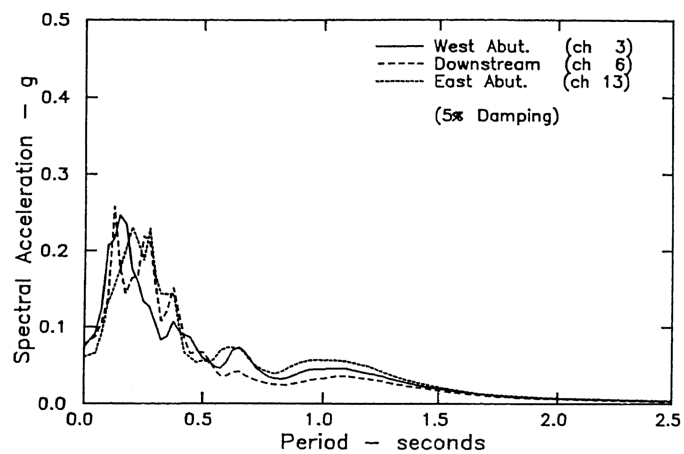

Fig. 3. Response Spectra for the Recorded Transverse Abutment Motions at Puddingstone Dam

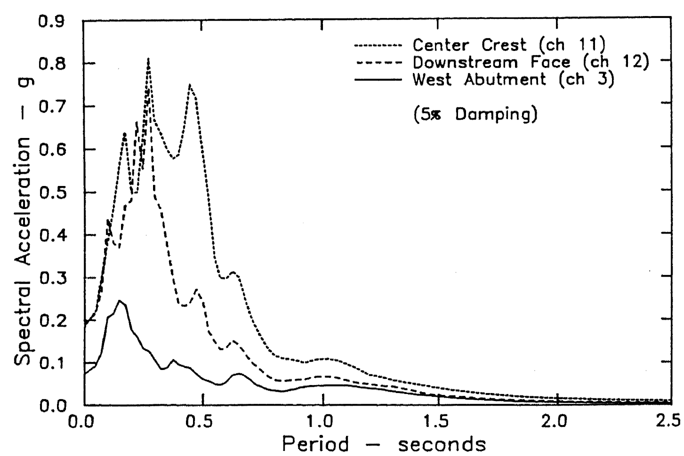

Fig. 4. Response Spectra for the Recorded Transverse Motions at Puddingstone Dam

Analyses of Dam Response

All analyses of dynamic response performed as part of these studies used the equivalent linear complex response method to model strain dependent moduli and damping characteristics. One-dimensional (columnar) response analyses were performed using the program SHAKE (Schnabel et al. 1972), and two-dimensional (plane strain) finite element analyses were performed using the program FLUSH (Lysmer et al. 1975).

Information regarding soil properties within the main Puddingstone embankment section were available from previous studies of the dam (IECO 1976). These data were used as a basis for evaluation of dynamic soil properties. Shear moduli at small strains (G_{max}) were evaluated using a variety of techniques (e.g. Hardin and

Drnevich 1970), and an average value of G_{max} = 4,000 kips/ft^2 was selected for use in all analyses presented herein. A full parameter study using a range of moduli is presented by Bray et al. (1990). Moduli varied slightly with confinement, and $G_{max} \approx 3{,}600$ kips/ft^2 was used at the crest and faces, and $G_{max} \approx 4{,}400$ lb/ft^2 within the interior of the embankment. A shear modulus vs. shear strain relationship for the sandy silty clay comprising the main embankment section was selected, based on recent studies by Sun et al. (1988), and is presented in Table 1. A review of the damping vs. shear strain data presented by Sun et al. (1988) and Seed et al. (1984) suggested that sandy clayey silts typically have somewhat higher than average damping ratios relative to the relationships they suggested for cohesive soils, so the upper bound damping ratios proposed by Sun et al. (1988) for cohesive soils were used, as shown in Table 1. This damping curve is intermediate between the average curves recommended for cohesive soils and sandy soils in these two references. Dynamic properties of the cohesionless toe drain were relatively unimportant, and were modeled using modulus degradation and damping vs. shear strain relationships recommended for gravelly soils by Seed et al. (1984), with $(K_2)_{max}$ = 90. The abutment rock shear wave velocity was modeled as V_s = 5,000 ft/sec.

