ANALYSIS OF THE RECORDED RESPONSE OF LEXINGTON DAM DURING VARIOUS LEVELS OF GROUND SHAKING

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ABSTRACT

Lexington Dam, a 200-foot-high compacted earthfill embankment, was strongly shaken by the Loma Prieta 1989 earthquake (M_W 7) as well as by two smaller events (M_L 5) in June 1988 and August 1989.

The dam was instrumented as part of the California Strong Motion Instrumentation Program. The recordings at Lexington Dam due to at least three different levels of ground shaking provided valuable data for examining the nonlinear strain-dependent behavior of the embankment materials due to earthquake shaking. This paper summarizes the results of analyses of the recordings at the dam crest and abutment for two of the three earthquakes described above. The analyses included Fourier spectral ratio computations (crest to abutment) to examine changes in the fundamental natural period of the dam due to different levels of shaking; and one- and two-dimensional dynamic response analyses to evaluate the nonlinear strain-dependent behavior of the embankment materials at two levels of earthquake shaking.

LEXINGTON DAM

Lexington Dam is located on the east flank of the Santa Cruz Mountains, off Highway 17 between San Jose and Santa Cruz, about 17 miles north of Santa Cruz. The dam is 205 ft high and has a crest length of 810 ft. It is a zoned earthfill embankment with a downstream slope of 3:1 (H:V) and an upstream slope of 5½:1. A cross section of the embankment through the maximum section is shown in Figure 1. The embankment consists of four zones, upstream and downstream shells of gravelly clayey sands, a thick core of sandy gravelly clay, and an internal drain zone between the core and downstream shell. The downstream shell contains about 15 to 35% fines and the upstream shell about 20 - 65%, in both cases the fines are medium plasticity clays with LL ≈ 33-39 and PI ≈ 14-24. The core, below a depth of 80 ft, is composed of 85% fines of medium to high plasticity (LL ≈ 61-67 and PI ≈ 38-44), between the crest and a depth of 80 ft, the core material resembles more the upstream shell material with about 30 to 50% fines of medium plasticity (LL ≈ 31-39 and PI ≈ 14-18).

The foundation and abutments consist of bedrock of the Franciscan formation, which is composed chiefly of interbedded sandstone and shale, greenstone, and minor amounts of chert and schist. The topsoil at the foundation area was stripped prior to dam construction.

Recorded Ground Motions

The dam is instrumented with 3 sets of strong motion accelerographs, one set is located at a rock outcrop at the left abutment (west of the concrete spillway), and two sets are located on the crest to measure the response of the embankment. The locations of these instruments are shown on a layout of the dam presented in Figure 1. At each of the three locations the accelerographs were oriented in three orthogonal directions: transverse (normal to the dam axis), longitudinal (along the dam axis), and vertical. During the Loma Prieta earthquake (M_W 7) of October 17, 1989, peak accelerations (in the transverse direction) of 0.39 and 0.45 g were recorded at the left and right crest of the dam, respectively, and 0.45 g at the rock formation of the left abutment. The dam response was also recorded during two smaller magnitude ($M_L \sim 5$) earthquakes around Lake Elseman, one event on June 27, 1988, and the other on August 8, 1989. Peak accelerations recorded in the transverse direction during these events were significantly smaller than those of the Loma Prieta earthquake: 0.11 to 0.16 g on the crest and 0.03 g at the left abutment, for the event of June 27, 1988; 0.16 to 0.18 g on the crest and 0.08 g at the left abutment, for the earthquake of August 8, 1989. These data and earthquake information are summarized in Table 1.

TABLE 1

RECORDED PEAK ACCELERATIONS AT LEXINGTON DAM
DURING VARIOUS EARTHQUAKES

Earthquake (Date)	Magnitude M_L	Approximate Distance to Rupture Zone (km)	Peak Acceleration Values (g)		
			Left Abutment	Left Crest	Right Crest
Loma Prieta (Oct. 17, 1989)	6.9	6	E-W: 0.41 Up: 0.15 N-S: 0.45	0.40 0.22 0.39	0.34 0.20 0.45
Lake Elseman (Aug. 8, 1989)	5.2	18	E-W: 0.11 Up: 0.03 N-S: 0.08	0.17 0.08 0.18	0.22 0.10 0.16
Lake Elseman (June 27, 1988)	5.0	19	E-W: 0.04 Up: 0.02 N-S: 0.03	0.11 0.07 0.11	0.12 0.07 0.16

The recordings from these three earthquakes were digitized by the staff of the Strong Motion Instrumentation Program (Shakal et al., 1989). Upon examination of the records from the June 27, 1988 event it was noticed that the records appeared to be missing the early portion of ground motions and that the instruments may have triggered late during this event. Accordingly, this set of records was not used in the analyses and dynamic response computations described in the subsequent sections of this paper. Plots of the 5% damped response spectra of recordings from the two larger events are presented in Figure 2.

