INVESTIGATION OF THE RESPONSE OF COGSWELL DAM IN THE WHITTIER NARROWS EARTHQUAKE OF OCTOBER 1, 1987

by

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DISCLAIMER

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PREFACE

The California Strong Motion Instrumentation Program (CSMIP) in the Division of Mines and Geology of the California Department of Conservation promotes and facilitates the improvement of seismic codes through the Data Interpretation Project. The objective of the this project is to increase the understanding of earthquake strong ground shaking and its effects on structures through interpretation and analysis studies of CSMIP and other applicable strong motion data. The ultimate goal is to accelerate the process by which lessons learned from earthquake data are incorporated into seismic code provisions and seismic design practices.

The specific objectives of the CSMIP Data Interpretation Project are to:

1. Understand the spatial variation and magnitude dependence of earthquake strong ground motion.

2. Understand the effects of earthquake motions on the response of geologic formations, buildings and lifeline structures.

3. Expedite the incorporation of knowledge of earthquake shaking into revision of seismic codes and practices.

4. Increase awareness within the seismological and earthquake engineering community about the effective usage of strong motion data.

5. Improve instrumentation methods and data processing techniques to maximize the usefulness of SMIP data. Develop data representations to increase the usefulness and the applicability to design engineers.

This report is the fifth in a series of CSMIP data utilization reports designed to transfer recent research findings on strong-motion data to practicing seismic design professionals and earth scientists. CSMIP extends its appreciation to the members of the Strong Motion Instrumentation Advisory Committee and its subcommittees for their recommendations regarding the Data Interpretation Research Project.

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ABSTRACT

The Whittier Narrows Earthquake of October 1, 1987 ($M_L \approx 5.9$) shook Cogswell Dam, a concrete faced rockfill dam, which had previously been instrumented as part of the California Strong Motion Instrumentation Program (CSMIP). The maximum recorded crest acceleration was 0.15 g. The resulting recorded accelograms 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. In particular, the recorded accelograms provided a rare opportunity to investigate the in-situ dynamic properties of rockfill materials. Presented in this report are the results of dynamic analysis studies of the response of Cogswell Dam to the 1987 Whittier Narrows Earthquake and the subsequent back-calculated estimates of the in-situ dynamic properties of the rockfill materials which comprise Cogswell Dam. Dynamic moduli and damping factors back-calculated from this valuable field case history were found to be in good agreement with similar values back-calculated from other recent case histories. Dynamic modulus degradation relationships, as a function of shear strain, were found to be better modelled by recent relationships proposed specifically for gravelly soils (Seed et al., 1984), than by the heretofore more widely used relationships proposed for sandy soils (e.g. Seed and Idriss, 1970, Seed et al., 1984). In addition to providing an opportunity for evaluation of in-situ dynamic properties of rockfill, this case study also provided an opportunity for evaluation of the ability of two-dimensional (plane strain) dynamic finite element analyses to model the fully three-dimensional behavior of a relatively tall dam in a narrow, steep-walled, V-shaped canyon. The results of these analyses suggest that three-dimensional effects are significant for this type of geometry, and that these are not well modelled by two-dimensional analysis methods.

ACKNOWLEDGEMENTS

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Chapter 1

INTRODUCTION

Considerable attention has been devoted over the past twenty years to development and verification of modelling and analysis techniques for evaluation of the seismic response of earth and rockfill dams. The recorded responses of earth dams and earth embankments during seismic loading provide the necessary field evidence against which the engineering procedures used to perform such dynamic analyses can be checked. In the case of rockfill dams, full-scale field response records are particularly valuable as they provide an opportunity to back-calculate the dynamic properties of the rockfill which, unlike sands, cannot easily be measured in the laboratory or in-situ. Thus, the response data recorded on the Cogswell Dam by the State of California Strong Motion Instrumentation Program (CSMIP) during the Whittier Narrows Earthquake of October 1, 1987 provided a rare opportunity to investigate the dynamic properties of rockfill materials as well as an opportunity to evaluate the accuracy of present engineering dynamic analysis procedures.

Figure 1-1 shows the location of Cogswell Dam, which is located in Los Angeles County, California, about 15 miles to the northeast of the City of Los Angeles. As shown in Figure 1-2, the Sierra Madre and San Andreas fault zones are located only 8 and 14 miles from the dam site, respectively. These fault systems are considered capable of producing earthquakes of magnitude 7 to 7½ and 8 to 8½, respectively. In addition, southern branches of the San Gabriel fault zone pass within less than 3 miles of the dam site, and may be capable of producing earthquakes of magnitude 7 or greater. Because of the area's high seismicity, and the interesting geometry and material composition of the dam, Cogswell Dam was selected by the State of California Strong Motion Instrumentation Program for

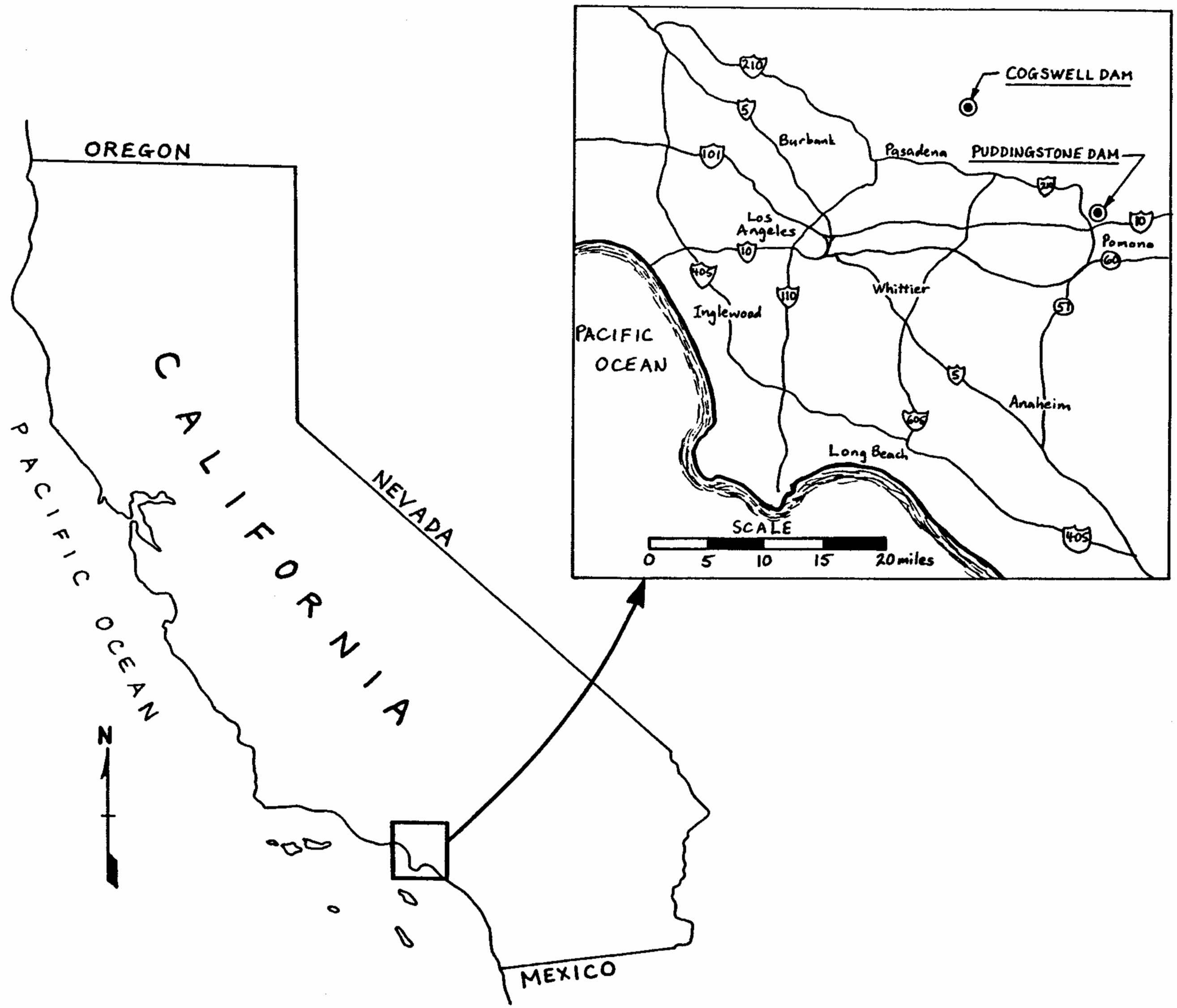


Figure 1-1: LOCATION OF COGSWELL DAM

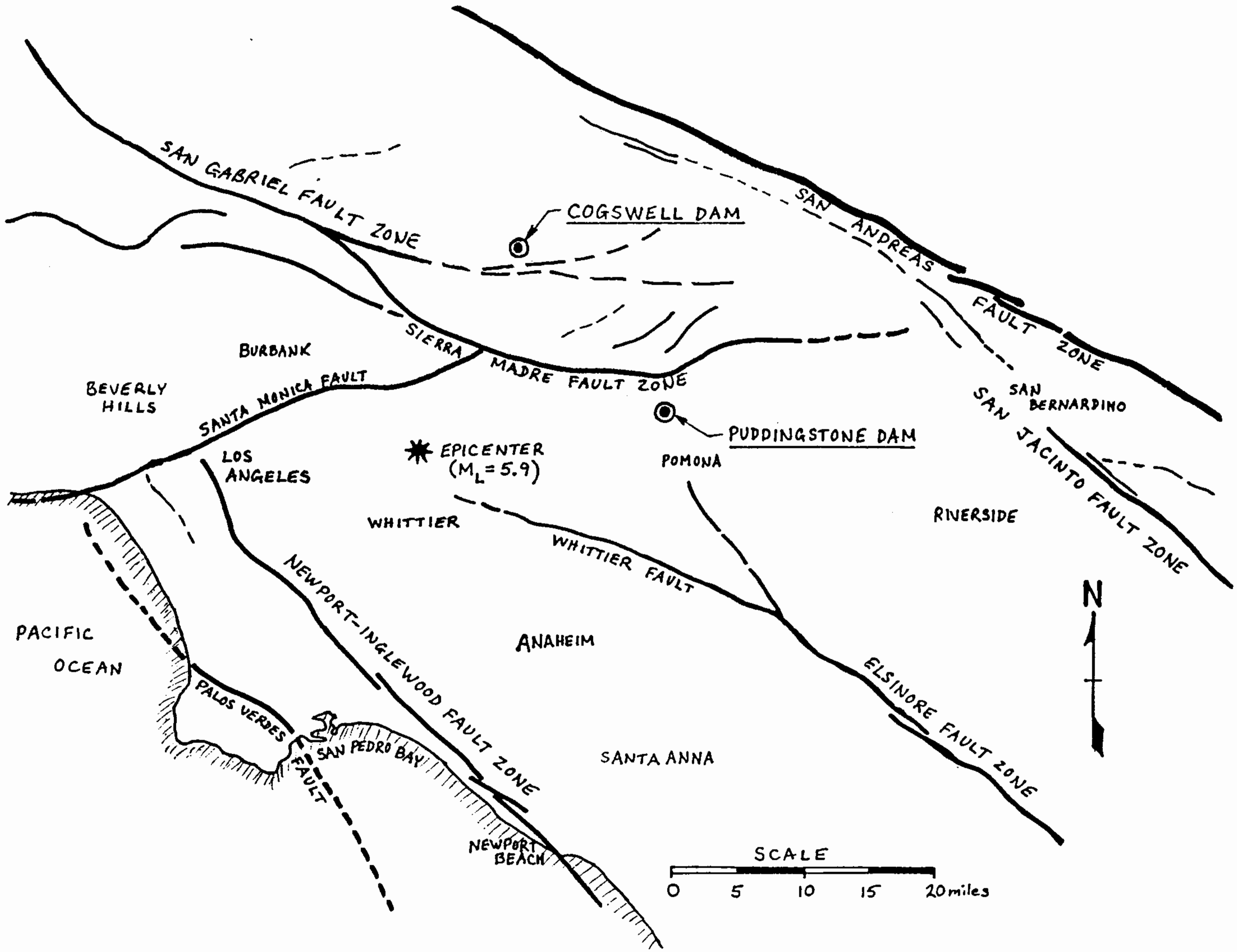


Figure 1-2: **FAULTS IN THE VICINITY OF COGSWELL DAM**

comprehensive instrumentation to investigate the seismic response of rockfill dams during strong earthquake shaking.

In this study, empirical relationships and two-dimensional dynamic response analysis methods are used to study and to predict the observed response of Cogswell Dam to the Whittier Narrows Earthquake of October 1, 1987. As shown in Figure 1-2, this earthquake, with a magnitude of $M_L \approx 5.9$, occurred on a previously unrecognized segment of the Whittier Fault with an epicenter approximately 18 miles southwest of the dam. This event produced maximum bedrock accelerations of approximately 0.06 g at the dam site, and the maximum acceleration recorded at the crest of the dam was 0.15 g. Comparison between the observed (recorded) response characteristics and the results of dynamic response analyses provides a basis for improving our understanding of the seismic response of earth and rockfill dams, and serves to validate the current use of some of these analytical models and analysis techniques. Specifically, this study attempts to evaluate the predominant period of Cogswell Dam, to predict the peak acceleration and maximum spectral acceleration at the crest of the embankment, to predict the correct shape of the acceleration response spectra at the crest of the embankment, and to estimate the actual dynamic properties of the rockfill material comprising the dam.

Chapter Two presents a description of the Cogswell Dam, and a discussion of the characteristics and engineering properties of the rockfill in the dam. Chapter Three presents a description of the instrumentation system installed by CSMIP to record strong motion response data for the dam and abutments, as well as the processed response data obtained during the Whittier Narrows Earthquake of October 1, 1987. Numerical modelling and seismic response analyses are described in Chapter Four, and the results of analyses performed using two-dimensional response analysis techniques are compared with observed field response data. These analyses, using two-dimensional dynamic finite element analysis techniques, were performed in order to: (1) evaluate the suitability of applying two-dimensional analytical procedures to dams with highly three-dimensional geometries (Cogswell Dam has a crest length to maximum crest height ratio of only 2.1:1); and (2) investigate the dynamic properties of the embankment's rockfill materials. Chapter Five presents a brief summary of the results of these studies and the principal conclusions drawn from them.

Chapter 2

DESCRIPTION OF COGSWELL DAM

Cogswell Dam is located in the San Gabriel Mountains in Los Angeles County, just northeast of the city of Los Angeles (Figure 1-1). This rockfill dam was constructed by the Los Angeles County Flood Control District beginning in 1932 and finally completed in 1947 when a permanent concrete facing was placed. The dam retains Cogswell Reservoir, with a capacity of 8850 acre-feet and a surface area of 146 acres, for the purposes of flood control and water conservation.

Cogswell Dam is a largely homogeneous loosely dumped rockfill embankment with an upstream concrete facing slab and concrete cutoff walls. The dam is wedged in a steep walled canyon as illustrated by the plan view in Figure 2-1 and the cross-sections presented in Figure 2-2. The dam has a maximum height of 280 ft above bedrock and a crest length of 585 ft, for a length to height ratio of only 2.1:1. The 18 foot wide crest is at Elevation 2405.0 feet, which provides 20.0 feet of freeboard above the certified reservoir storage level of Elevation 2385.0 feet. The upstream face has a slope of 1.25:1 near the crest, flattening slightly to 1.35:1 near the toe. The downstream face has a slightly flatter slope, beginning at 1.30:1 near the crest and flattening to 1.6:1 at the toe. The spillway and outlet works are both located in the right abutment.