Table 1. Dynamic Shear Moduli and Damping Ratios vs. Shear Strain Used for Puddingstone Analyses

Shear Strain (γ)	Normalized Shear Modulus ($\frac{G}{G_{max}}$)	Damping Ratio
10^{-4}%	1.00	2%
10^{-3}%	0.99	4%
10^{-2}%	0.86	7.5%
10^{-1}%	0.40	15%

Figure 5 shows the finite element mesh used for 2-D finite element (FEM) analyses of Puddingstone dam. Comparative analyses showed that a frequency cut-off above 12 Hz provided a negligible loss of accuracy in performing these analyses. Figure 6 shows a comparison between the response spectra for the resulting predicted crest and mid-downstream face motions vs. those actually recorded. The predicted peak acceleration of a_{max} = 0.21 g at the crest agrees well with the recorded peak of a_{max} = 0.19 g, and the predicted crest response spectra is in good general agreement with the observed crest motions. The predicted peak acceleration of a_{max} = 0.15 g at the downstream face station also agrees well with the recorded

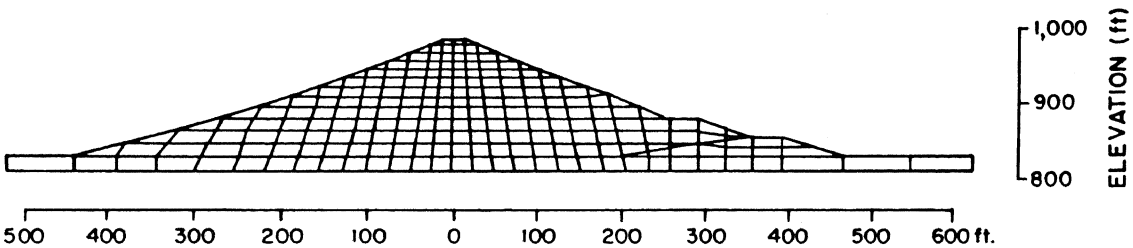

Fig. 5. Finite Element Mesh Used to Model Puddingstone Dam

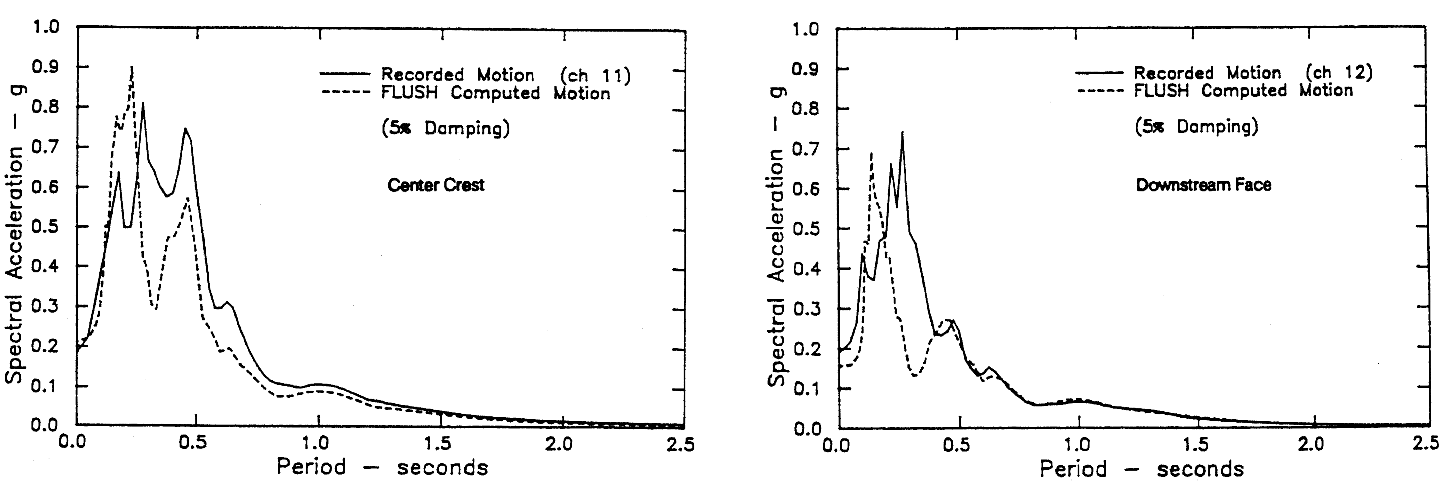

Fig. 6. Comparison Between Predicted and Observed Response Spectra: 2-D FEM Analyses of Puddingstone Dam

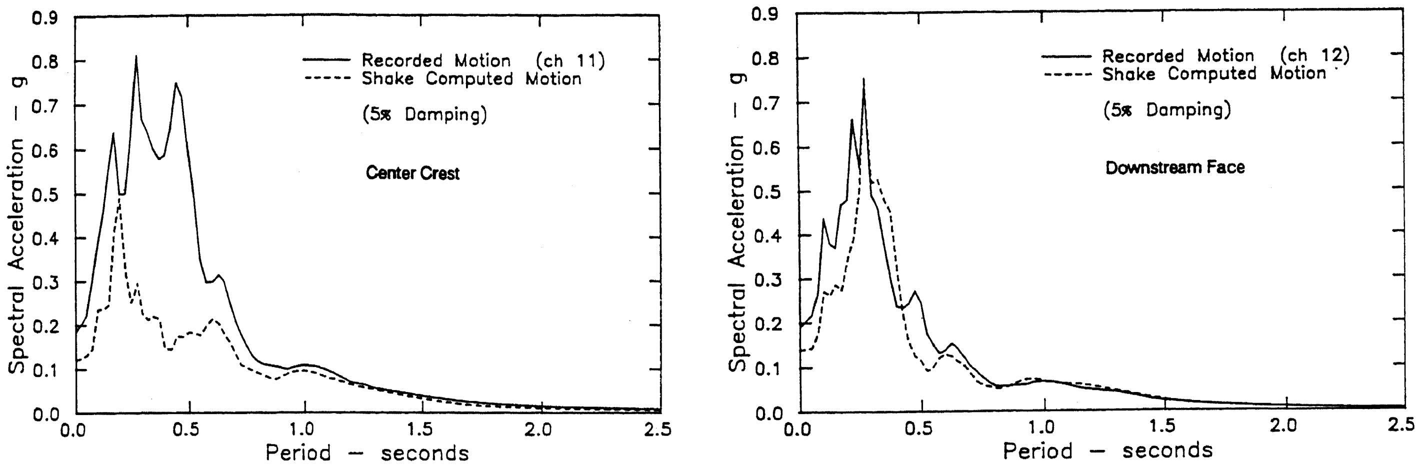

Fig. 7. Comparison Between Predicted and Observed Response Spectra: 1-D Columnar Analyses of Puddingstone Dam

