SEISMIC RESPONSE OF THE PUDDINGSTONE AND COGSWELL DAMS IN THE 1987 WHITTIER NARROWS EARTHQUAKE

by

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ABSTRACT

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-, two- and three-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 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 silty clay with weathered shale fragments. Typically, the soil is composed of 60 to 90% fines of medium to high plasticity (CH-MH), 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.

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, longitudinally (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 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 Whittier Narrows Earthquake. Sensors 7-12 were sited on the main dam's crest and downstream face.

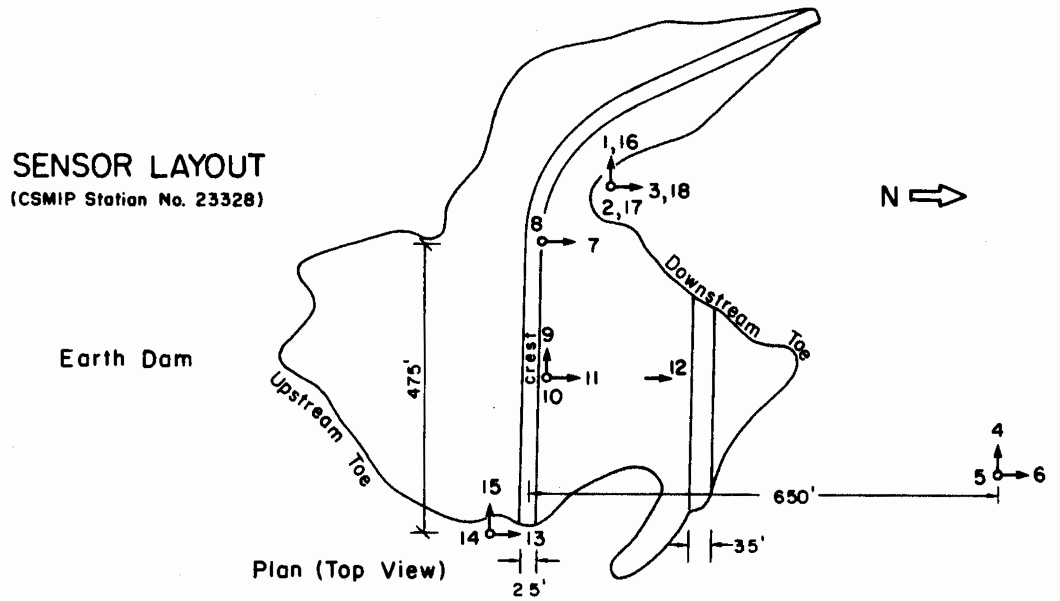

Fig. 1: Plan View of Puddingstone Main Dam Showing Sensor Locations

The Whittier Narrows Earthquake of October 1, 1987 provided an excellent record of the seismic response and performance of Puddingstone Dam. This 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 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