Three size classifications of quarried rock were specified for use in the embankment. As shown in Figure 2-2, Class A rockfill comprises the main body of the dam. Class A rockfill was a well graded mixture with the following specifications by weight: 40 percent from quarry chips to 1000 lbs; 30 percent between 1000 lbs to 3000 lbs; 30 percent between 3000 lbs and 14,000 lbs; and no more than 3 percent quarry dust. A second specification, Class B rockfill, was used to place both a 50 foot high downstream toe and a downstream facing layer varying

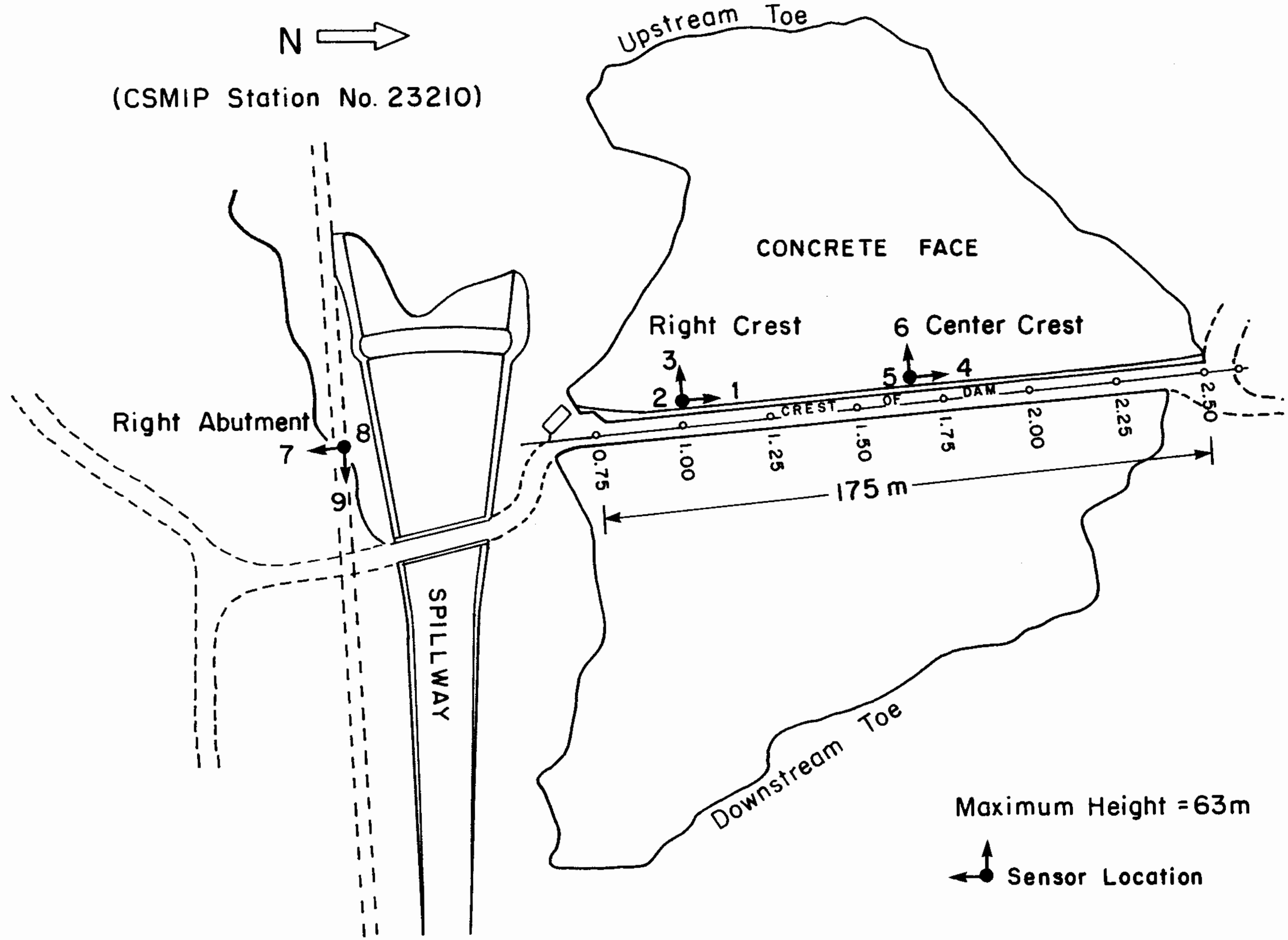


Figure 2-1: PLAN VIEW OF COGSWELL DAM SHOWING INSTRUMENT LOCATIONS

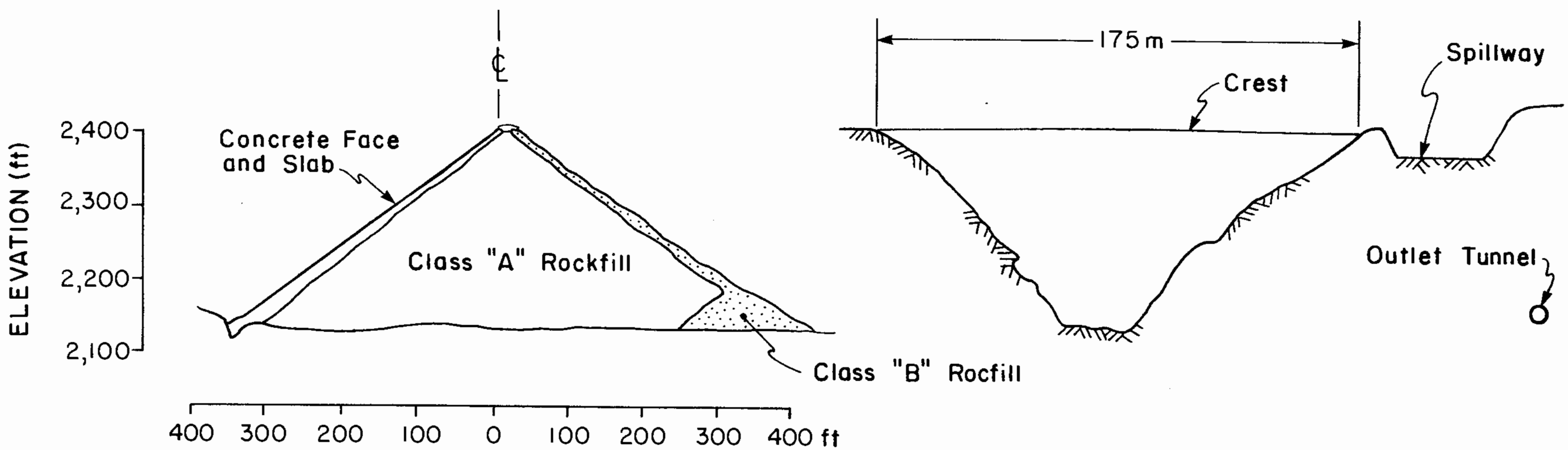


Figure 2-2: TRANSVERSE AND LONGITUDINAL CROSS-SECTIONS; COGSWELL DAM

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. The third specification, Class C rockfill, was used in placing 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 was obtained from a quarry in Devils Canyon, and quality control tests indicated an average compressive strength of 6,600 psi, an average unit weight of 174.7 pcf, and a 5.04 percent breakdown by the Rock Drop test.

The foundation of Cogswell Dam was described by a Board of Consultants prior to construction, as ". . . a dense granitic rock, technically known as alaskite. It is hard, brittle, and has been sheared and fractured."

A concrete cutoff wall was built along the contact between the foundation and the upstream face of the dam. The cutoff wall was 10 feet wide at the top, tapering to a minimum of 5 feet wide at its bottom. The wall was extended well into sound bedrock, so that the depth of the wall is believed to range between 12 feet and 54 feet. The area beneath the wall was pressure grouted to seal the foundation.

The upstream facing slab connects with the concrete cutoff wall to provide the primary seepage barrier for the dam. The design of the upstream facing slab evolved in response to unexpectedly large settlements which will subsequently be described.

Placement of the rockfill began in the spring of 1932 and progressed by dry dumping of 25 foot lifts. The rockfill was placed dry instead of by the usual sluicing procedure because of the scarcity of water at the Cogswell site. By the fall of 1933, most of the rockfill had been placed and the original upstream concrete facing was nearly completed.

An intense storm during the latter stages of construction on December 31, 1933 and January 1, 1934, resulted in flooding which had the effect of sluicing the dam; this resulted in excessively large settlements of the loose rockfill. This storm alone produced 5.8 feet of settlement at the crest of the dam near the south abutment. All together, 13.6 feet of settlement occurred as a result of rains during December of 1933 and January and February of 1934. One of the effects of these large settlements was the partial destruction of the original upstream concrete facing.

The repair of the dam began with a thorough sluicing of the rockfill until settlements were no longer significant. At that point, additional rockfill was placed to reshape the dam and then a temporary timber facing was installed on the upstream face. The timber facing, completed in April of 1935, was left in place during operation of the dam for 10 years until monitoring suggested that further settlements could be expected to be negligible. In 1947, the timber facing was replaced with a permanent reinforced concrete facing composed of a single layer of 30 foot square reinforced concrete slabs ranging in thickness from 8 inches near the crest to 24 inches at the base.

The instrumentation at Cogswell Dam includes six piezometers, 23 survey monuments, seven leakage measurement points, and nine strong motion accelographs. The locations of these accelerographs, and the strong motion recordings obtained during the 1987 Whittier Narrows Earthquake, are discussed in Chapter Three.

Chapter 3

SEISMIC INSTRUMENTATION AND PERFORMANCE DURING EARTHQUAKE LOADING

Cogswell Dam has, since its construction, been shaken by 17 earthquakes of magnitude 5.0 or greater occurring within 75 miles of the dam. The only earthquake to produce any damage was a magnitude 5.0 earthquake on August 2, 1952, with an epicenter 23 miles from the dam. This earthquake resulted in a crack in a metal expansion joint and a subsequent increase in leakage through the dam. Repairs were completed shortly after the earthquake. The greatest magnitude earthquake felt by Cogswell Dam was a magnitude 6.4 earthquake on February 9, 1971, with an epicenter 27 miles west of the dam. This earthquake did not result in any damage to Cogswell Dam. Unfortunately, many of these earthquakes occurred prior to the installation of strong motion accelographs so recorded motions are not available.

In more recent years, Cogswell Dam has been instrumented by the State of California Strong Motion Instrumentation Program (CSMIP) so that recorded motions could be obtained for the purpose of investigating the dynamic behavior of rockfill dams. As part of the CSMIP program, nine strong motion accelographs were placed on or near Cogswell Dam. The accelographs were placed in groups of three in order to record three components of motion: vertical; transverse (normal to the dam's axis); and longitudinal (parallel to the dam's axis). As shown previously in Figure 2-1, groups of accelographs were located at three locations: on the right abutment; at the center of the dam's crest; and at a point on the crest between the crest center and the right abutment.

In 1987, the Whittier Narrows Earthquake ($M_L \approx 5.9$) shook Cogswell Dam and recorded motions were subsequently obtained by the CSMIP instrumentation. The epicenter of this earthquake was 18 miles southwest of the dam. Maximum accelerations were approximately 0.06 g on the right abutment and 0.15 g at the center of the dam's crest. A survey of the dam following the earthquake indicated no apparent damage.

The 1987 Whittier Narrows Earthquake produced recorded motions in all 9 accelographs installed by the CSMIP. Accelograms for each of the three instrument locations are presented in Figures 3-1 through 3-3. For both the transverse and longitudinal motions at each location, response spectra have been calculated and these are presented in Figures 3-4 through 3-6. A summary of the recorded maximum surface accelerations and the calculated maximum spectral accelerations is given in Table 3-1.

The motions recorded at the right abutment show a maximum surface acceleration of 0.064 g in the transverse direction and 0.061 g in the longitudinal direction. The response spectra were similar for both components of motion, as shown in Figure 3-4. These response spectra indicate that the energy of the recorded rock motions from the right abutment is concentrated in the range of periods (T) less than about 0.25 seconds (frequencies greater than about 4 Hz), with the peak spectral acceleration occurring at a period of about 0.075 seconds (frequency of about 13 Hz).

The motions recorded on the right crest (between the right abutment and the crest center) show a maximum surface acceleration of 0.100 g in the transverse direction and 0.087 g in the longitudinal direction. The response spectra, shown in Figure 3-5, are similar for both components of motion, but the longitudinal motions produced a slightly greater maximum spectral acceleration. As expected, the motions at the right crest show amplification over the abutment motions, and a slight increase in predominant period. Note that the maximum spectral acceleration for the right crest's transverse motions occurred at a period of 0.20 seconds (5.0 Hz),