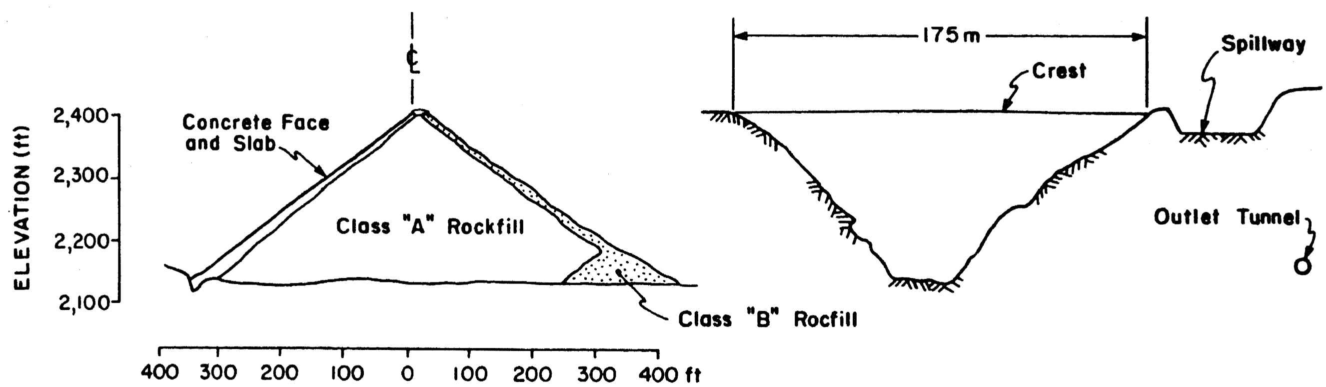

Fig. 8. Cross Section Views of the Maximum Height Sections of Cogswell Dam

a_{max} = 0.18 g, and the spectral response agreement is fairly good here too. Overall, these 2-D FEM analyses provided a good prediction of the observed response, with an accuracy level amply sufficient as to provide a good basis for engineering analyses.

Similar analyses were performed using 1-D analyses of "representative" columnar sections through the crest and downstream face, and the results are shown in Figure 7. As expected, these analyses greatly under-predicted both the peak acceleration and the spectral response at the crest station, but they provided a somewhat better (but still only fair) prediction of the observed response at the downstream face station.

COGSWELL DAM

Cogswell dam, located 20 miles north of Whitter, was designed as a conventional concrete-faced rockfill dam. At the time of its construction (1931-34) the conventional way of placing the rockfill in such a dam was by dumping and sluicing the rock with large volumes of water. Because of the scarcity of water at the Cogswell site, the sluicing part of the usual procedure was omitted and the rockfill was dumped in 25 ft lifts with no compaction, leading to a very loose condition of the fill.

The quarried rockfill was predominantly granitic, with the body of the dam (Class A in Figure 8) consisting of a well graded mixture specified as follows: 40 percent by weight from quarry chips to 1,000 lbs; 30 percent from 1,000 lbs to 3,000 lbs; 30 percent from 3,000 lbs to 14,000 lbs; and with no more than 3 percent quarry dust. Class B rockfill (downstream face and toe) was extra larger rocks, and Class C (upstream face) was derrick-placed to maximum density. The foundation bedrock is predominantly light-colored (augen) gneiss intruded by numerous dikes of andesite porphyry and hornblende amphibolite. Within the dam foundation, these rocks generally are closely to moderately jointed, frequently sheared, moderately to strongly weathered, moderately hard, and moderately strong.

The entire rockfill section, some 280 ft high with average side slopes of 1.3:1 (H:V) upstream and 1.3:1 to 1.6:1 downstream was placed by dry dumping between Spring 1931 and Fall of 1933. At this stage construction began on placement of the concrete-facing with the intention of completing this work by 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 dam deformations. During one particularly severe rainstorm of December 31, 1933 the crest of the dam settled about 5 ft, and throughout the four months of rain the total settlement of the crest was as much as 15 ft. 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.

Figure 8 shows a transverse cross section through the maximum height section of the completed dam as it stands today, and a longitudinal profile along the crest showing the geometry of the steep-walled, V-shaped canyon.

Seismic Instrumentation and the Recorded Motions

A total of 9 strong motion accelerographs were installed at three locations on and near the dam, as shown in Figure 9. At each location, motions were recorded in three orthogonal directions: vertical, parallel to the main dam axis, and transverse to the main dam axis. Again, this paper will concentrate on the transverse motions, as these are the motions of primary engineering interest. The recorded right abutment transverse motions were used as the input "rock" motions for the analyses described herein.