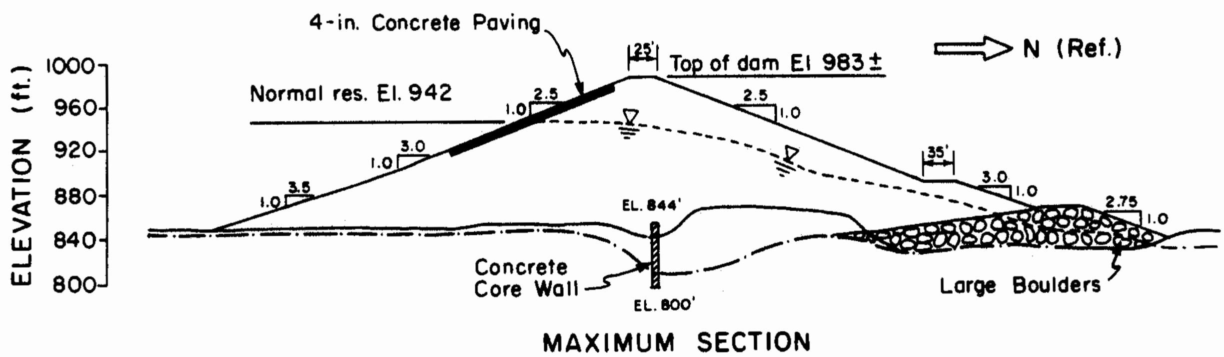

Fig. 2: Cross-Section Through the Maximum Height Section of Puddingstone Dam

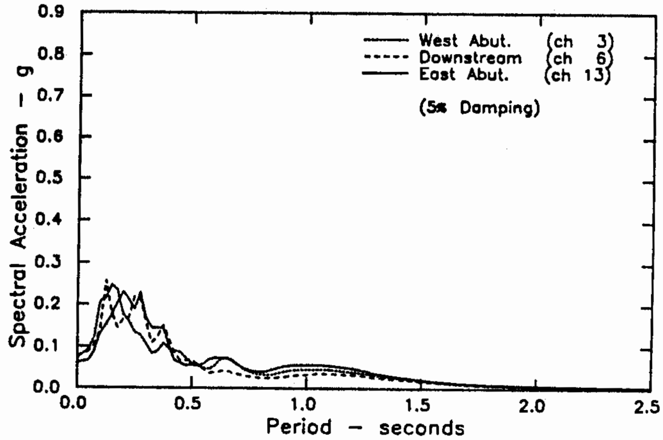

Fig. 3: Response Spectra for the Recorded Transverse Motions at the East and West Abutment, and Downstream of Puddingstone Dam

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 recorded transverse motions (a) "near" rock [Channel 3], (b) at the middle of the downstream face [Channel 12], and (c) at the center of the crest [Channel 11], clearly demonstrating 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.

Analyses of Dam Response:

All analyses of dynamic response performed as part of these studies used the equivalent linear 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 a 1973 study of the dam. These data were used as a basis for evaluation of dynamic shear moduli (G). Shear moduli at small strains (G_{max}) were

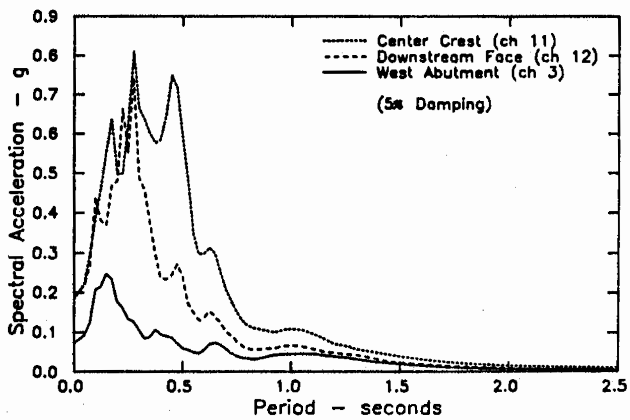

Fig. 4: Response Spectra for the Recorded Transverse Motions at the Crest, Mid-Face (Downstream) and Abutment of Puddingstone Dam

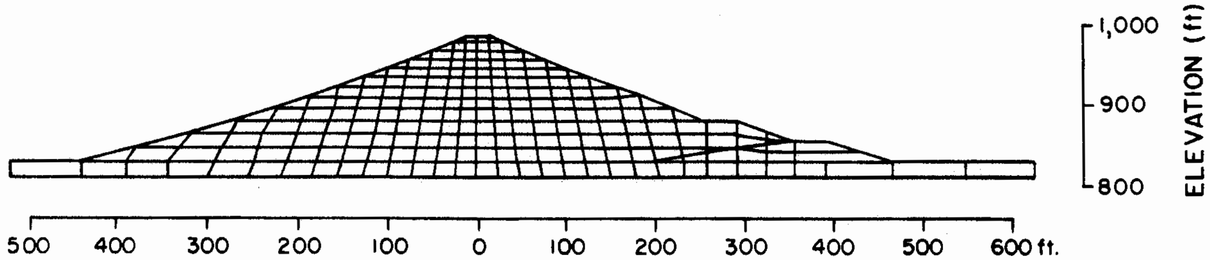

Fig. 5: Finite Element Mesh Used to Model Puddingstone Dam

evaluated using a variety of techniques (Bray et al., 1989), 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. (1989). 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 deep 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. (1989) and Seed et al. (1984) suggested that sandy silty clays 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 recommend for cohesive soils and sandy soils in these two references. Dynamic properties of the cohesionless toe drain were relatively unimportant, and were modelled 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 modelled as v_s = 5000 ft/sec.