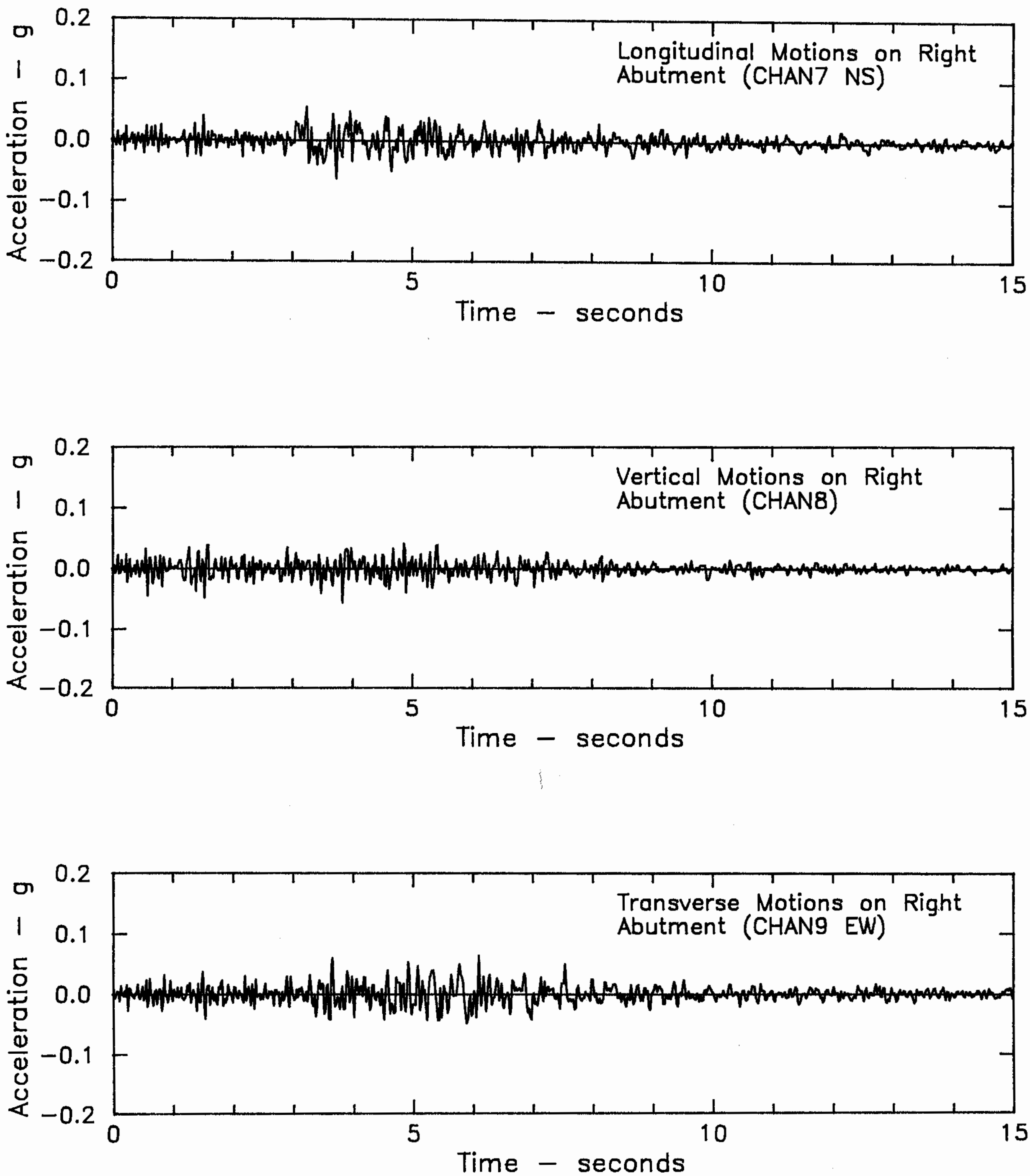


Figure 3-1: ACCELOGRAMS FOR MOTIONS RECORDED ON THE RIGHT ABUTMENT

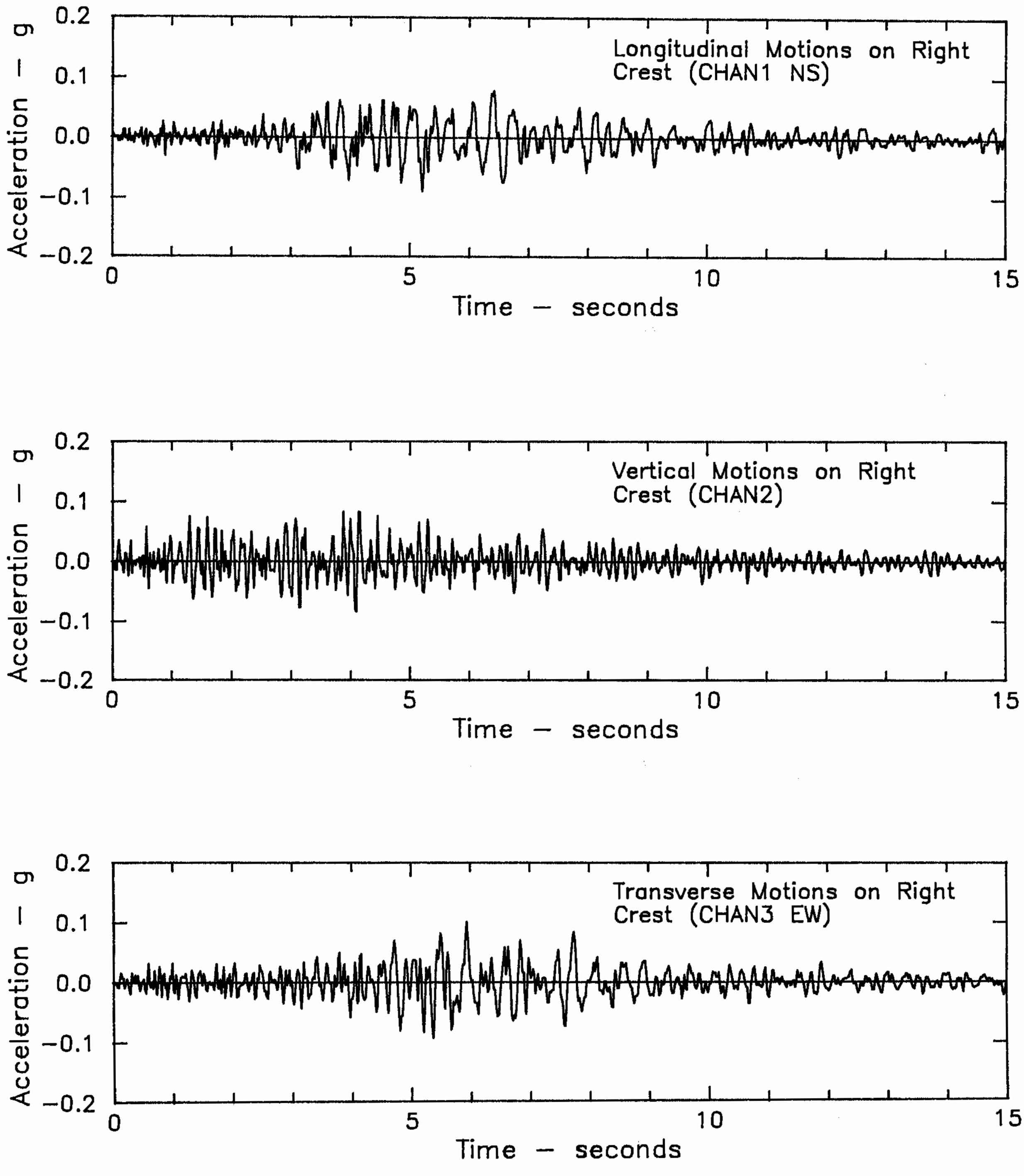


Figure 3-2: ACCELOGRAMS FOR MOTIONS RECORDED ON THE CREST, BETWEEN THE RIGHT ABUTMENT AND THE CREST CENTER

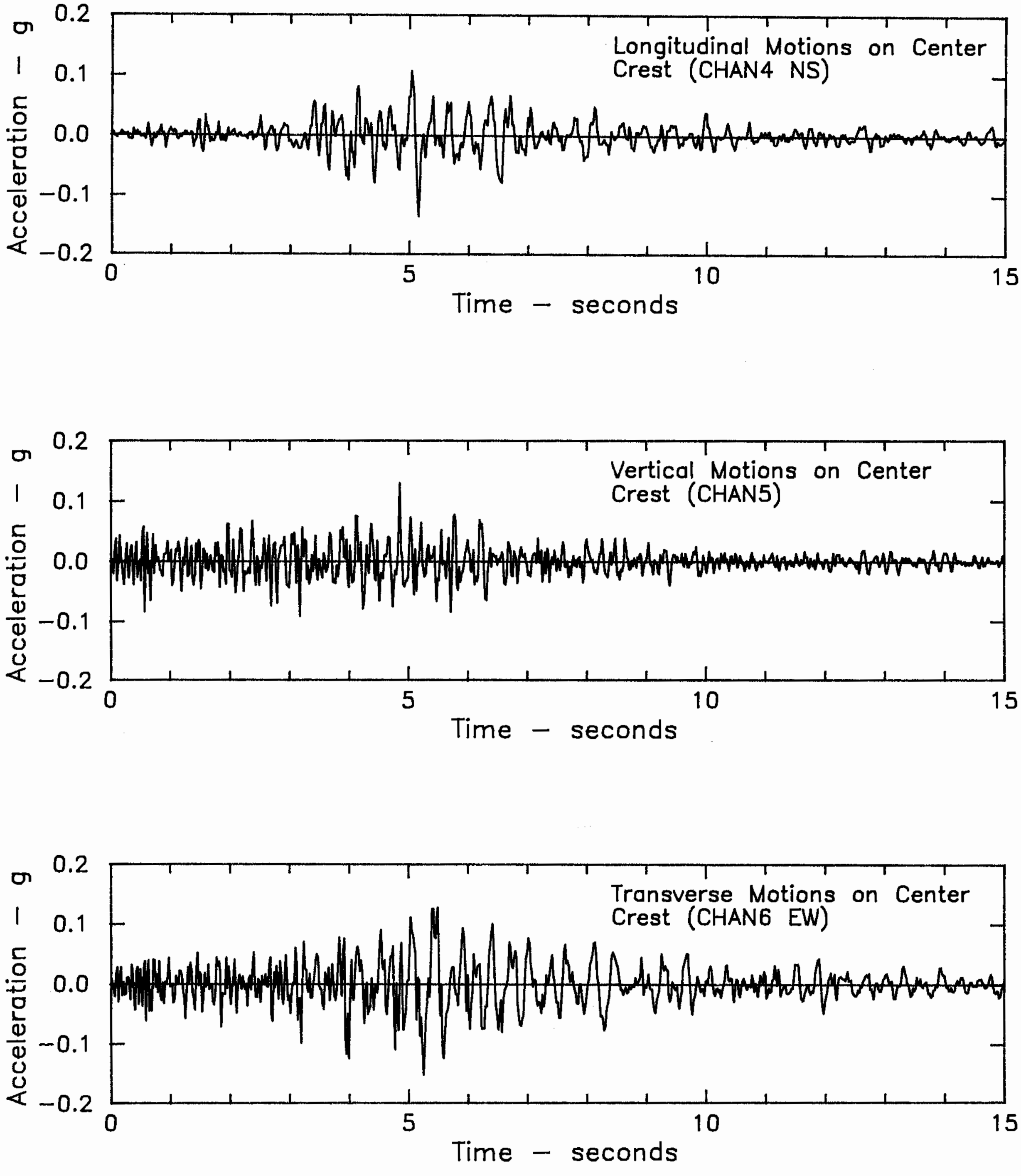


Figure 3-3: ACCELOGRAMS FOR MOTIONS RECORDED ON THE CREST, AT THE CENTER POINT

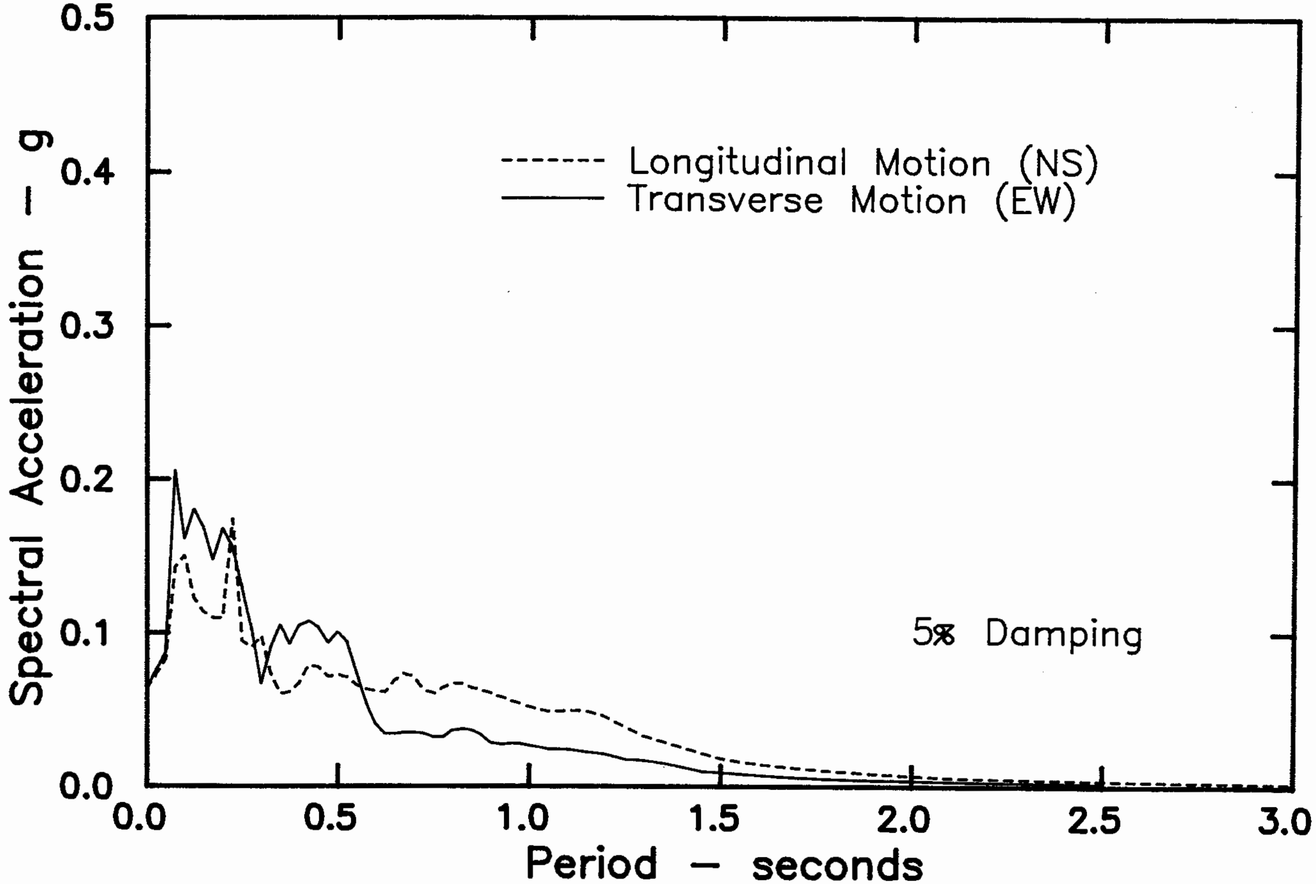


Figure 3-4: RESPONSE SPECTRA FOR MOTIONS RECORDED ON THE RIGHT ABUTMENT

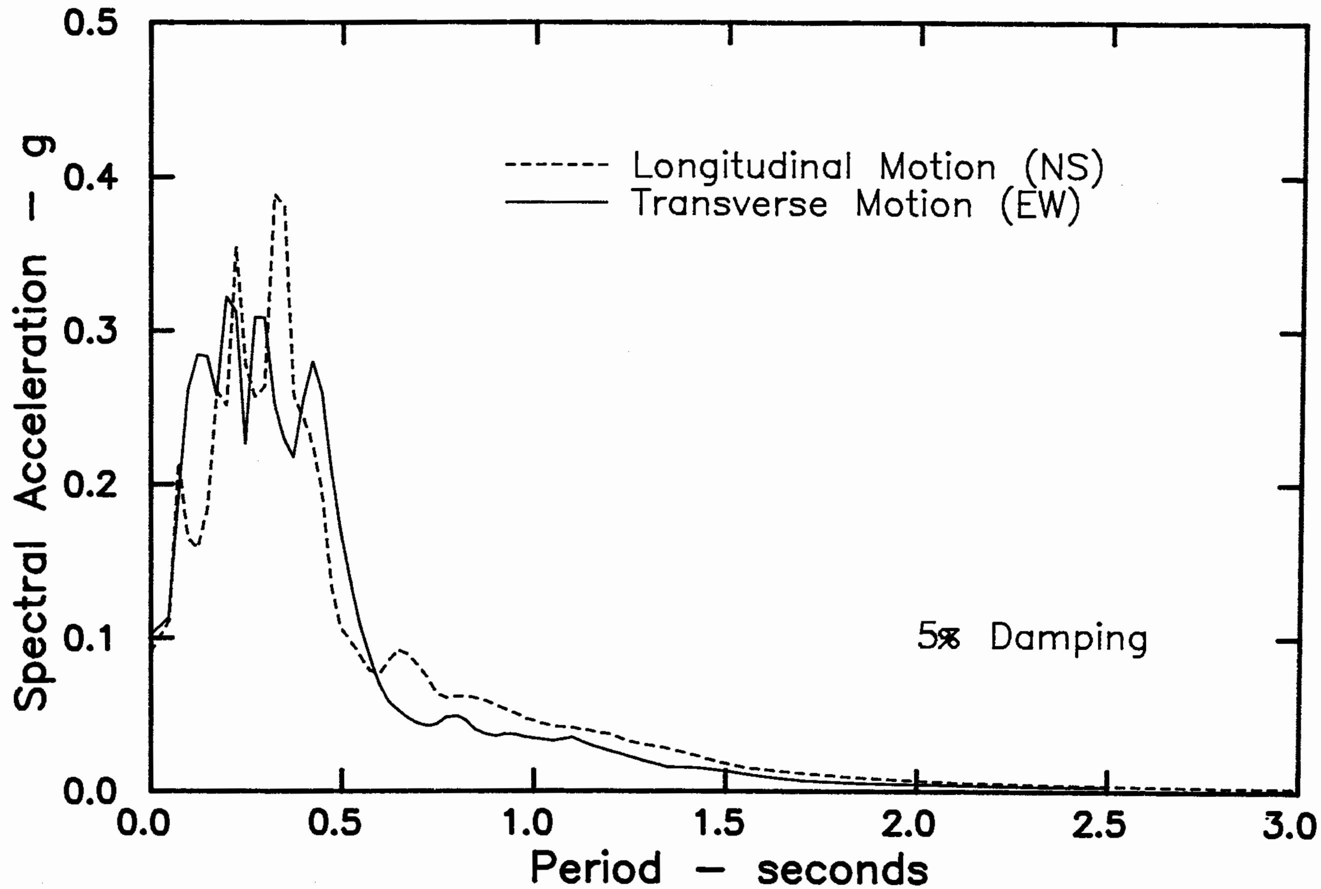


Figure 3-5: RESPONSE SPECTRA FOR MOTIONS RECORDED ON THE CREST, BETWEEN RIGHT ABUTMENT AND CREST CENTER

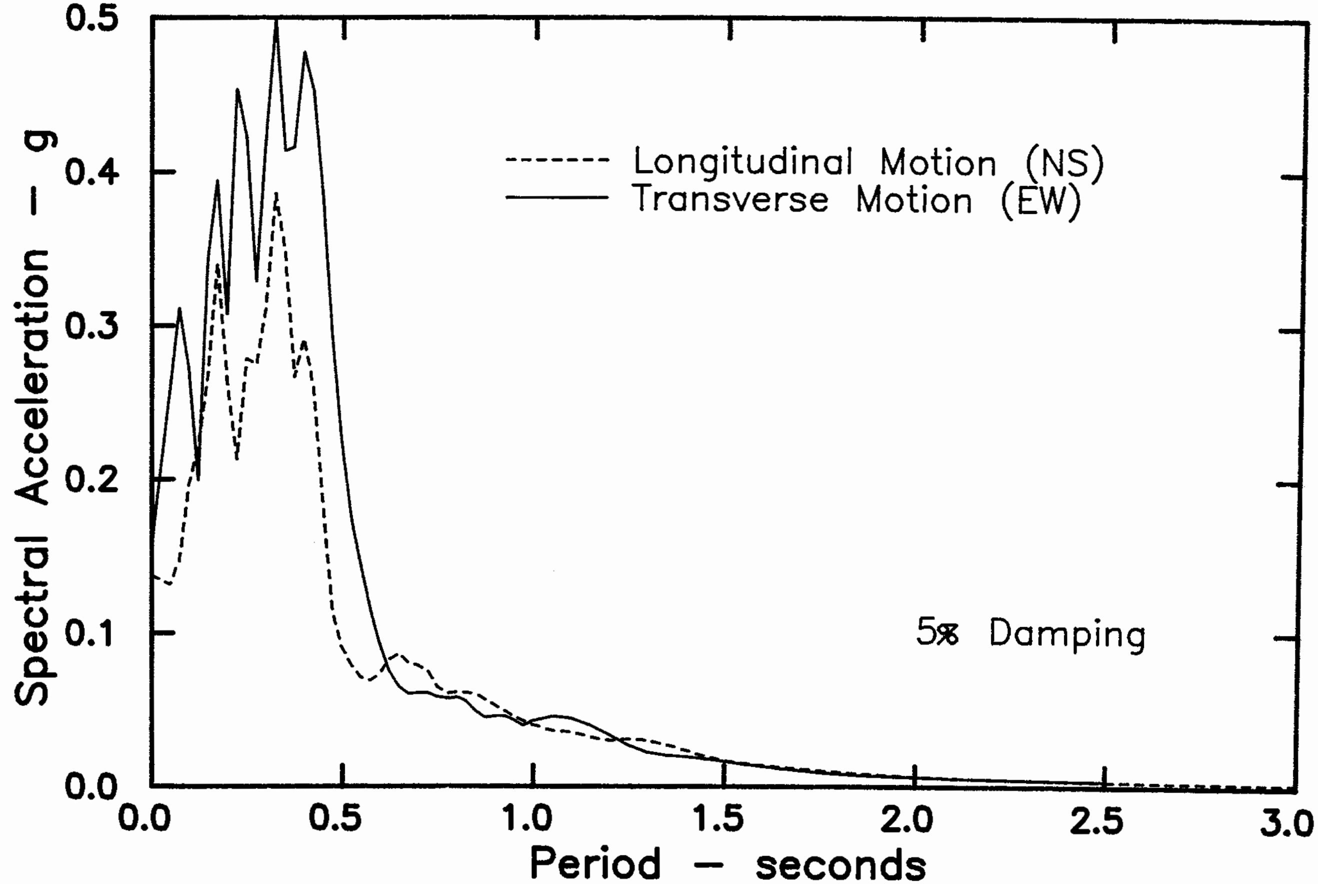


Figure 3-6: RESPONSE SPECTRA FOR MOTIONS RECORDED ON THE CREST, AT THE CENTER POINT

Table 3-1: Characteristics of Recorded Motions

Location	Direction of Motion	Maximum Surface Acceleration: $a_{g,max}$	Maximum Spectral Acceleration: $a_{s,max}$	Period at Which $a_{s,max}$ occurs
Abutment	Transverse	0.064	0.200	0.075 sec
Right Crest	Transverse	0.100	0.323	0.200 sec
Center Crest	Transverse	0.151	0.500	0.320 sec
Abutment	Longitudinal	0.061	0.175	0.225 sec
Right Crest	Longitudinal	0.087	0.391	0.325 sec
Center Crest	Longitudinal	0.137	0.385	0.320 sec
Abutment	Vertical	0.06	--	--
Right Crest	Vertical	0.11	--	--
Center Crest	Vertical	0.14	--	--

but that pronounced spectral peaks also occurred at periods of about 0.29 and 0.41 seconds (3.45 and 2.44 Hz respectively).

The motions recorded at the center of the crest are stronger than the motions at the right crest. Maximum surface accelerations at the crest center were 0.151 g in the transverse direction and 0.137 g in the longitudinal direction. The response spectra, shown in Figure 3-6, are in accord with expected patterns in that the transverse response is stronger than the longitudinal response, and the amplication of peak transverse response relative to the peak horizontal abutment acceleration is roughly a factor of two. Both of these central crest motions produced maximum spectral accelerations for a period of 0.32 seconds (3.1 Hz), but both also contain a strong spectral peak for a period of about 0.40 seconds.

As the transverse motions are considered to be of primary engineering interest, the response spectra for the transverse motions at the right abutment, right crest, and center crest are plotted together in Figure 3-7. In accord with expected patterns, the motions on the dam show amplification relative to the bedrock motions, particularly for periods close to the predominant period (T_p) of the dam, which appears to be between 0.32 and 0.40 seconds (3.1 and 2.5 Hz).

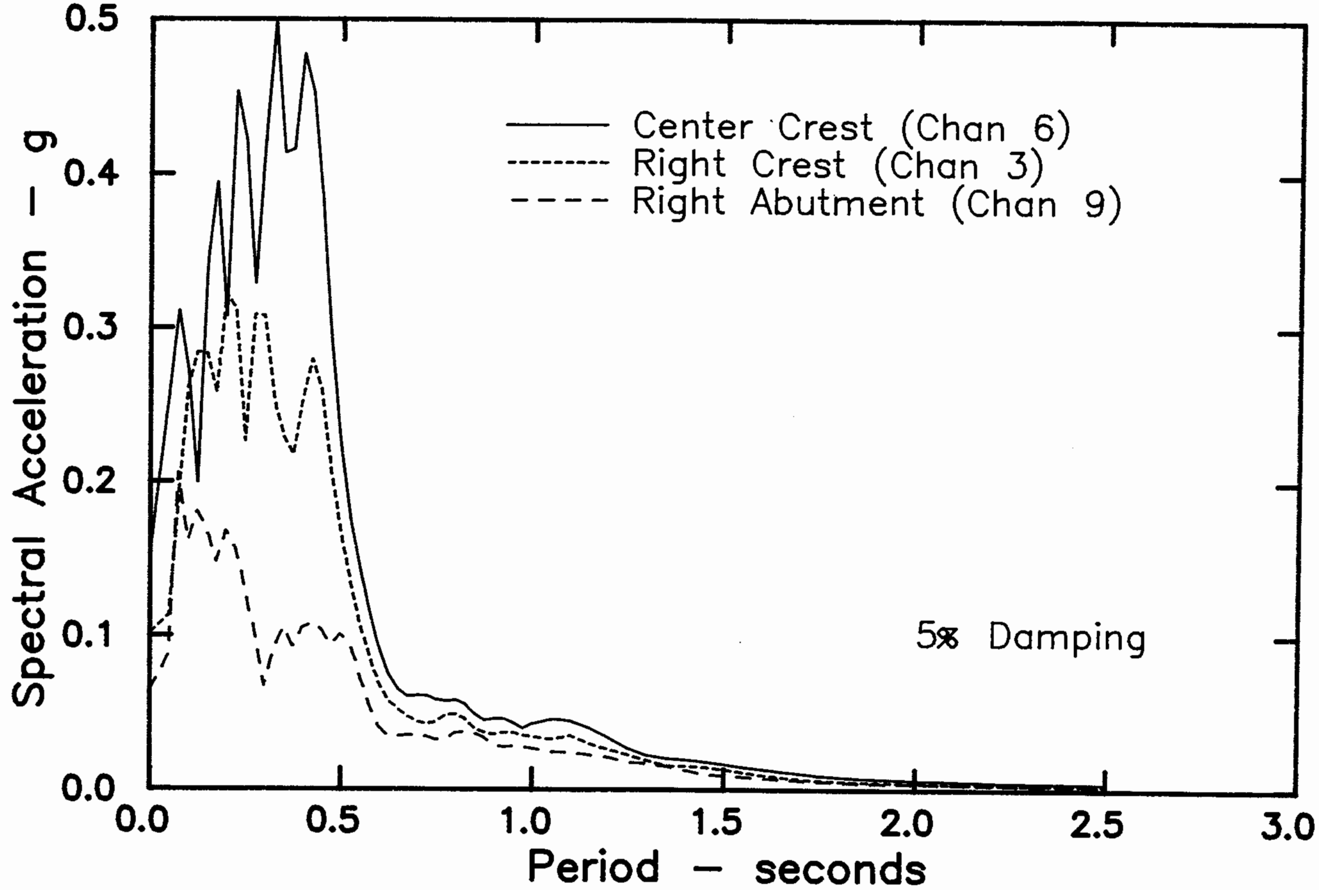


Figure 3-7: RESPONSE SPECTRA FOR RECORDED TRANSVERSE MOTIONS

Chapter 4

DYNAMIC RESPONSE STUDIES

Modelling the dynamic response of Cogswell Dam is complicated by the small length to height ratio of the dam, 2.1:1. For such a highly three-dimensional (3-D) geometry, the straightforward application of two-dimensional (2-D) analysis procedures would not be expected to give accurate results. However, 3-D analyses are much more time consuming than 2-D analyses and so are infrequently used in practice. In addition, the use of fully three-dimensional finite element analyses rapidly increases the required number of nodal points and elements to unmanageable numbers, and requires the use of a "coarse" mesh which can have a deleterious effect on computational accuracy. Accordingly, it was decided to begin with 2-D analyses to see if meaningful results could be obtained. Initially, 2-D analyses were performed but were not found to accurately reproduce the recorded response. However, the results of these 2-D analyses, combined with a more detailed interpretation of the recorded crest response, were found to be sufficient to investigate the dynamic properties of the rockfill materials in the dam.