Figure 10 shows the response spectra for the transverse motions recorded at the right abutment, the right crest, and the center crest with peak recorded accelerations of a_{max} = 0.06 g, 0.10 g and 0.16 g, respectively.

The response recordings obtained at Cosgwell dam provided a valuable opportunity to obtain field data regarding the dynamic response characteristics of rockfill materials. Only limited data regarding their behavior exists, as they cannot be adequately evaluated using laboratory testing techniques, and field shear wave velocity measurements in coarse rockfills are fraught with difficulty.

Analysis of Dam Response

Modeling the dynamic response of Cogswell dam is complicated by the small length to height ratio of the dam (L/H = 2.1:1). This steep-faced dam in such a narrow V-shaped canyon was not expected to be amenable to the straightforward application of 2-D FEM analysis techniques, and this proved to be the case. Nonetheless, the results of 2-D FEM analyses (using the program FLUSH) combined with appropriate allowances for 3-D effects sufficed to provide valuable information on the properties of the rockfill comprising this embankment.

The dynamic shear modulus degradation (relationship G/G_{max} vs. shear strain relationship) and the damping ratio vs. shear strain relationship used to model the rockfill were the modulus degradation curve and upper bound damping curve recommended by Seed et al. (1984) for gravelly soils. It then remained to select a value for the parameter $K_{2,max}$ which would then establish G_{max} (psf) as

$$G_{max} = 1{,}000 \cdot K_{2,max} (\sigma_m')^{1/2} \quad (1)$$

where σ_m' (psf) is the mean effective confining stress, as determined by static FEM analyses using the program SSCOMPPC (Boulanger et al. 1991).

The value of $K_{2,max}$ for the upstream facing (Class C), downstream facing (Class B), and downstream toe (Class B) was taken as 1/3 greater than the $K_{2,max}$ value used for the main body of the dam (Class A). Different analyses are thus identified only by the value of $K_{2,max}$ assigned to the main body of the dam.