Analysis of Recorded Motions

Fourier spectral ratios of the recorded motions at the crest and the abutment were computed to examine any changes in the fundamental natural period of the embankment due to different levels of ground shaking. An increase in period with an increase in level of shaking reflects a decrease in the shear modulus of the embankment material (Chang et al., 1989).

A plot of the Fourier spectral ratios (crest to abutment) for both the smaller (M_L 5) event and the Loma Prieta (M_W 7) earthquake are shown on Figure 3. An examination of Figure 3 shows a substantial increase in the fundamental natural period of the embankment from about 0.5 seconds for the M_L 5 earthquake to about 1.4 seconds for the Loma Prieta event. It should be noted that the level of shaking represented by the peak ground acceleration at the left abutment rock for the smaller event was about 0.16 g compared to a value of about 0.45 g for the Loma Prieta event. The increase in the fundamental period of the embankment suggests a significant reduction in shear modulus of the embankment material due to the Loma Prieta earthquake.

Analysis of Embankment Response

To analyze the embankment response to the two levels of recorded ground motions, one- and two-dimensional dynamic finite element analyses were performed. The analyses employed the method of complex response and an equivalent linear approximation of the strain dependent modulus and damping properties. The program SHAKE (Schnabel et al., 1972) was used for the one-dimensional analyses, and FLUSH (Lysmer et al., 1975), a plane strain finite element analysis program was used for the two-dimensional analyses. In all analyses described herein, the transverse component (component normal to the dam axis) of ground motion was used.

The values of shear modulus at low strain (G_{max}) for various zones of the embankment were estimated on the basis of field shear wave velocity measurements (Wahler Associates, 1982). These values are summarized in Table 2 below:

TABLE 2

Zone	Shear Wave Velocity (ft/sec)
All zones at shallow depth (0-20 ft)	1200
Lower core (below 80 ft depth)	1100 - 1500
Upper core and upstream shell	1400
Downstream shell (20-50 ft depth)	1700
Downstream shell (below 50 ft depth)	2200
Foundation rock	1800 - 3000

The results of the geophysical field survey revealed shear wave velocities in the foundation bedrock that varied between 1800 and 3000 ft/sec with average values in the shallow foundation of about 2200 ft/sec. Theoretical studies of dams located in canyons with flexible rock foundations (Gazetas, 1991) showed that for cases where the ratio of canyon rock velocity to the dam velocity was less than 10, assuming a rigid rock foundation resulted in differences in the computed amplification function at the dam crest by more than 70%. Accordingly, it was considered more appropriate to model the upper 300 to 350 ft of rock foundation below the dam as a flexible rock foundation and include it in the finite element mesh. Figure 4 shows the two-dimensional finite element representation of Lexington Dam and its flexible rock foundation. Shear wave velocities in the rock foundation were specified as follows, 0 to 150 ft depth: v_s = 2500 ft/sec, 100 to 200 ft: v_s = 3000 ft/sec, and 200 to 350 ft depth: v_s = 4000 ft/sec. This distribution of rock shear wave velocities with depth was based on data from similar sites underlain by Franciscan rock formations where shear wave velocities have been measured with depth. The transverse component of the rock motion recorded at the left abutment outcrop was used as input to the finite element analysis and was specified as surface motion of a free field rock column. Horizontal transmitting boundaries were incorporated in the finite element idealization of the dam-foundation system. This assumption implies that rock motions at the abutment and at the valley floor are similar. One-dimensional wave propagation studies were made to verify this assumption as well as the use of results of two-dimensional finite element and finite difference studies of topographic effects on similar slopes at the Diablo Canyon site (Pacific Gas and Electric Co., 1988).

Relationships for variation of modulus and damping properties with strain were obtained from published literature on similar soils: Seed and Idriss (1970) and Seed et al. (1984) for cohesionless soils; and Sun et al. (1988) for cohesive soils. A number of parametric studies of the effects on the predicted response of using various modulus reduction and damping curves, different shear wave velocities, as well as the effects of assuming rigid rock foundation were made (Makdisi et al., 1991). For the results presented herein, the modulus reduction and damping curves used in the analyses are shown below in Table 3. These properties are based on the mid-range shear modulus reduction curve for Sands proposed by Seed and Idriss (1970) and the lower bound curve for damping proposed by the same authors.

TABLE 3.

Modulus Reduction and Damping Curves Used in Analyses

Shear Strain %	G/Gmax	Damping Ratio
10^{-4}	1.0	0.3
10^{-3}	0.97	0.8
10^{-2}	0.73	2.8
10^{-1}	0.30	10.0
1	0.05	21.0

The value of shear wave velocity for the lower core zone (Table 1) was specified at 1200 ft/sec.