Two-dimensional dynamic finite element analyses were performed using the program FLUSH (Lysmer et al., 1975), which uses the equivalent-linear complex response method to represent the strain dependent properties of a material. Hydrodynamic effects were not included as they are not considered important for rockfill dam analysis.

The 2-D analysis considers only transverse accelerations, which are considered to be of primary engineering interest. Thus the transverse motions recorded at the right abutment (Channel 9) were used as the input motions, with the calculated response then being compared against the transverse motions recorded at the center of the dam's crest (Channel 6).

There is some uncertainty regarding the use of the recorded abutment motions as input to the analysis because the abutment motions might be expected to reflect some level of amplification and there are no other bedrock motions available against which such a possibility can be checked. Amplifications of motions in rock formations above the crest of Long Valley Dam were observed by Lai and Seed (1985), who subsequently suggested such amplification may be due to topographic effects. Bray et al. (1990) observed significant differences between transverse motions recorded at three different abutment locations around Puddingstone Dam and found that when used as input motions, they resulted in significantly different calculated responses. Since the rock motions at Cogswell Dam were recorded on the right abutment, at a higher elevation than the dam's crest, there is some uncertainty as to how representative these motions are of the bedrock motions which occurred at the base of the dam.

The analyses were performed for a cross-section taken normal to the dam's axis, through the maximum height section shown previously in Figure 2-2. The finite element mesh for this cross-section, as shown in Figure 4-1, incorporated 215 elements and 237 nodal points. This mesh was found capable of accurately modelling frequencies of at least 12 Hz for the range of effective shear moduli obtained in the analyses. A 2-D analysis of a section through the right crest (location of Channel 3) was not performed, as Mejia and Seed (1981) found that for dams in narrow canyons with steep valley wall slopes ($L/H \approx 2:1$) a plane strain analysis through the quarter section yielded a significantly different response from that of a 3-D analysis.

The foundation of Cogswell Dam was modelled as a linear elastic material. Parameter studies showed that the calculated response was insensitive to the properties assigned to the rock and subsequent analyses used a shear wave velocity of 8000 feet/second and a damping ratio of 1.0 percent.

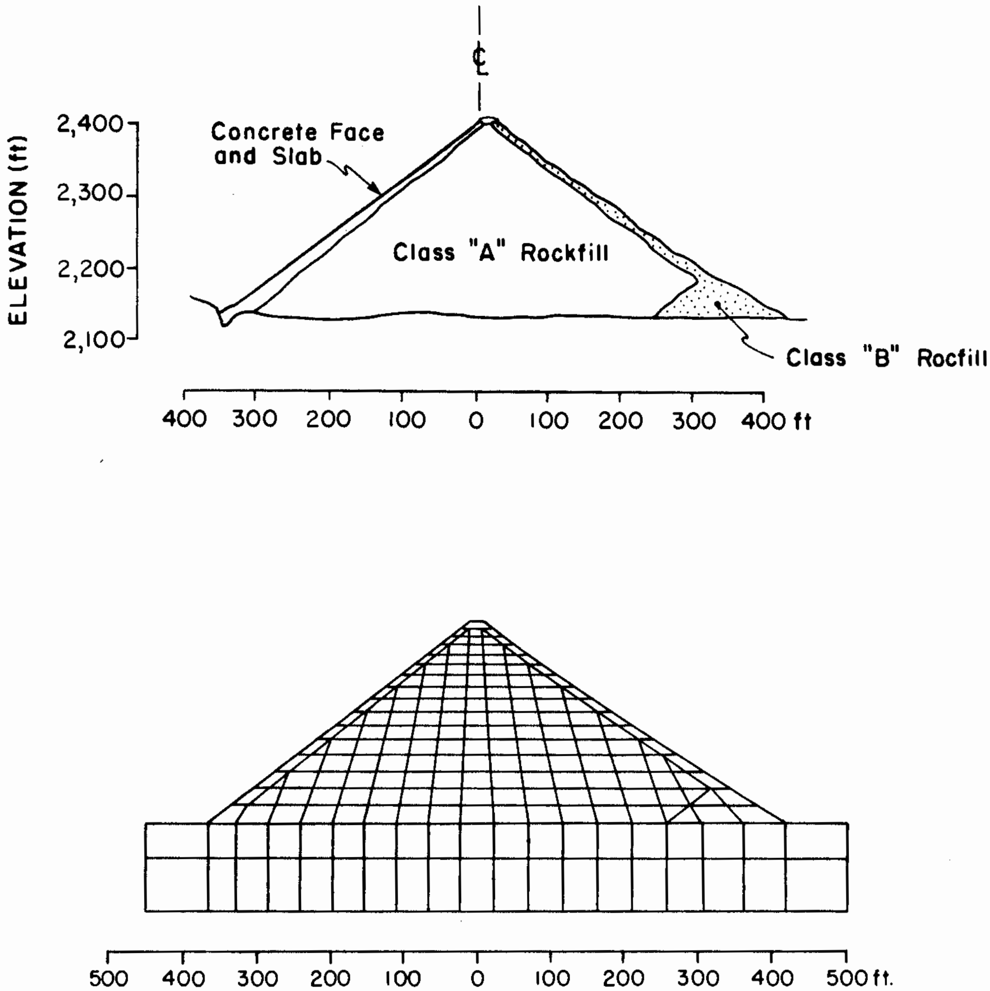


Figure 4-1: FINITE ELEMENT MESH USED FOR ANALYSES OF COGSWELL DAM

The maximum shear modulus at low shear strain levels ($10^{-4}\%$) for the rockfill materials was represented by the equation,

$$G(\text{psf}) \approx 1000\,(K_{2,max})\,\sqrt{\sigma_m'(\text{psf})} \qquad (1)$$

as proposed by Seed and Idriss (1970). The mean principal effective stress was obtained by static finite element analyses which will be described in a following section. The value of $K_{2,max}$ was varied during the analyses to obtain the best fit with observed response of the dam. The value of $K_{2,max}$ for the upstream facing, downstream facing, and the downstream toe was taken as 1/3 greater than the $K_{2,max}$ value used for the body of the dam. Since the results are dominated by the stiffness of the body of the dam, different analyses are identified by the value of $K_{2,max}$ assigned to the body of the dam.