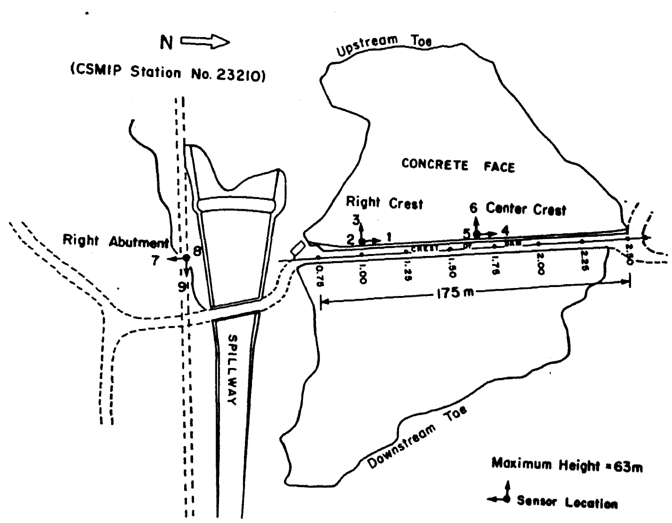

Fig. 9. Plan View of Cogswell Dam Showing Sensor Locations

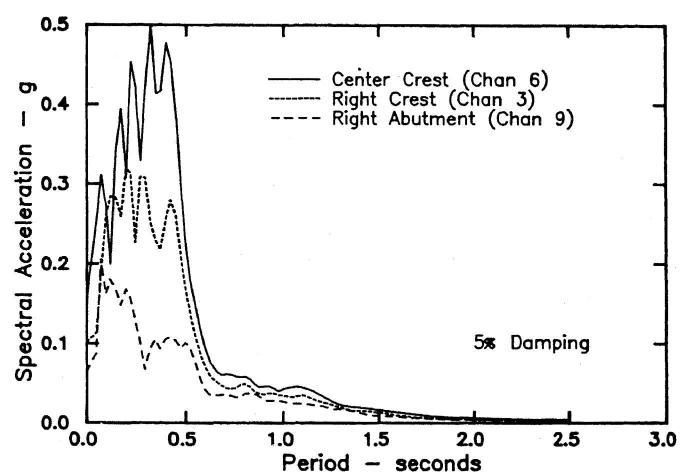

Fig. 10. Response Spectra for the Transverse Motions Recorded at Cogswell Dam

Figure 11 shows the finite element mesh used for 2-D FEM analyses of Cogswell dam. Comparative studies showed that a frequency cutoff of 10 Hz was sufficiently accurate for these studies. Dynamic analyses were performed using $K_{2,max}$ values of 80 to 240 (the fully likely range). In all cases, the effective shear strain (taken as 65% of the maximum shear strain) induced in the embankment ranged from 4.0 x 10^{-3} percent to 1.1 x 10^{-2} percent. Figure 12 shows response spectra for crest motions calculated using $K_{2,max}$ = 120, 180, and 240. As these example spectra illustrate, the 2-D analyses significantly over-predicted the actual observed 3-D response (see Figure 10 for comparison). One possible reason for the differences between computed and recorded responses may be that the input motions recorded on the right abutment are not representative of the bedrock motions which occurred near the base of the dam. For example, Lai and Seed (1985) found that significant amplifications may be present in motions recorded at rock outcrops above the crest of the dam. A second possible reason is inherent difficulties with 2-D modeling of such a highly 3-D dam. Despite these limitations, the best overall match with respect to shape and frequency content of the calculated and recorded responses was obtained using $K_{2,max}$ values of 150 to 180 for the body of the dam. These high $K_{2,max}$ values apparently compensate for the "stiffening" effect of the V-shaped canyon on the true dam response.

Further analyses to evaluate the rockfill response characteristics concentrated on the predominant period of the observed response. Mejia and Seed (1981, 1984) proposed a relationship between the predominant frequency of a fully 3-D dam in a V-shaped canyon vs. an infinitely long dam with the full maximum crest section (based on 2-D, plane strain analysis) as a function of dam height (H) over crest length (L). Their relationship was based on 2-D and 3-D back-analyses of the response of several such dams, and was supported by similar theoretical analyses by Ambraseys (1960). For Cogswell dam, with L/H = 2.1:1, the plane section 2-D period would be approximately 1.65 times the actual 3-D period.

Two approaches were taken to evaluate the observed 3-D period. The recorded crest response motion had a peak spectral acceleration of 0.51 g at a period of 0.33 seconds, as shown in Figure 10. Because of the broad band spectral crest response with its multiple peaks, however, there was some question as to whether this represented interaction with the high frequency input motions, in which case the slightly lower peak spectral acceleration of 0.48 g at a period of 0.40 seconds might better represent the dam's predominant period at the observed strain levels. Accordingly, sections of the crest response accelerogram representing periods of strong shaking and initial decay of strong shaking were analyzed, and found to indicate predominant periods of 0.35 to 0.37 seconds, as shown in Figure 13. As a second approach, the Fourier amplification ratio between the center crest and abutment transverse motions was calculated, and it exhibited a strong peak in the amplification ratio at a period of about 0.36 seconds. The dam's predominant period, T_p, was thus taken to be between about 0.35 and 0.40 seconds. By scaling for 3-D geometry effects, the corresponding maximum plane section (2-D) predominant period would then be $T_{p(2\text{-}D)} \approx$ 0.58 to 0.66 seconds.