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 a_{max} = 0.18 g, and the spectral response

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

Shear Strain (γ)	Normalized Dynamic Modulus $\left(\frac{G}{G_{max}}\right)$	Damping Ratio
10^{-4}%	1.00	2%
10^{-3}%	0.99	4%
10^{-2}%	0.86	7.5%
10^{-1}%	0.40	15.0%

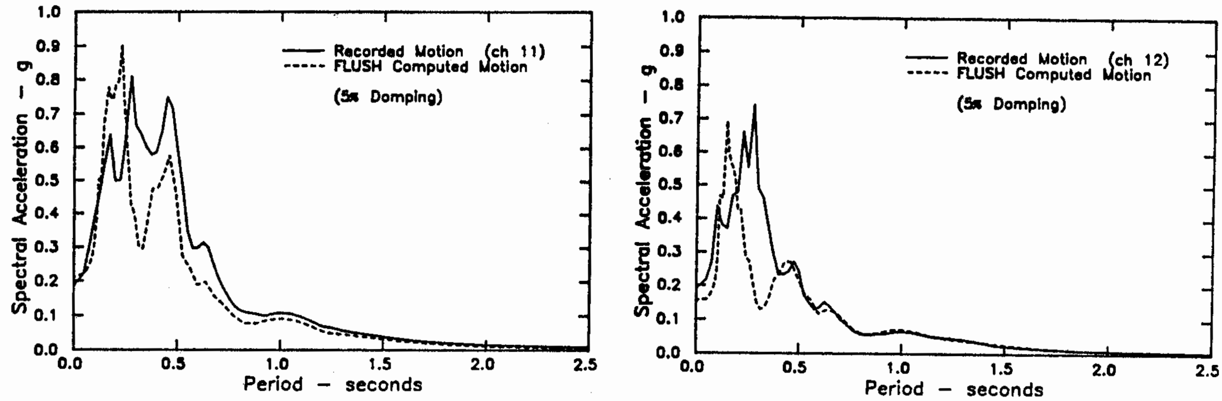

Fig. 6: Comparison Between Predicted and Observed Response Spectra: 2-D FEM Analyses of Puddingstone Dam

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 Whittier, was designed as a conventional Concrete-Faced Rockfill Dam of the old fashioned type. 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 entire rockfill section, some 280 ft high with average side slopes of 1.3:1 upstream and 1.3:1 to 1.6:1 downstream was placed in this manner between Spring 1931

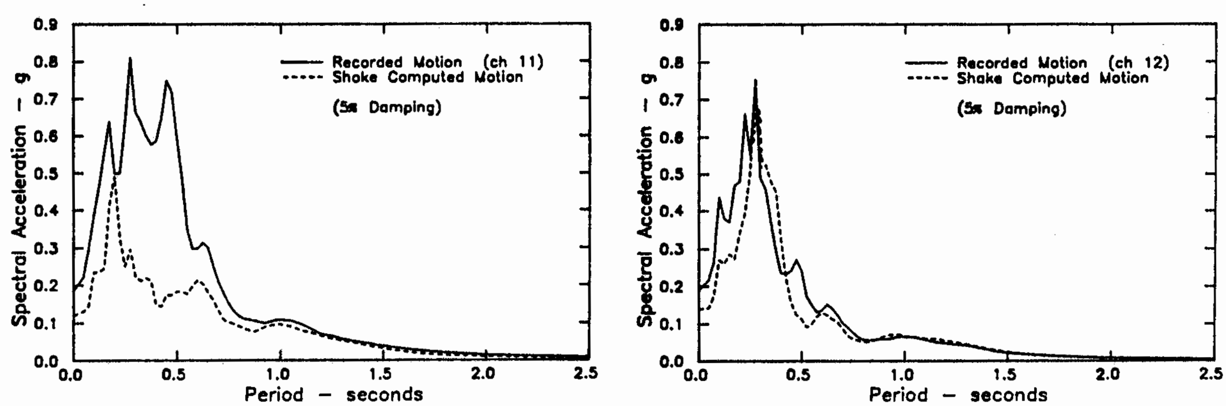

Fig. 7: Comparison Between Predicted and Observed Response Spectra: 1-D Columnar Analyses of Puddingstone Dam