The degradation of shear modulus with increasing shear strain for the rockfill was represented by the relationship suggested for gravel by Seed et al. (1984), as illustrated in Figure 4-2. The effective shear strain to be used in obtaining G/G_{max} by this relationship was taken as 65 percent of the maximum shear strain. As will be shown later, it is the combination of the G/G_{max} degradation curve and the $K_{2,max}$ value which determines the calculated response to a given loading condition.

The equivalent damping ratio for the rockfill was taken as being similar to that suggested for sands and gravel (Figure 4-3) by Seed et al. (1984). The upper bound shown in Figure 4-3 was found to give better results and was used for the analyses presented in this report. Lai (1985) also used this upper bound damping ratio curve in his back-analyses of rockfill dams.

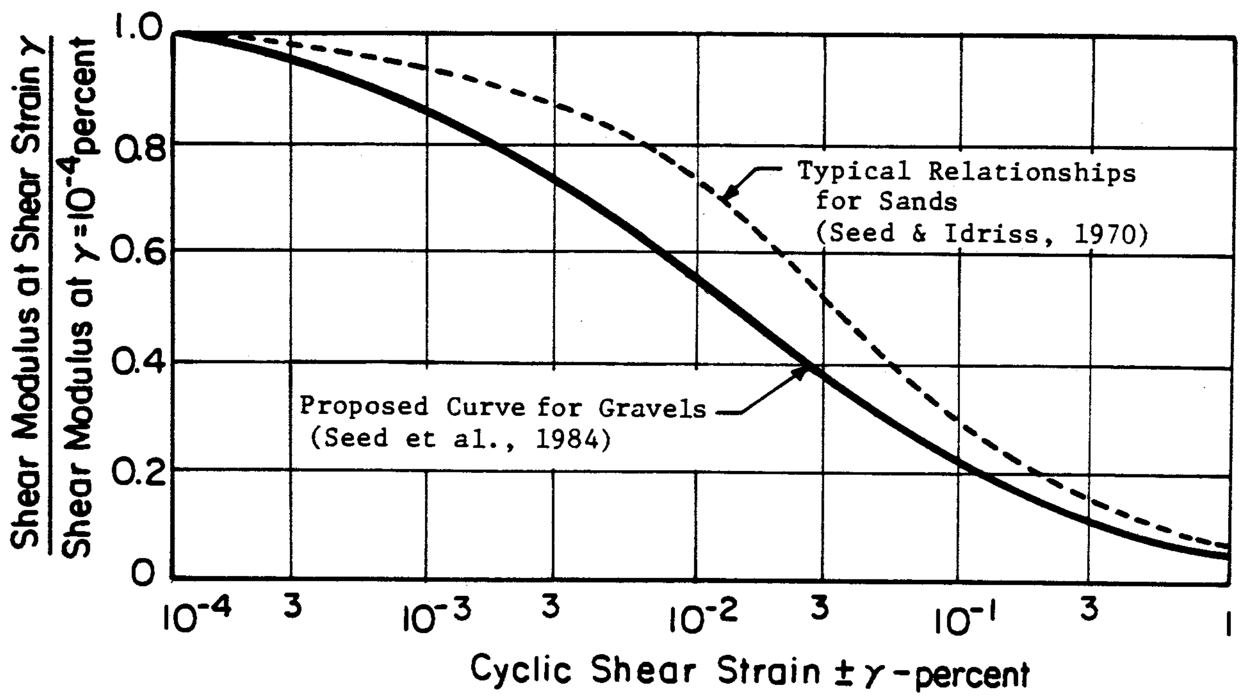


Figure 4-2: SHEAR MODULUS ATTENUATION CURVES FOR GRANULAR SOILS (after Lai and Seed, 1985)

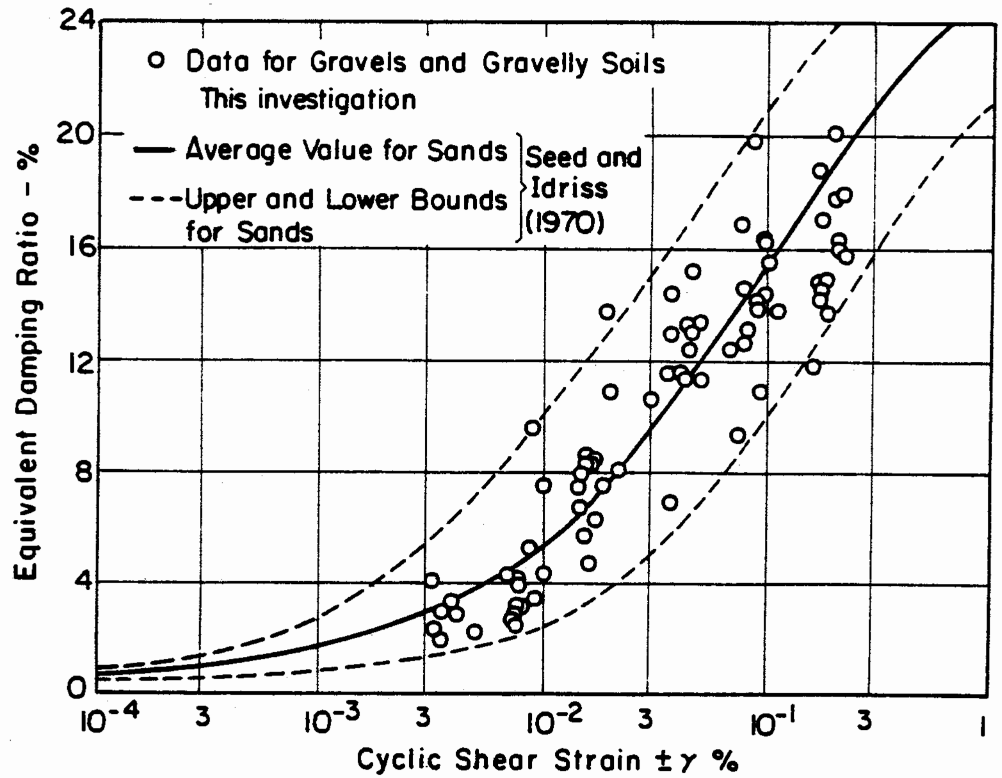


Figure 4-3: COMPARISON OF DAMPING RATIOS FOR GRAVELLY SOILS AND SANDS (after Seed et al., 1984)

Initial Static Stress Analysis

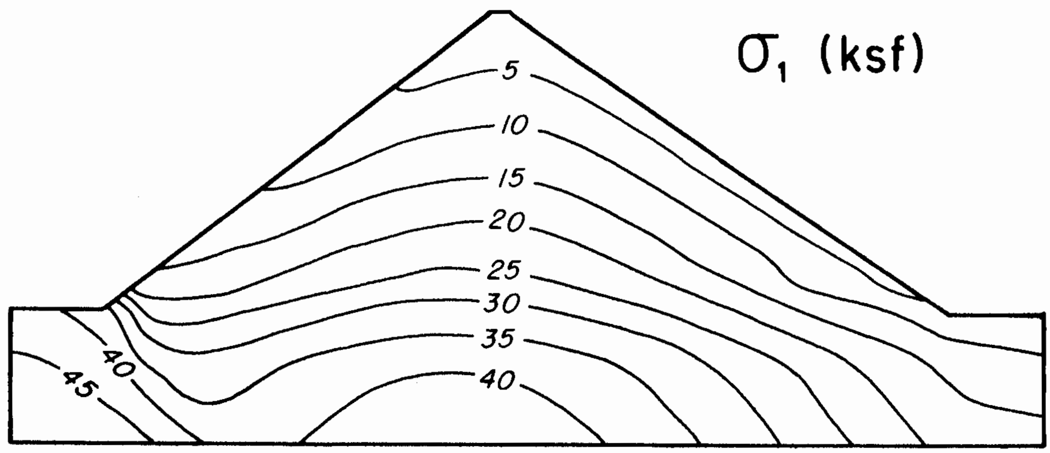
The initial static effective stresses throughout the dam were required in order to calculate the maximum dynamic shear moduli of the rockfill material (Eqn. 1). To estimate these static stresses, a 2-D static finite element analysis was performed for the same maximum cross-section used in the dynamic analyses. Lefebvre and Duncan (1971) suggest that for a L/H ratio of 2:1, a 2-D analysis of the maximum cross-section may overestimate the static stresses in the lower part of the embankment by as much as 40%. A 40% error in the mean principal stress corresponds to about 20% error in G_{max}. Given the other uncertainties involved, and the localized error in σ_m', the results of a 2-D analysis are considered sufficiently accurate for estimating shear moduli.

The static stress analysis was performed using the program FEADAM (Duncan et al. 1980), a plane strain finite element code for incremental nonlinear analysis. The finite element mesh used for the static stress analysis was the same mesh used for the dynamic analyses (Figure 4-1). Four node isoparametric elements were used and the nonlinear stress-strain and volumetric strain behavior of the rockfill was modelled using the hyperbolic model proposed by Duncan et al. (1980). The parameters used to model the rockfill materials are listed in Table 4-1. The analysis was performed in steps to incrementally model placement of the rockfill and the subsequent loads produced against the upstream face by the raising of the reservoir. The results of the static stress analysis are illustrated in Figures 4-4(a) and 4-4(b), which show contours of the major principal stress and of the principal stress ratio, σ_3'/σ_1'.

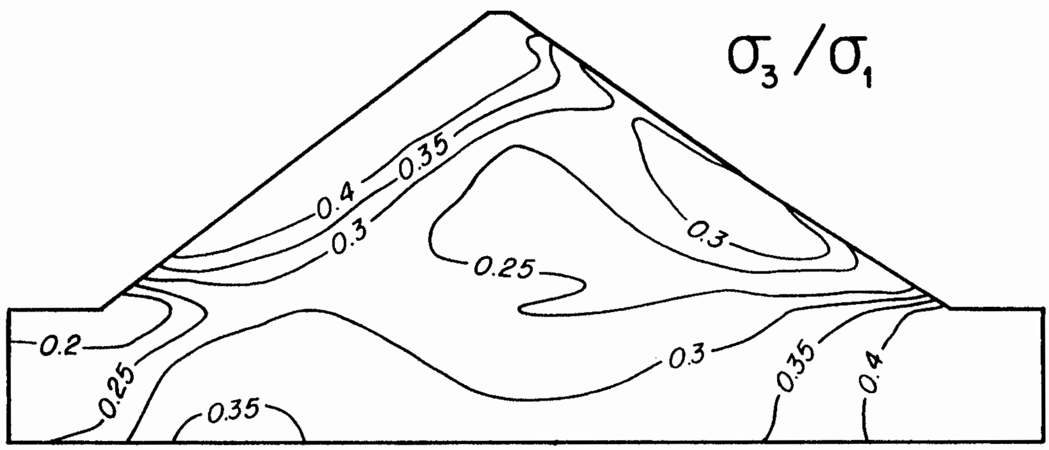
For the purpose of calculating the dynamic shear moduli to be used in the dynamic analyses, it is desirable to have a "smooth" estimate of the mean effective stress field. The resulting "smooth" transitions in shear moduli may avoid possible numerical errors due to the discretized nature of the analysis. Thus, it is convenient

Table 4-1: Soil Parameters for Initial Static Stress Analysis (Cogswell Dam)

Parameter	Symbol	Class A Rockfill (dumped)	Class B and C Rockfills
Unit Weight (t/m^3)	γ	1.78	1.87
Modulus Number	K	200	350
Elastic Unloading Modulus Number	K_{ur}	600	735
Modulus Exponent	n	0.55	0.39
Failure Ratio	R_f	0.55	0.62
Bulk Modulus Number	K_b	45	86
Bulk Modulus Exponent	m	0.57	0.38
Friction Angle	ϕ_o	47	48
Decrease in Friction Angle	$\Delta\phi$	7.5	8.3
Earth Pressure Coefficient	K_o	0.35	0.34
Cohesion (t/m^2)	c	--	--



(a) Major Principal Effective Stress (σ_1')



(b) Effective Principal Stress Ratio (σ_3'/σ_1')

Figure 4-4: INITIAL STATIC EFFECTIVE STRESSES to adopt a representative value of the principal stress ratio (σ_3'/σ_1') for the whole embankment and to assume a value for the intermediate principal stress (σ_2') so that the mean principal effective stress (σ_m') can be expressed as a constant proportion of the major principal effective stress (σ_1') which has a relatively "smooth" distribution throughout the embankment. For Cogswell Dam, a representative value of the principal stress ratio is 0.32 and with an intermediate principal stress of 0.63 σ_1', the mean principal effective stress can be expressed as 0.65 σ_1'. Thus, the maximum dynamic shear moduli of the rockfill materials were calculated using $\sigma_m' = 0.65\,\sigma_1'$ in Equation 1, and the values of σ_1' calculated for each element in the mesh.

Comparison Between Calculated and Recorded Responses

The recorded transverse motions at the center of the crest (Channel 6) of Cogswell Dam provide the data against which the 2-D dynamic analysis results can be checked. The response spectra for the transverse motion recorded at the center crest point, as described earlier in this study (see Figure 3-6), is a convenient measure against which calculated responses can be checked. So for each dynamic analysis performed, the response spectra were calculated at nodal points 109 and 126 (which both correspond to points at the dam crest.) The response spectra for both nodal points were essentially identical, so only the response spectra for nodal point 109 are presented.

To investigate the effects of input frequency cutoff on calculated response, analyses for a $K_{2,max}$ value of 150 were repeated with a frequency cutoff of 8 Hz, 10 Hz and 12 Hz. The difference in calculated response was found to be negligible between the results for the 10 Hz and 12 Hz cutoffs, so a cutoff frequency of 10 Hz was used for subsequent analyses.

Dynamic analyses were performed using $K_{2,max}$ values of 120, 150, 180 and 240 for the dumped rockfill comprising the majority of the dam. As previously mentioned, the $K_{2,max}$ values for the compacted rockfill on the dam faces were taken as one third greater than the value for the dumped rock. In all cases, the effective shear strain induced in the embankment ranged between 4.0 x 10^{-3} percent and 1.1 x 10^{-2} percent, with slightly higher strain levels occurring in analyses using the lower values of $K_{2,max}$. The central crest response spectra computed for each of these four values of $K_{2,max}$ are compared to the recorded response spectra in Figures 4-5 through 4-8.