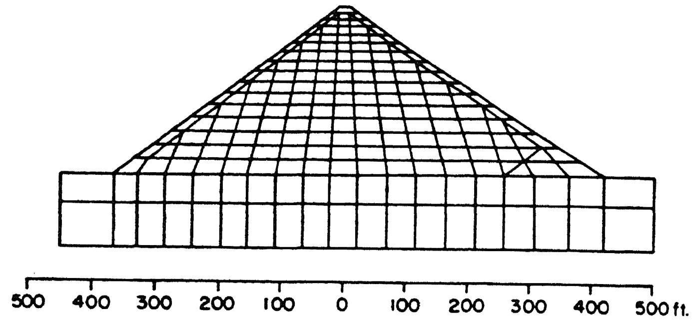

Fig. 11. Finite Element Mesh Used to Model Cogswell Dam

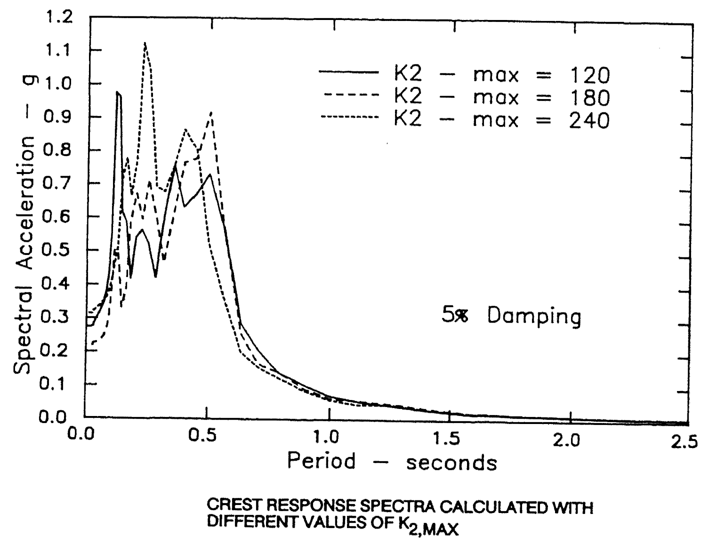

Fig. 12. Crest Response Spectra Calculated with Different Values of $K_{2,max}$

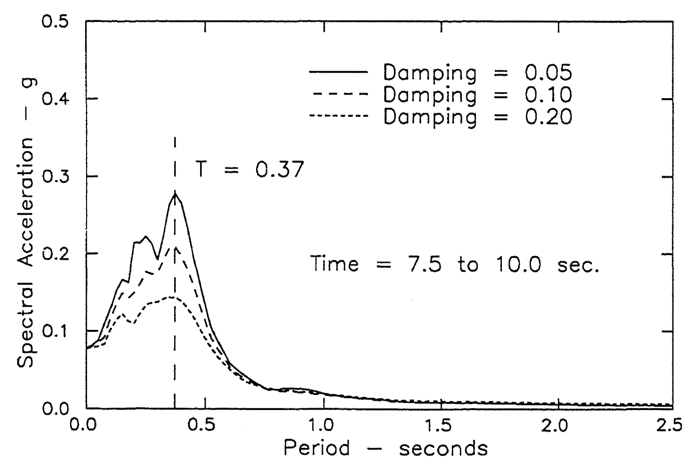

Fig. 13. Crest Transverse Response Spectra from t = 7.5 sec to 10.0 sec

The calculated predominant periods for the 2-D planar FEM model using different $K_{2,max}$ values were:

$K_{2,max}$ in body of dam	Predominant Period (sec)
80	0.745
100	0.650
120	0.592
150	0.536
180	0.480
240	0.410

The range of $K_{2,max}$ values producing the desired range in the calculated 2-D predominant period was $K_{2,max} \approx 100$ to 125.

The results of the preceding analyses agree well with the simple relationship proposed by Ambraseys and Sarma (1967) for estimating the predominant period of 2-D planar dam sections as

$$T_p \approx 2.61 \times H/V_s \qquad (2)$$

where V_s is the average shear wave velocity (based on G_{avg}, the average shear modulus) within the embankment, and H is the embankment height. For the levels of shear strain likely to have been induced within the Cogswell embankment by the Whittier Narrows earthquake, the representative G_{avg} is likely to have been about 55% to 60% of G_{max}, so that $K_{2,max}$-values of about 80 to 100 would result in an estimated T_p of 0.58 to 0.66 seconds, in fairly good agreement with the results of the 2-D dynamic analyses.