Figure 5 shows the results of the two-dimensional finite element analyses for both events in terms of the computed and recorded 5% damped acceleration response spectra at the crest of the dam. There is reasonable agreement between the computed and recorded spectra for the Loma Prieta 1989 event at periods of 0.2 seconds and longer, although the peak acceleration is over-predicted by as much as 65%. For the smaller Lake Elseman event, although the analysis approximately predicts the first natural period of the embankment, the predicted spectral accelerations are about 60% of the recorded ones over the entire spectral range.

Figure 6 shows the computed Fourier amplitude transfer functions between the dam crest and left abutment recordings for both events analyzed. Again there is reasonable agreement between the computed transfer functions shown in Figure 6 and those obtained from the recorded motions shown in Figure 3. The computed first natural periods of the embankment using the finite element analysis are about 0.55 seconds for the smaller Lake Elseman event and about 1.3 seconds for the Loma Prieta event. The shift in the fundamental natural period of the embankment with the increase in level of ground shaking reflects the strain dependent nonlinear behavior of the embankment soils. This is shown in a plot of the G/Gmax and damping values vs. strain for the finite elements located at the centerline of the dam between the crest and the foundation and presented in Figure 7. This plot shows that for the Lake Elseman earthquake, the strain levels developed were in the range of about 0.004 - 0.03%, with corresponding reduction in Gmax of about 30% to 40%. In contrast with the Loma Prieta event, the developed strain level varied between about 0.1% and 1.0% with a corresponding reduction in shear modulus of about 60 to 95% of the initial low strain value.

SUMMARY AND CONCLUSIONS

The recorded response at Lexington during the Loma Prieta 1989 earthquake and two smaller events provided an opportunity to evaluate the nonlinear behavior of the embankment soils during various levels of ground shaking. Fourier spectral ratios of the recorded motions showed a definite shift in the fundamental natural period of the embankment with increased level of ground shaking. Two-dimensional finite element analyses of the embankment response using equivalent linear strain-dependent material properties provided predictions of embankment motions that are in reasonable agreement with the recorded motions.

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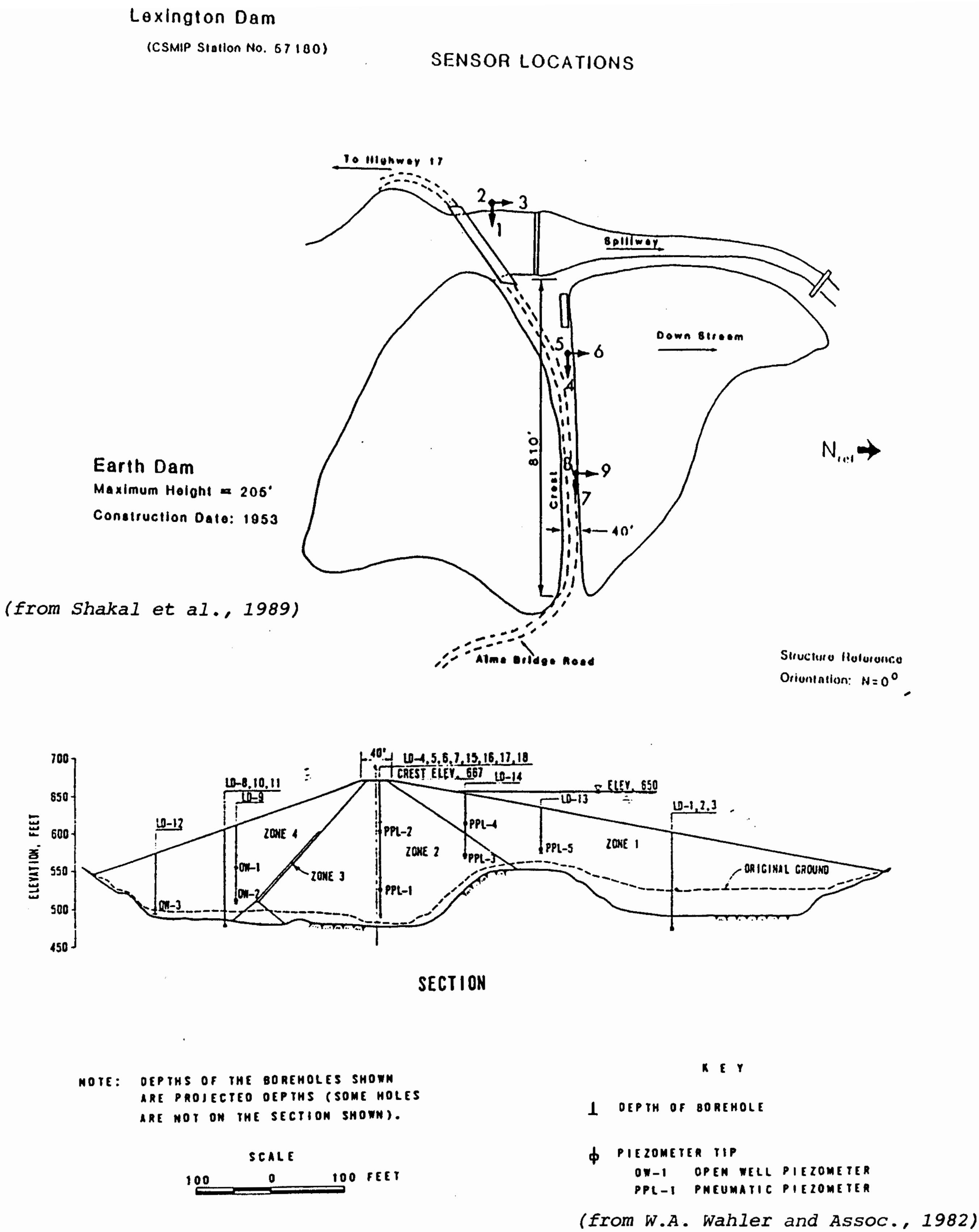