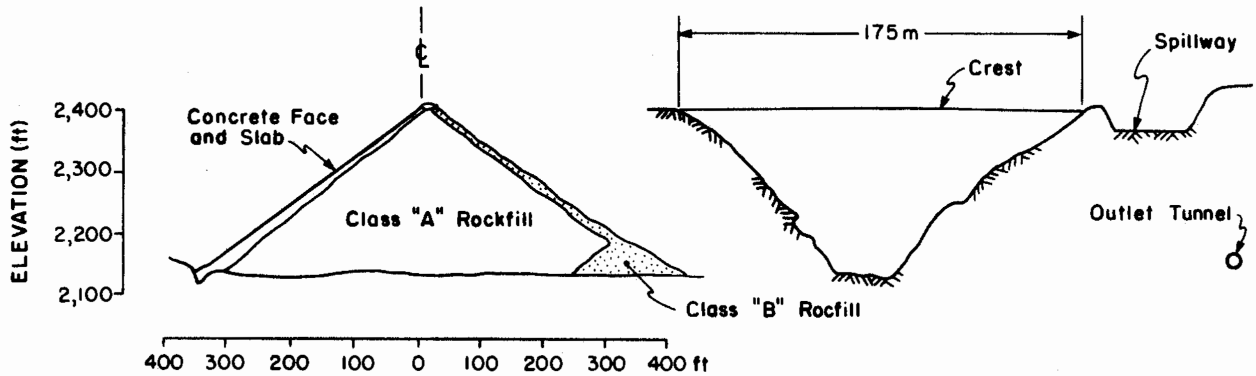

Fig. 8: Cross-Section Views of the Maximum Height Sections of Cogswell Dam

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 interfered with this plan. The heavy rains also wetted the fill and led to large settlements which disrupted the facing already constructed and caused deformations of the dam. 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 re-shape the dam and reconstruct the upstream facing. It was decided to use a temporary timber facing until settlements had essentially ceased, and then to replace the timber with a gunite facing. In the event, the timber was left in place for about 10 years before it was replaced by the gunite facing.

Figure 8 shows a transverse cross-section through the maximum height section of the completed dam as it stands today, and a longitudinal cross-section 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: vertically, longitudinally (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 right abutment sensors were well sited to record abutment "rock" motions, and the transverse motions recorded at this station were used as input motions for the analyses described herein.

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

Analyses of Dam Response:

As this steep-faced dam in a narrow, V-shaped canyon has a low crest length vs. maximum crest height ratio of only $L/H \approx 2.1{:}1$, this dam was judged likely to respond in a highly complex three-dimensional fashion not amenable to analysis by 2-D techniques, and this proved to be the case. Nonetheless, relatively simple analyses sufficed to provide valuable information regarding the dynamic response properties of the rockfill comprising this embankment.

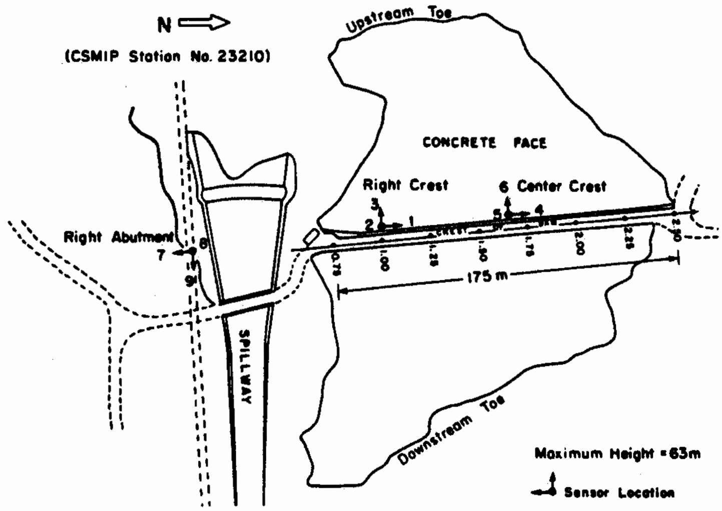

Fig. 9: Plan View of Cogswell Dam Showing Sensor Locations

The response recordings obtained at Cogswell 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.