Overall, it was concluded that best modeling of the recorded response of the Cogswell dam rockfill embankment was achieved with $K_{2,max} \approx 150$ to 180 for 2-D analyses, while the true (3-D) in situ properties were better represented by a $K_{2,max}$ value of about 100 to 125. This compares well with values of $K_{2,max}$ recommended for 2-D and 3-D analyses of similar rockfill materials by previous investigators (Romo et al. 1980; Lai 1985; Mejia et al. 1991) provided an appropriate allowance is made for the difference in shear modulus degradation relationships used and the level of shear strains induced. For example, Figure 14 illustrates how significantly lower $K_{2,max}$ values would have been obtained in this study if a modulus degradation relationship for sands were used (as was used in some of the earlier cited investigations).

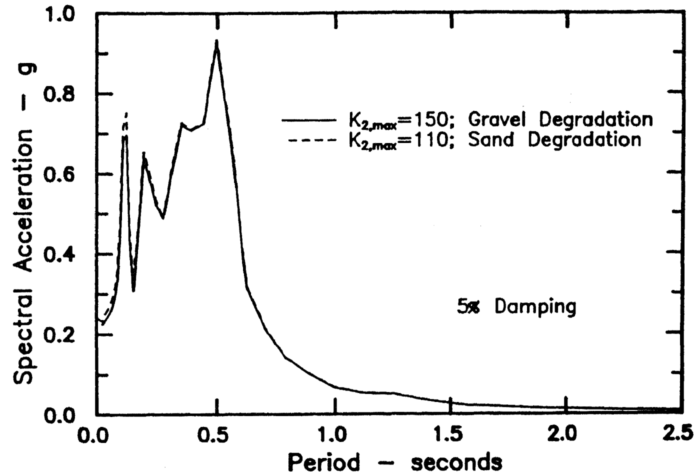

Fig. 14. Response Spectra for Equivalent Combinations of $K_{2,max}$ Values and Modulus Degradation Curves

SUMMARY AND CONCLUSIONS

Good agreement between the observed response characteristics of Puddingstone dam and the response characteristics predicted using both simple empirical methods as well as 2-D finite element analyses, based on established methods for evaluation and modeling of strain-dependent dynamic shear moduli and damping, provides good support for these modeling and analytical techniques. Three-dimensional effects were only moderate for this dam in a V-shaped canyon with a crest length vs. dam height ratio of $L/H \approx 4.5:1$, and the 2-D finite element analyses provided response predictions for both the crest and downstream face motions which were in sufficient agreement with observed response as to provide a good basis for engineering analyses. Even the simpler 1-D analyses provided good approximate predictions of peak accelerations for the downstream face, though these simpler analyses were unable to provide a reasonable prediction of the observed crest response.

The Cogswell dam response recordings provided a rare opportunity to obtain additional insight into the parameters suitable for modeling strain-dependent dynamic moduli for rockfills. Only very limited previous data from several similar back-analyses of observed rockfill dam response exist. Analyses of the observed dam response characteristics resulted in selection of a value of $K_{2,max} \approx$ 100 to 125 as best modeling the behavior of this loosely-dumped and then sluiced rockfill, in good general agreement with the values of $K_{2,max}$ recommended by previous investigators (Romo et al. 1980; Lai 1985; Mejia et al. 1991), based on similar back-analyses of the response of rockfill dams of similar composition. Although a good estimate of $K_{2,max}$ could be obtained from the response recordings, the geometry of Cogswell dam (a steep-faced dam in a steep-walled, V-shaped canyon with a crest length vs. dam height ratio of $L/H \approx 2.1:1$) was shown not to be amenable to reliable 2-D analysis. This finding was in agreement with previous studies which suggest that dam response is increasingly affected by abutment constraints for dams in V-shaped canyons with $L/H < 3:1$, and that 3-D analyses are required for accurate and comprehensive prediction of the strong motion response characteristics of such dams.

ACKNOWLEDGEMENTS

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