It is readily apparent that the computed spectral accelerations at the crest of the dam are generally greater than the recorded spectral accelerations by about 40 to 80 percent, regardless of the value of $K_{2,max}$. If computed and recorded responses are instead compared on a basis of general shape and predominant period, then the best general match is obtained with the $K_{2,max}$ values of 150 and 180. The computed responses for $K_{2,max}$ values of 120 and 240 gave the poorest general match with recorded response, in that the maximum calculated spectral accelerations occurred at periods of about 0.11 sec (9 Hz).

Two additional analyses were performed using $K_{2,max}$ values of 80 and 100, because it was noted that as $K_{2,max}$ became smaller, the spectral acceleration became smaller for periods between 0.2 and 0.5 seconds. The response spectra for those two analyses are shown in Figures 4-9 and 4-10. For the $K_{2,max}$ value of 100, the maximum spectral acceleration increased to 2.0 g at a period of 0.117 sec (9 Hz) even though the spectral accelerations were reduced for higher periods. When $K_{2,max}$ was reduced to 80, the high spectral peak at a period of 0.117 sec was significantly reduced and the computed response spectra appeared to give the best match yet with respect to the overall magnitude of spectral accelerations. However, the reduced spectral peak at at a period of 0.117 sec is due in large part to inability

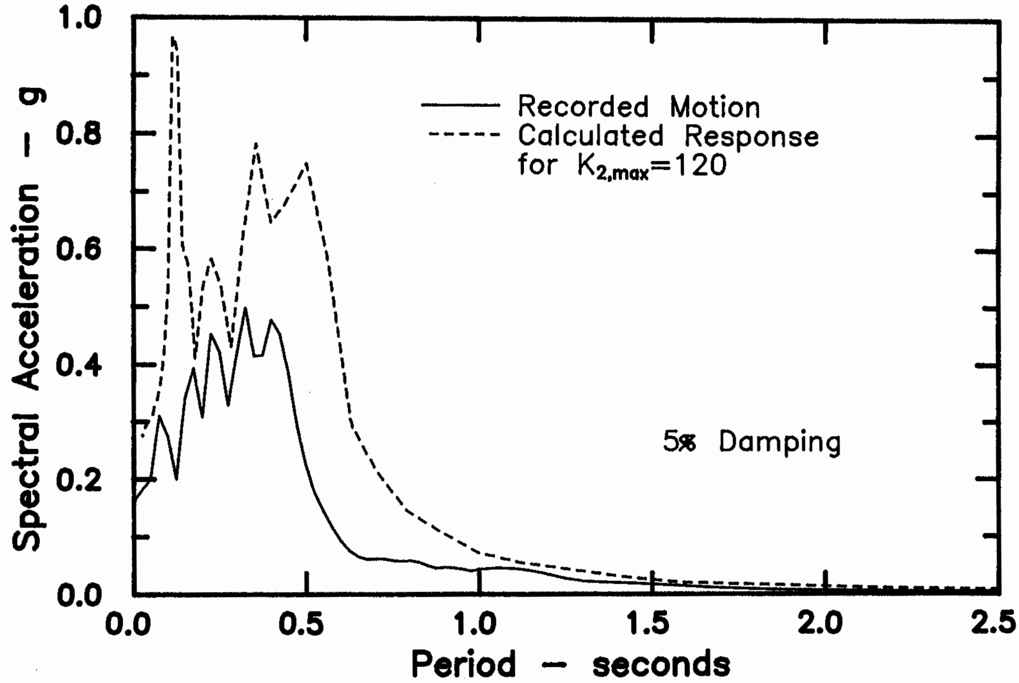


Figure 4-5: CALCULATED CREST RESPONSE FOR A $K_{2,max}$ OF 120

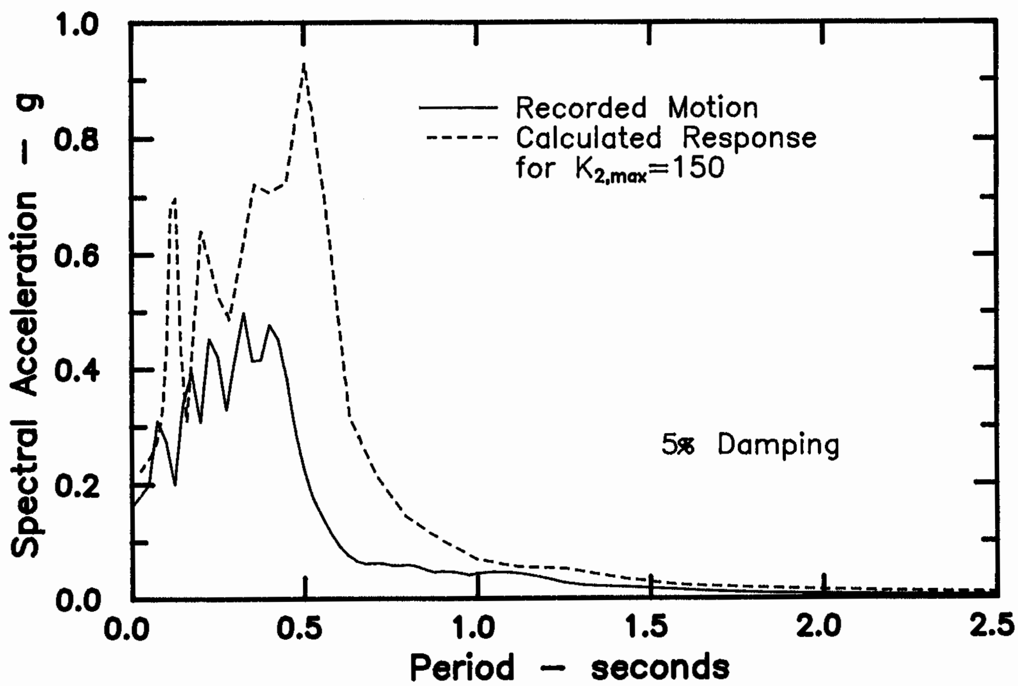


Figure 4-6: CALCULATED CREST RESPONSE FOR A $K_{2,max}$ OF 150

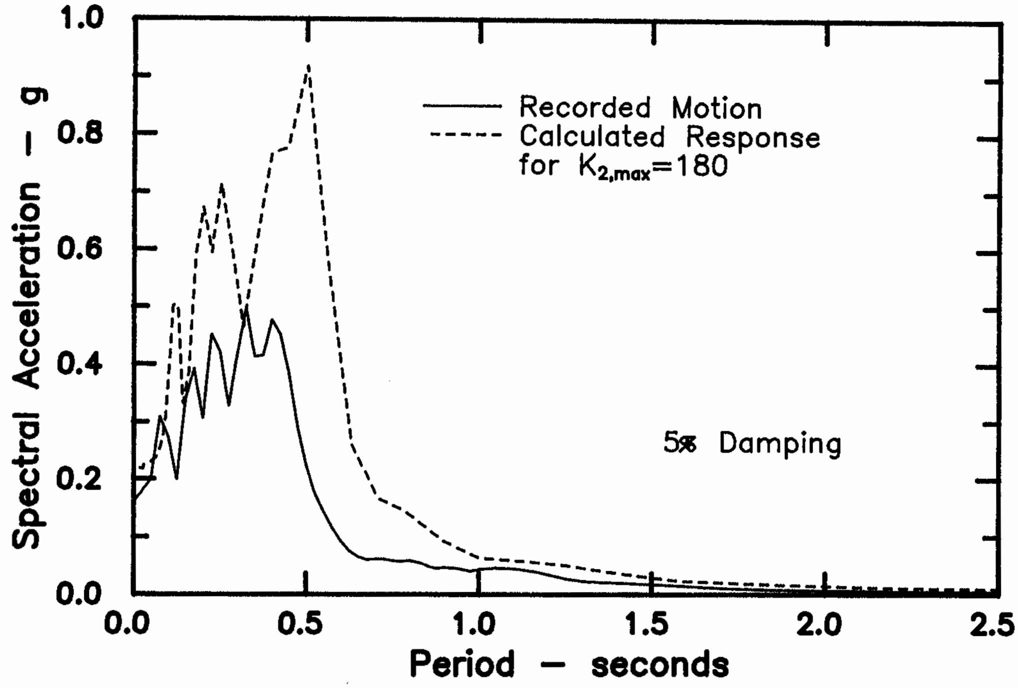


Figure 4-7: CALCULATED CREST RESPONSE FOR A $K_{2,max}$ OF 180

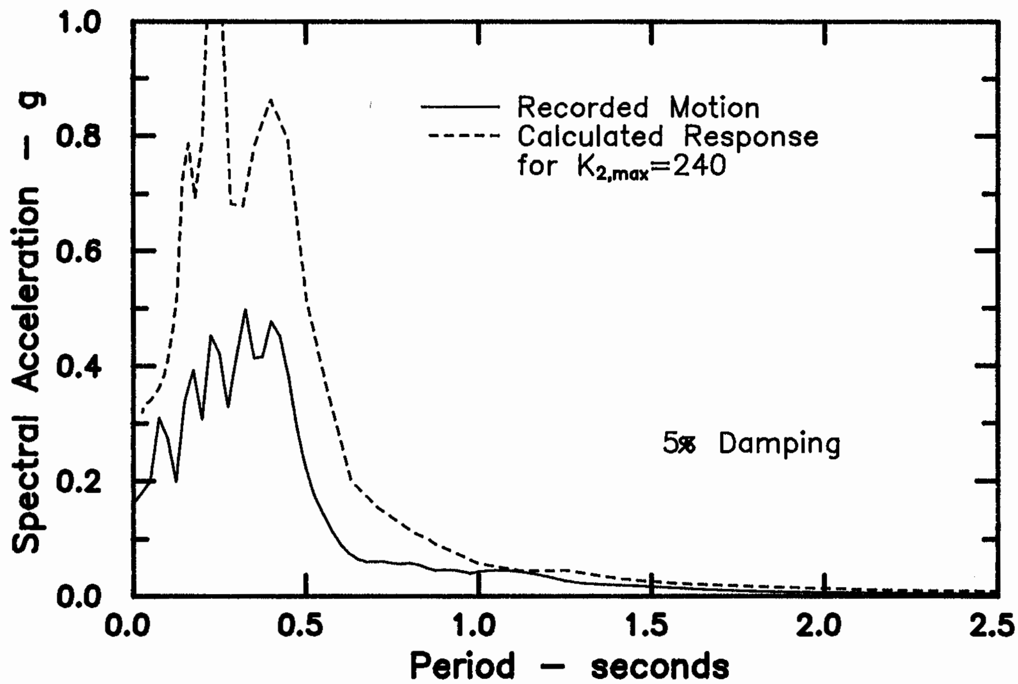


Figure 4-8: CALCULATED CREST RESPONSE FOR A $K_{2,max}$ OF 240

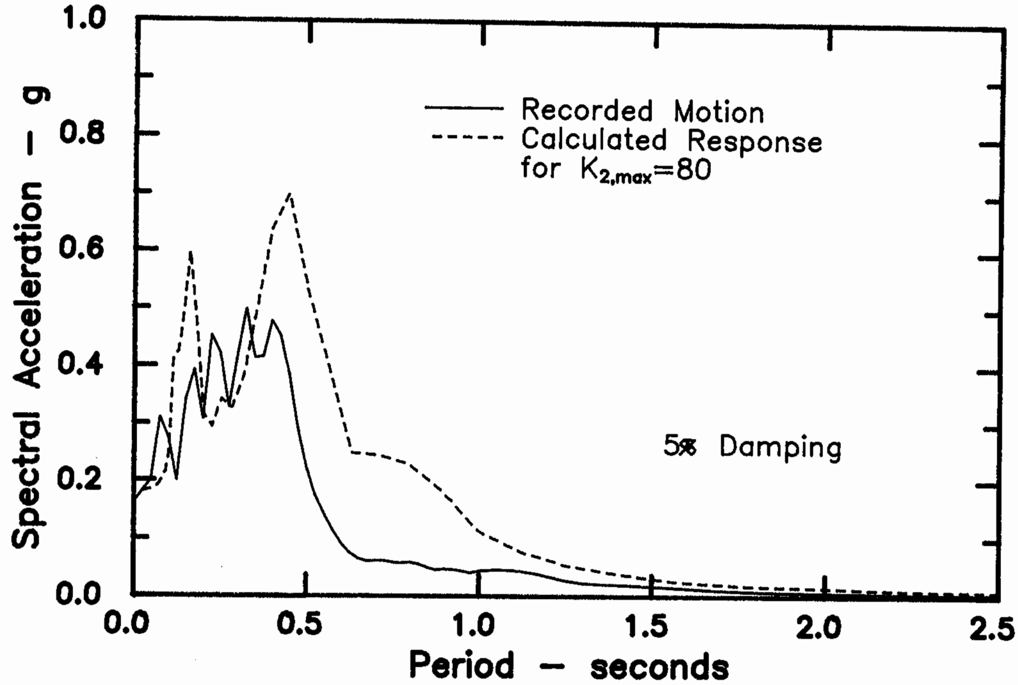


Figure 4-9: CALCULATED CREST RESPONSE FOR A $K_{2,max}$ OF 80

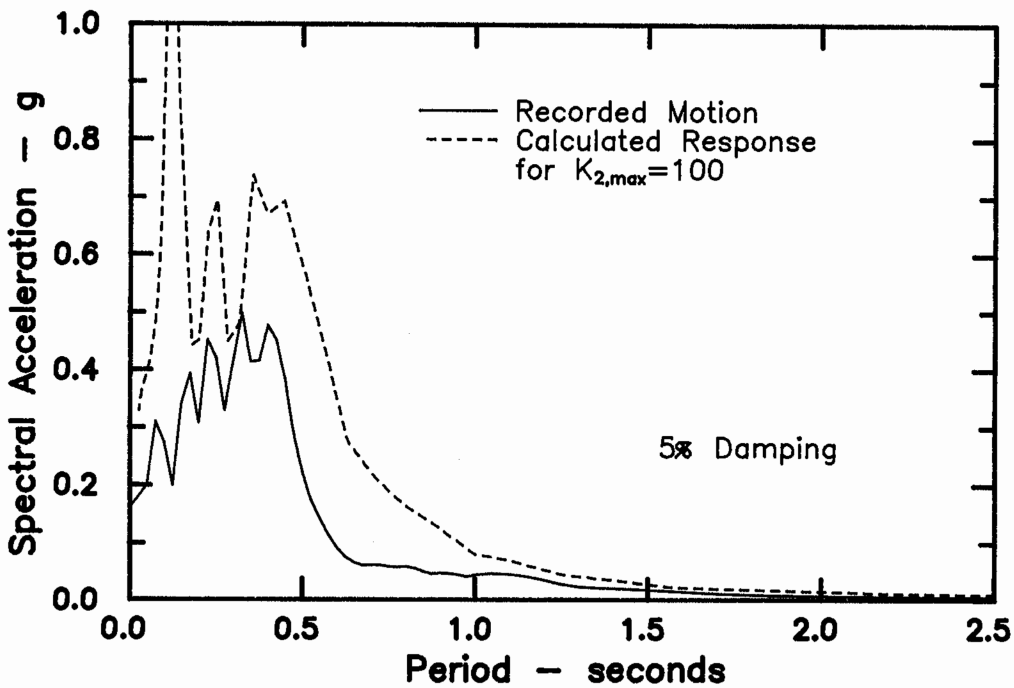


Figure 4-10: CALCULATED CREST RESPONSE FOR A $K_{2,max}$ OF 100

of the "softer" material to effectively pass frequencies of about 9 Hz or greater for the mesh used to model Cogswell Dam. Furthermore, the "softer" material results in lengthened predominant periods for the 2-D section (T_p = 0.75 sec for $K_{2,max}$ = 80) which reduces the response to the high frequency input motions because the dominant response is limited to the 2nd and higher modes of vibration of the dam. Thus, despite the apparent improvement in the match between computed and recorded response spectra, a $K_{2,max}$ value of 80 is believed to be much too low for a 2-D analysis of Cogswell Dam.