Since the geometry of Cogswell Dam would require fully three-dimensional analyses to accurately reproduce the full observed response, 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 H/L = 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 predominant period of 0.33 seconds, as shown in Figure 10. Because of the broad band spectral crest response with its multiple peaks, however, there

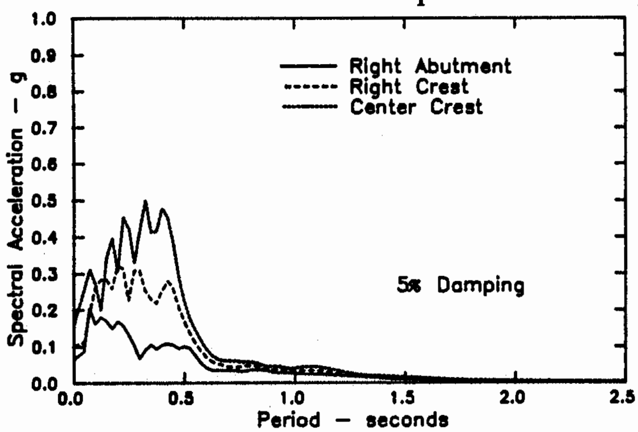

Fig. 10: Response Spectra for the Transverse Motions Recorded at Cogswell Dam

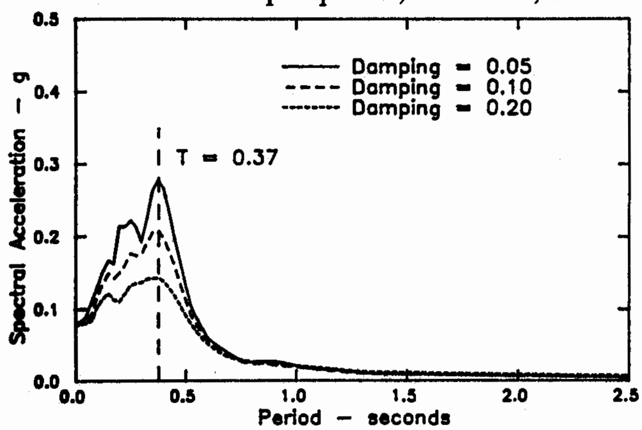

Fig. 11: Crest Transverse Response Spectra from t = 7.5 sec to 10.0 sec.

was some question as to whether this represented interaction with the high frequency input motions, in which case a slightly lower peak on the spectral response might better represent the dam's predominant period at the observed strain levels. Accordingly, a section of the crest response accelerogram representing the initial period of decay of strong shaking was analyzed, and found to have a predominant period of approximately 0.37 seconds, as shown in Figure 11. The dam's predominant period was thus taken to be $T_p \approx 0.33$ to 0.37 seconds. By scaling for 3-D geometry effects, the corresponding maximum plane section (2-D) predominant period would then be T_p (2-D) $\approx$ 0.54 to 0.61 seconds.

The dynamic shear modulus degradation curve (the 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. Having thus selected a dimensionless modulus degredation curve, it then remained to give it scale by selection of a value for the paramter $K_{2,max}$ which would then establish G_{max} as

$$G_{max} = 1000\, K_{2,max} (\sigma_m')^{\frac{1}{2}} \qquad \text{(Eq. 1)}$$

Three approaches were taken to evaluate $K_{2,max}$ based on the Cogswell Dam response recordings. Ambraseys and Sarma (1967) developed a relationship for estimating the predominant period of 2-D planar dam sections as

$$T_p \approx 2.61 \times H/V_s \qquad \text{(Eq. 2)}$$

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% of G_{max}, so that the $K_{2,max}$-values necessary to produce a T_p of about 0.54 to 0.61 seconds are $K_{2,max} \approx 110$ to 130.

A second, more robust estimation of $K_{2,max}$ was achieved by performing 2-D, plane strain response analyses of Cogswell Dam using the mesh shown in Figure 12. Using the abutment recording as an input motion, and varying $K_{2,max}$ over a range from $K_{2,max}$ = 95 to 240 (the full likely range) produced varying predominant periods, as shown in Figure 13. The values producing the desired 2-D predominant periods of $T_p \approx 0.54$ to 0.61 seconds were $K_{2,max} \approx 120$ to 150, in good agreement with the values estimated above.

Figure 14 shows response spectra for crest motions calculated using $K_{2,max}$ = 120, 180 and 240. Similar spectra were produced for other choices of $K_{2,max}$. As shown in Figure 14, all 2-D analyses over-predicted the actually observed 3-D response (see Figure 10 for comparison). A careful evaluation of these calculated crest response spectra,

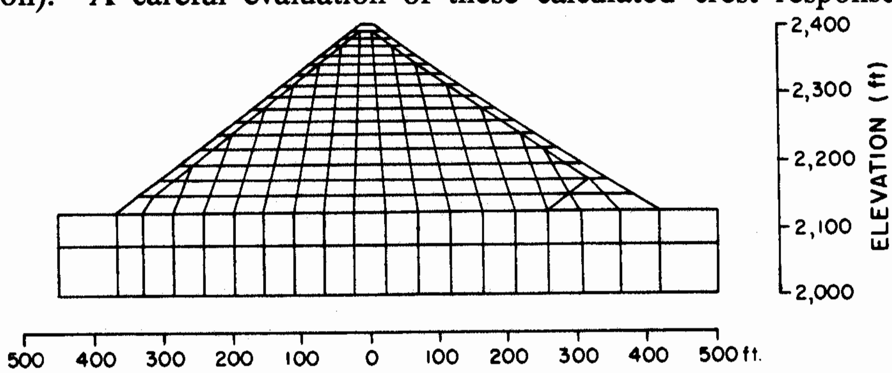

Fig. 12: Finite Element Mesh Used to Model Cogswell Dam

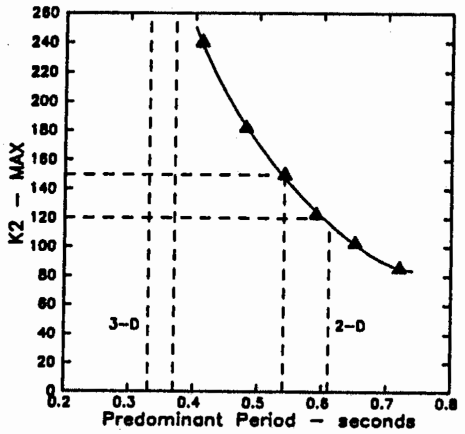

Fig. 13: Predominant Period vs. $K_{2,max}$

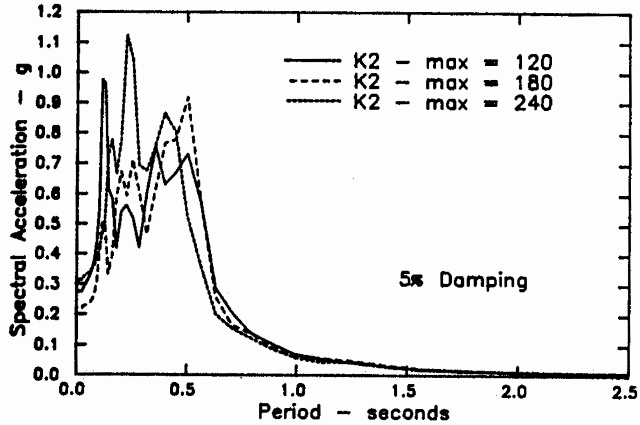

Fig. 14: Crest Response Spectra Calculated with Different Values of $K_{2,max}$

however, with allowance for 2-D vs. 3-D period response shifts, suggested that optimum overall spectral response modelling was achieved with $K_{2,max} \approx$ 150 to 180.

Overall, based on these studies, it was concluded that best modelling of the recorded response of the Cogswell Dam rockfill embankment was achieved with $K_{2,max} \approx$ 150. This compares well with the value of $K_{2,max} \approx$ 100 to 130 developed by Lai and Seed (1985) by similar back analyses of the recorded response of two similar rockfill dams, as they used the modulus degradation curve recommended by Seed et al. (1984) for sands. Considering the representative shear strain levels in their analyses, and substituting the modulus degradation curve for gravelly soils used in this study, their corresponding estimates would have been $K_{2,max} \approx$ 130 to 170, in very good agreement with the value developed in these studies for a similar, loosely dumped and sluiced rockfill mass.

SUMMARY AND CONCLUSIONS

Good agreement between the observed response characteristics of Puddingstone Dam and response characteristics predicted using both simple empirical methods as well as 2-D finite element analyses, based on established methods for evaluation and modelling of strain-dependent dynamic shear moduli and damping, provides good support for these modelling and analytical techniques. Proper interpretation of the analytically predicted response characteristics requires appropriate consideration of three-dimensional effects not modelled in the 1-D and 2-D analyses performed. These effects were only moderate, however, 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 an excellent opportunity to obtain additional insight into the parameters suitable for modelling 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 $(K_2)_{max} \approx$ 120 to 150 as best modelling the behavior of this loosely-dumped and then sluiced rockfill, in good general agreement with the values of $(K_2)_{max}$ suggested by Lai and Seed (1985) and Seed et al., (1984) based on previous, similar back-analyses of the response of several loosely dumped 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 good 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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