The recorded and calculated crest responses can also be compared on the basis of maximum horizontal acceleration, which is a response parameter often used in evaluation of permanent deformations. Table 4-2 lists the calculated maximum horizontal crest accelerations obtained in the 2-D analyses using the different values of $K_{2,max}$ previously listed. The calculated values are all significantly higher than the observed maximum horizontal crest acceleration of 0.151 g. Of particular interest is the calculated maximum crest acceleration for a $K_{2,max}$ value of 240, since this $K_{2,max}$ value produced a 2-D predominant period of 0.41 seconds which is quite close to the observed predominant period (discussed in the following section). It has been suggested (Vrymoed, 1981) that the use in a 2-D analysis of a $K_{2,max}$ value which produces a 2-D predominant period similar to the observed 3-D predominant period, would result in reasonable estimates of accelerations. However the use of $K_{2,max}$ equal to 240 did not result in an improved calculated response as had been hoped.

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. As previously mentioned, Lai and Seed (1985) found that significant amplifications may be present in motions recorded at rock outcrops above the crest of the dam. Similar

Table 4-2: Calculated Maximum Horizontal Crest Accelerations for Different Values of $K_{2,max}$

$K_{2,max}$	Maximum Surface Acceleration at the Crest: $a_{g,max}$
80	0.182
100	0.379
120	0.298
150	0.236
180	0.227
240	0.336
	0.151 recorded

observations were made by Bray et al. (1990). If the recorded motions from the right abutment do represent some amount of amplification over the dam's true foundation motions, this might explain (at least in part) why the calculated crest response is much higher than the recorded crest response. This possible explanation is feasible since the ratio of maximum recorded acceleration at the crest center to the maximum recorded acceleration at the right abutment is 2.3 (transverse motions) and this magnification ratio could instead have been as high as 4.0 and still have been consistent with observed magnification ratios in other dams (Lai, 1985) for similar levels of earthquake shaking.

A second possible reason for the differences between computed and recorded responses may be inherent limitations in the applicability of 2-D analyses to dams with crest length to maximum crest height ratios of only 2.1:1. However, analytical experience suggests that the inclusion of 3-D effects, using the same soil properties, would stiffen the dam bringing its predominant frequency closer to that of the input motions such that even higher crest responses might be calculated. Thus, it was decided that a 3-D analysis would not likely improve the match between recorded and calculated responses and subsequently was not warranted in light of the uncertainty regarding the use of the recorded abutment motions as input motions to any analyses.

A comparison of the calculated and recorded responses of Cogswell Dam to the 1987 Whittier Narrows Earthquake did not show good agreement. The calculated response was consistently stronger than the recorded response for a wide range of dynamic properties assigned to the rockfill. This lack of agreement might be attributable to differences between the dam's foundation motions and the recorded right abutment motions, and/or inherent difficulties with the 2-D modelling of a dam with a length to height ratio of only 2.1:1. Despite these limitations, the best overall match with respect to the shape and frequency content of the calculated and recorded responses was obtained using $K_{2,max}$ values of 150 to 180 for the dumped rockfill material comprising the bulk of Cogswell Dam.

Comparison Based on Predominant Period

A valuable opportunity to estimate the dynamic properties of loosely dumped rockfill has been provided by the recorded response of Cogswell Dam. An estimate of the $K_{2,max}$ value for the rockfill comprising the bulk of Cogswell Dam was obtained in the previous section by comparing recorded and calculated responses. In this section, the $K_{2,max}$ value will be estimated based on a comparison of recorded and calculated predominant periods for Cogswell Dam.

It is not appropriate to compare the recorded predominant period of Cogswell Dam with values calculated directly from a two-dimensional plane strain analysis, since the dam has a crest length to maximum crest height ratio of only 2.1:1. Mejia and Seed (1981, 1983) proposed a relationship between the predominant frequency of a fully 3-D dam in a V-shaped canyon versus an infinitely long dam with the same maximum crest section, as a function of the ratio of crest length (L) to dam height (H), as shown in Figure 4-11. 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) and Makdisi (1976). For Cogswell Dam, with a length to height ratio of 2.1:1, Figure 4-11 indicates that a plane section 2-D analysis using the true material properties should calculate a predominant period which is approximately 1.65 times greater than the recorded predominant period.

The recorded predominant period of Cogswell Dam was evaluated by calculating response spectra for the transverse motions recorded at the center of the dam's crest. The response spectra, as shown in Figure 3-6, for the full recorded

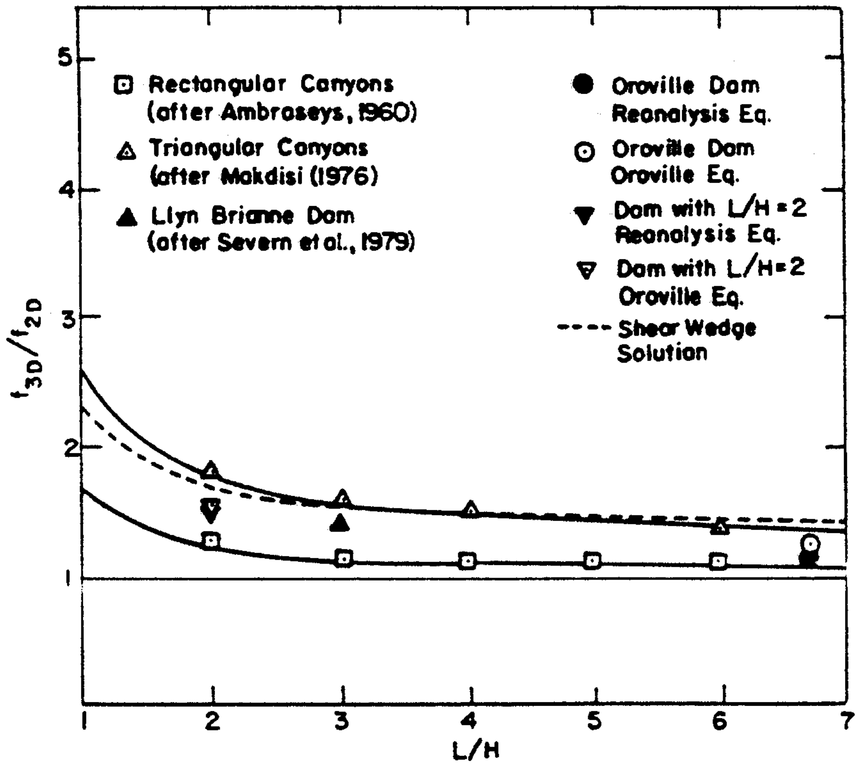


Figure 4-11: COMPARISON BETWEEN FULL THREE-DIMENSIONAL FREQUENCY AND IDEALIZED TWO-DIMENSIONAL FREQUENCY AS A FUNCTION OF CREST LENGTH vs. CREST HEIGHT (Mejia and Seed, 1983)

motion indicates a predominant period of 0.33 seconds. However, because of the broad spectral response with its multiple peaks, there was some question as to whether the peak at a period of 0.33 seconds represented interaction between the high frequency input motions and the dam's higher modes of vibration, in which case a slightly lower spectral peak might better represent the dam's predominant period. The issue of evaluating the predominant period is further complicated by the fact that during the course of earthquake loading, the level of induced shear strain is constantly changing which means that the dam has a constantly changing stiffness and predominant period.

To investigate how the predominant period of Cogswell dam may have varied during earthquake loading, the accelogram for the center crest transverse motions was divided into time "windows" which represented different levels of shaking. Response spectra were calculated for each time "window" and are shown in Figure 4-12. The time "window" of 5.0 to 7.0 seconds, representing a period of strong shaking, indicates a predominant period of approximately 0.40 seconds. The "window" of 7.5 to 10.0 seconds, representing the initial period of decay of strong shaking, indicates a predominant period of 0.37 seconds. Lastly, the "window" of 30 to 40 seconds, representing low level shaking, indicates a predominant period of 0.325 seconds. This observed variation in predominant period with level of shaking is in good agreement with expected behavior. For purposes of comparison with analytical results, a "representative" predominant period would correspond to a "representative" strain level of 65 percent of the maximum shear strain, which was the simplifying assumption made in the 2-D analytical procedures. Thus, the recorded transverse crest motions suggest that a predominant period of between 0.35 to 0.40 seconds is representative of the shear strain levels induced by the 1987 Whittier Narrows Earthquake.

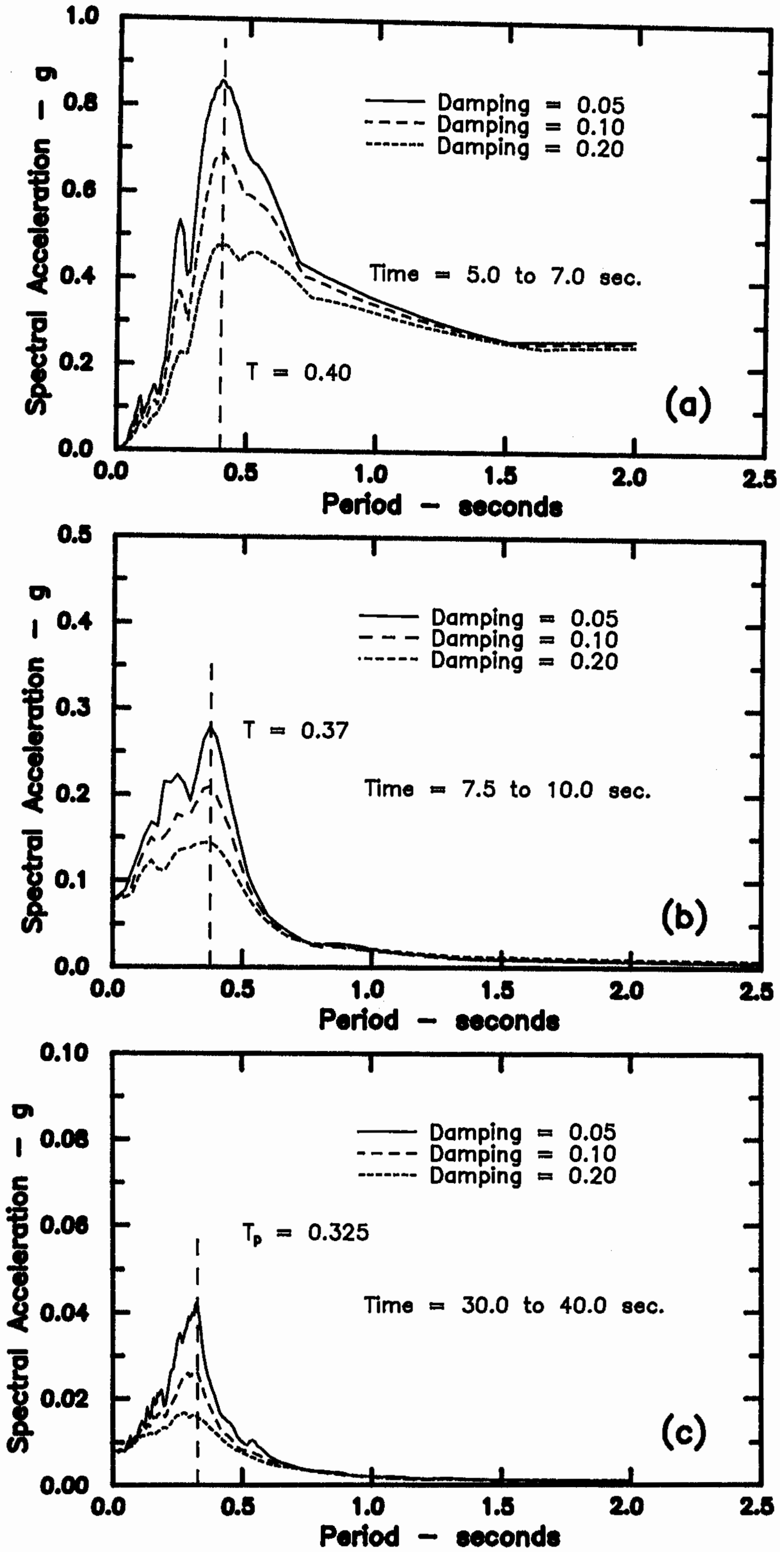


Figure 4-12: RESPONSE SPECTRA FROM RECORDED TRANSVERSE MOTIONS AT CENTER CREST

A 2-D plane section analysis of Cogswell Dam, using the true material properties, would then be expected to calculate a predominant period which is 1.65 times greater than the recorded predominant period of 0.35 to 0.40 seconds. Note that a 2-D analysis giving this desired 2-D predominant period does not imply that the calculated response should match the recorded response. Nonetheless, a 2-D analysis of the maximum cross-section of Cogswell Dam using the true material properties, should result in a calculated predominant period of about 0.58 to 0.66 seconds.

An estimate of $K_{2,max}$ for the loosely dumped rockfill in Cogswell Dam, was obtained by finding the $K_{2,max}$ value which produced the desired 2-D predominant period of 0.58 to 0.66 seconds. For each of the 2-D plane strain response analyses described earlier, with $K_{2,max}$ values ranging from 80 to 240, predominant periods were obtained from the fourier transform functions. Figure 4-13 shows how the predominant period varied with the value of $K_{2,max}$. As shown in this figure, a $K_{2,max}$ value of 100 to 125 would produce the appropriate predominant period of 0.58 to 0.66 seconds in the 2-D analysis.

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

$$Tp = 2.61 \times H/V_s \tag{2}$$

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. The representative G_{avg} was likely about 55 to 60 percent of G_{max}, based on the level of shear strain estimated to have been induced in the Cogswell embankment during the 1987 Whittier Narrows Earthquake. Subsequently, a $K_{2,max}$ value of about 80

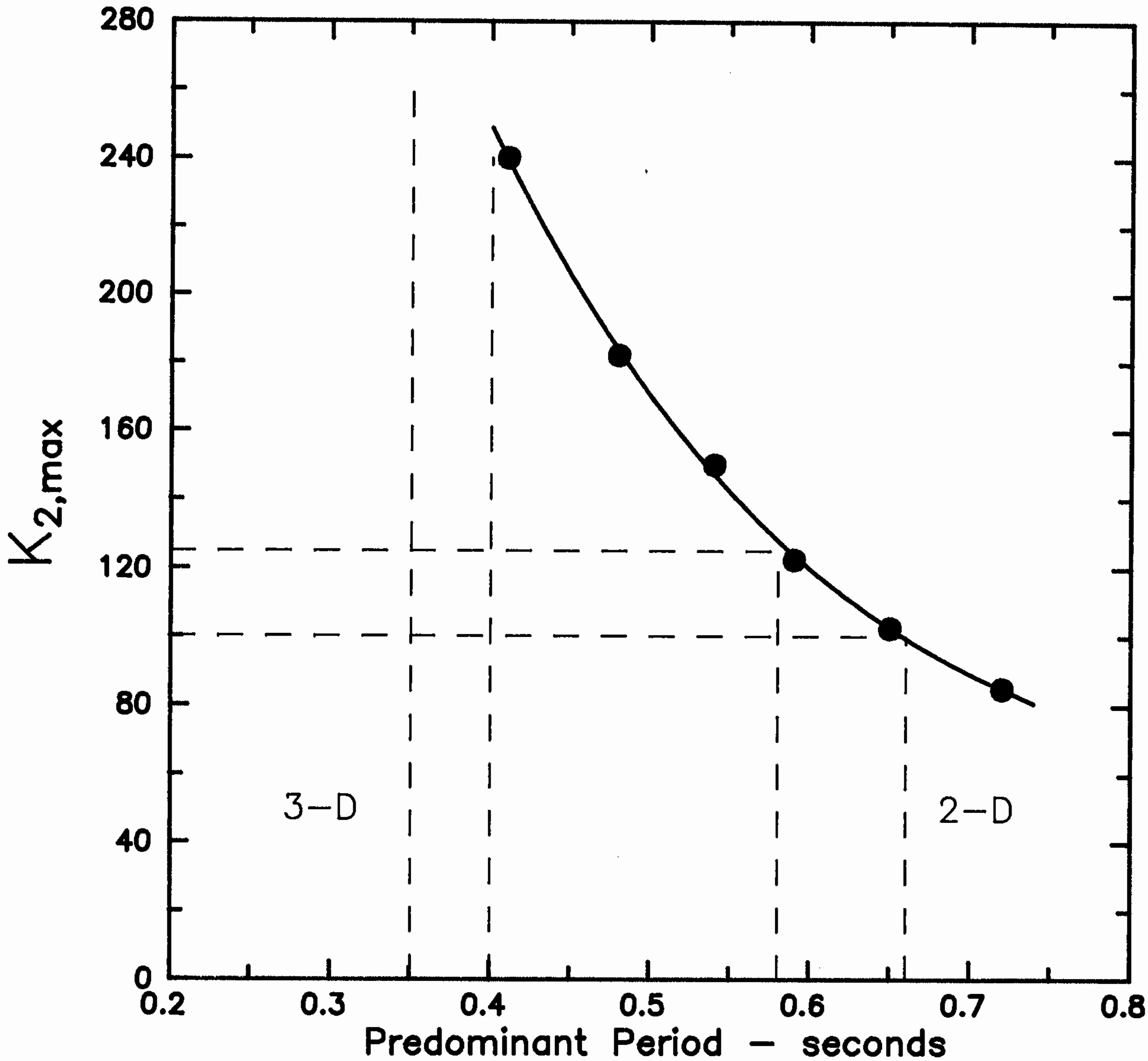


Figure 4-13: PREDOMINANT PERIOD OF 2-D MODEL VERSUS $K_{2,max}$

to 100 would result in an estimated predominant period of 0.58 to 0.66 seconds, in fairly good agreement with the results of the 2-D dynamic analyses.

Previous Investigations Into $K_{2,max}$ Values for Rockfill

Only a limited amount of data (obtained by means of back analyses similar to those of the present study) exist regarding appropriate $K_{2,max}$ values for dynamic analysis of rockfill dams. Based on similar studies of other rockfill embankments shaken by earthquakes, values of $K_{2,max}$ for loosely dumped rockfill have been suggested by Lai (1985) and Romo et al. (1980).

Before the $K_{2,max}$ values obtained in this study can be compared to values suggested by previous investigators, an allowance must be made for differences in the modulus degradation curves used. While this study used the shear modulus degradation curve for gravelly soils proposed by Seed et al. (1984), the above-mentioned previous investigators have used a modulus degradation curve for sands proposed by Seed and Idriss (1970), and Seed et al. (1984). It is the combination of $K_{2,max}$ and modulus degradation curve which, for a particular analysis, together result in an "equivalent linear modulus", and it is the "equivalent linear modulus" that determines the calculated response. This principal is demonstrated in Figure 4-14 where the calculated center crest response spectra for two different analyses of Cogswell Dam are presented: a $K_{2,max}$ of 180 with the modulus degradation curve for gravel; and a $K_{2,max}$ of 120 with the modulus degradation curve for sands proposed by Seed et al. (1984). The two calculated responses are essentially identical because for the average effective shear strain of about 1 x 10^{-2} percent induced in the embankment, the two different combinations of $K_{2,max}$ and modulus degradation curves resulted in the same "equivalent linear modulus". So in comparing the $K_{2,max}$ values of different investigations, consideration must be

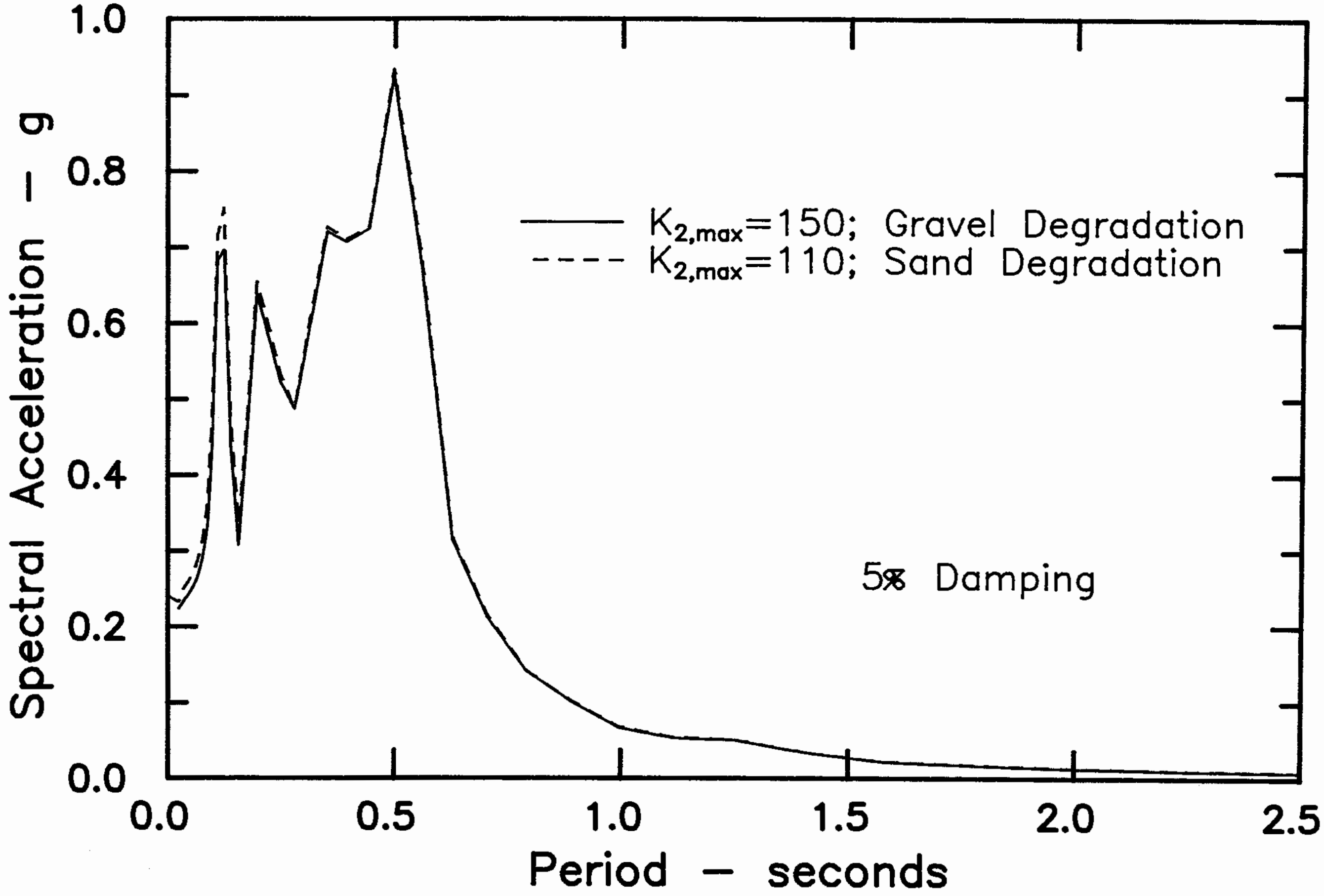


Figure 4-14: RESPONSE SPECTRA FOR EQUIVALENT COMBINATIONS OF $K_{2,max}$ VALUES AND MODULUS DEGRADATION CURVES

given to the modulus degradation curve used and the average induced shear strain level.

Back-analyses of the recorded motions at two similar rockfill dams form the basis for the $K_{2,max}$ values recommended by Lai (1985) and Romo et al. (1980). Of particular importance is the Infiernillo Dam which is comprised primarily of both dumped and compacted rockfill sections. The recommended $K_{2,max}$ value depends on whether 2-D or 3-D analysis procedures are used, with slightly higher $K_{2,max}$ values generally required in a 2-D analysis to compensate for the "softer" geometry (Lai, 1985). From analyses of Infiernillo Dam (Lai, 1985, and Romo et al., 1980) appropriate $K_{2,max}$ values for dumped rockfill are about 75 for a 3-D analysis and at least 100 for a 2-D analysis. Since the above $K_{2,max}$ values correspond to a modulus degradation curve for sands and an upper bound damping curve for sands, the $K_{2,max}$ values obtained in those 2-D and 3-D analyses must be increased by 20 to 40 percent (based on the range of induced shear strains calculated) for comparison with the results of this study which used a modulus degradation curve for gravels. Thus, the analyses of Infiernillo Dam suggest that for 3-D analyses of dumped rockfill dams using a modulus degradation curve for gravels, an appropriate $K_{2,max}$ value is about 90 to 105, in good agreement with the value of 100 to 125 obtained in this study (based on consideration of the predominant period). The 2-D analyses of Infiernillo dam suggest that for use with a gravel degradation curve, a lower bound value for $K_{2,max}$ of 120 to 140 is appropriate. Accordingly, despite the difficulties encountered in the present study with 2-D modelling of Cogswell Dam, the estimated $K_{2,max}$ values of 150 to 180 for use in 2-D analyses seem to be in reasonable agreement with Lai's observations for Infiernillo Dam.

Lai (1985) and Romo et al. (1980) also presented dynamic analyses of recorded motions at La Villita Dam, an earth and rockfill dam situated on a very deep alluvial deposit. Lai (1985) determined that for both 2-D and 3-D dynamic analysis procedures, the dynamic properties of the compacted rockfill comprising the outer shells of La Villita Dam were best modelled by a $K_{2,max}$ value of 130 with a modulus degradation curve for sands. Romo et al. (1980) suggested a $K_{2,max}$ value of 110 for the compacted rockfill in the outer shells, but this lower value may be due to the shallower depth of bedrock used in the finite element model by Romo et al. (1985) (Lai, 1985). If the modulus degradation curve for gravels proposed by Seed et al. (1984) had been used in these studies, then significant more modulus degradation would have been modelled at the computed strain levels, and thus the appropriate $K_{2,max}$ values would have been expected to be between 150 and 180. Since these values correspond to "compacted" rockfill, they appear to be in reasonable agreement with the values proposed for loosely dumped rockfill as obtained in the present study.

Modulus Degradation of Dumped Rockfill

The selection of a modulus degradation curve (as a function of cyclic shear strain) for dumped rockfill must be based largely on judgement considering the current lack of field information regarding this property. It has been fairly common practice to assume that a modulus degradation curve for sand is applicable and then concentrate on the other unknown dynamic property, $K_{2,max}$. While the need to assume a modulus degradation curve still remains, the results of this study suggest that the modulus degradation behavior of dumped rockfill may be better represented by the modulus degradation curve proposed for gravels by Seed et al. (1984) than by a modulus degradation curve for sands.

The amount of modulus degradation which occurred in Cogswell Dam during the 1987 Whittier Narrows Earthquake can be estimated based on the change in the dam's predominant period during the course of earthquake loading. As illustrated in Figure 4-12, the dam's predominant period varied from 0.40 seconds during the period of strongest shaking to 0.325 seconds during the last 10 seconds of light shaking. If the selected degradation curve and $K_{2,max}$ value are truly representative of the in-situ rockfill properties, then the selected properties should be able to model the change in predominant period for the different levels of shaking.

To investigate whether the rockfill's modulus degradation characteristics are better modelled by the curves for gravel or for sand (shown previously in Figure 4-2), the "comparison based on predominant period" was repeated for the last 10 seconds of light earthquake loading. The appropriate 2-D predominant period for the last 10 seconds of loading would be about 1.65 times 0.325 seconds which is 0.54 seconds. Using the modulus degradation curve for gravels, the desired 2-D predominant period was obtained using a $K_{2,max}$ value of 105 which agrees very well with the $K_{2,max}$ values of 100 to 125 obtained for the strongest levels of shaking. When a modulus degradation curve for sands was used, the desired 2-D predominant periods were obtained using $K_{2,max}$ values of 113 for the last 10 seconds of shaking and only 75 to 95 for the strongest levels of shaking, providing inconsistent results. Thus, it appears that, for the range of earthquake loading observed at Cogswell Dam during the 1987 Whittier Narrows Earthquake, rockfill modulus degradation as a function of cyclic shear strain can be better modelled by the use of the modulus degradation curve for gravel proposed by Seed et al. (1984) than by the relationships for sand proposed by Seed and Idriss (1970) and Seed et al. (1984).

Chapter 5
CONCLUSIONS

The observed response of Cogswell Dam during the 1987 Whittier Narrows Earthquake ($M_L \approx 5.9$) provided a rare opportunity to investigate the dynamic properties of loosely dumped rockfill and to evaluate the application of 2-D dynamic analysis procedures to a dam with a highly 3-D geometry. Cogswell Dam, a concrete faced dumped-rockfill dam with a height of 280 feet and a crest length of 585 feet (L/H = 2.1), experienced maximum crest accelerations of approximately 0.15 g with no apparent damage. The 2-D dynamic analyses of the response of this dam during the 1987 Whittier Narrows Earthquake, as presented in this report, resulted in the following conclusions:

(1) The 2-D dynamic response analyses consistently overestimated the observed response at the dam's crest, but this may be attributable at least in part to amplifications which might exist in the input motions which were recorded on the right abutment at an elevation above the crest of the dam (Lai, 1985; Lai and Seed, 1985; Bray et al., 1990), and/or to inherent difficulties in 2-D modelling of highly 3-D geometries. Despite these difficulties, the best overall match between observed response and the responses calculated by the 2-D analyses was obtained for $K_{2,max}$ values of 150 to 180 and with a modulus degradation relationship proposed for gravel by Seed et al. (1984).

(2) A $K_{2,max}$ value of 100 to 125 and the modulus degradation curve recommended for gravels by Seed et al. (1984) were found to be most representative of the true in-situ dynamic properties of the loosely dumped rockfill, and would thus be the values appropriate for use in

fully 3-D analyses. This recommendation is based on a comparison of predominant periods obtained from the recorded crest response and from the 2-D analyses, using a correction for geometric effects recommended by Mejia and Seed (1981, 1983). These $K_{2,max}$ values for loosely dumped rockfill are in good agreement with values recommended for 3-D analyses by previous investigators (Lai, 1985; Romo et al., 1980).

(3) The modulus degradation curve for loosely dumped rockfill must be assumed largely on the basis of judgement, due to a lack of data. However, the observed response of Cogswell Dam suggests that the rockfill modulus degradation can be modelled better by the modulus degradation curve recommend for gravels by Seed et al. (1984) than by a modulus degradation curve for sands.

(4) Future installations of strong motion instrumentation should reflect observations of possibly significant topographic amplifications in motions recorded on the abutments at elevations above the crests of dams. It would be useful to install additional "abutment" instruments at a lower position (e.g. downstream abutments) to provide comparative data for evaluation of this effect in future events.

(5) The absence of damage to Cogswell Dam as a result of the 1987 Whittier Narrows Earthquake is in accord with expected behavior at the observed levels of shaking, based on existing methods of analysis (Seed et al., 1985) for concrete faced rockfill dams.

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