FIGURE 1. Plan of Lexington Dam showing locations of instruments and cross section showing various zones of embankment.

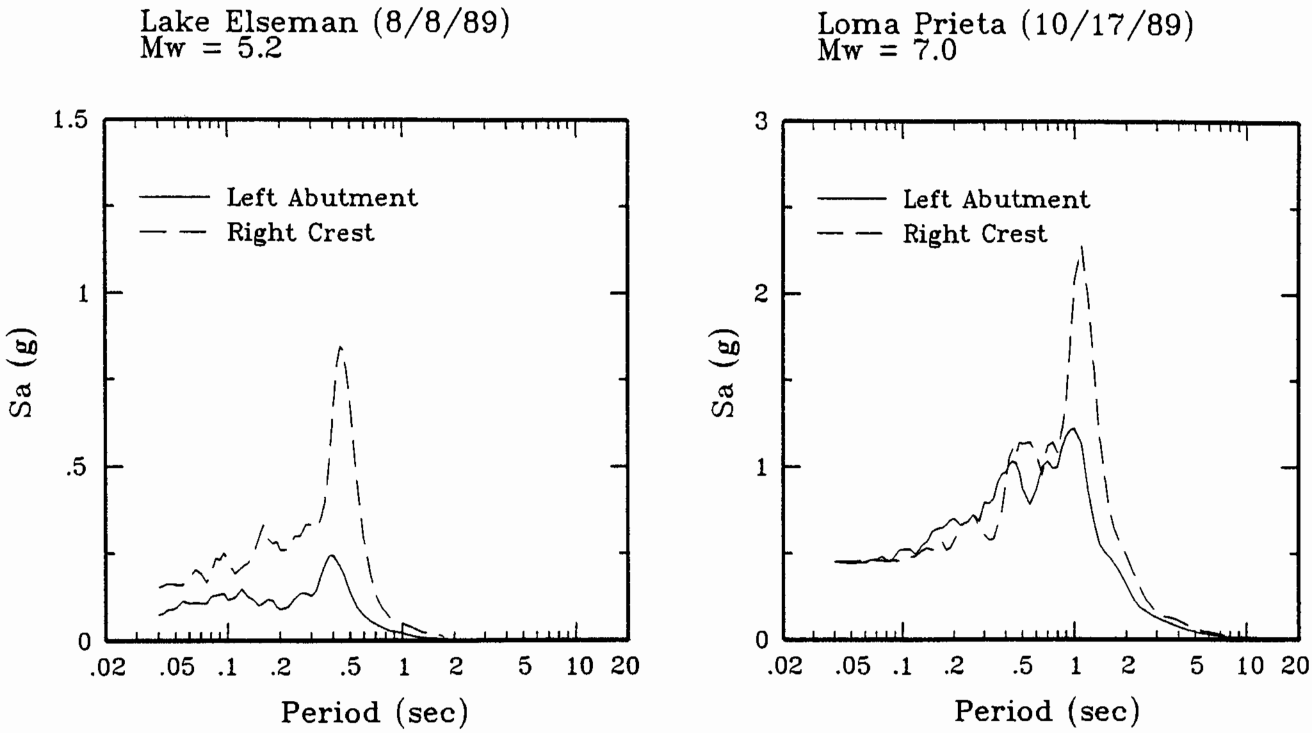

FIGURE 2. Response spectra of recorded motions at Lexington Dam for the transverse component (5% damping).

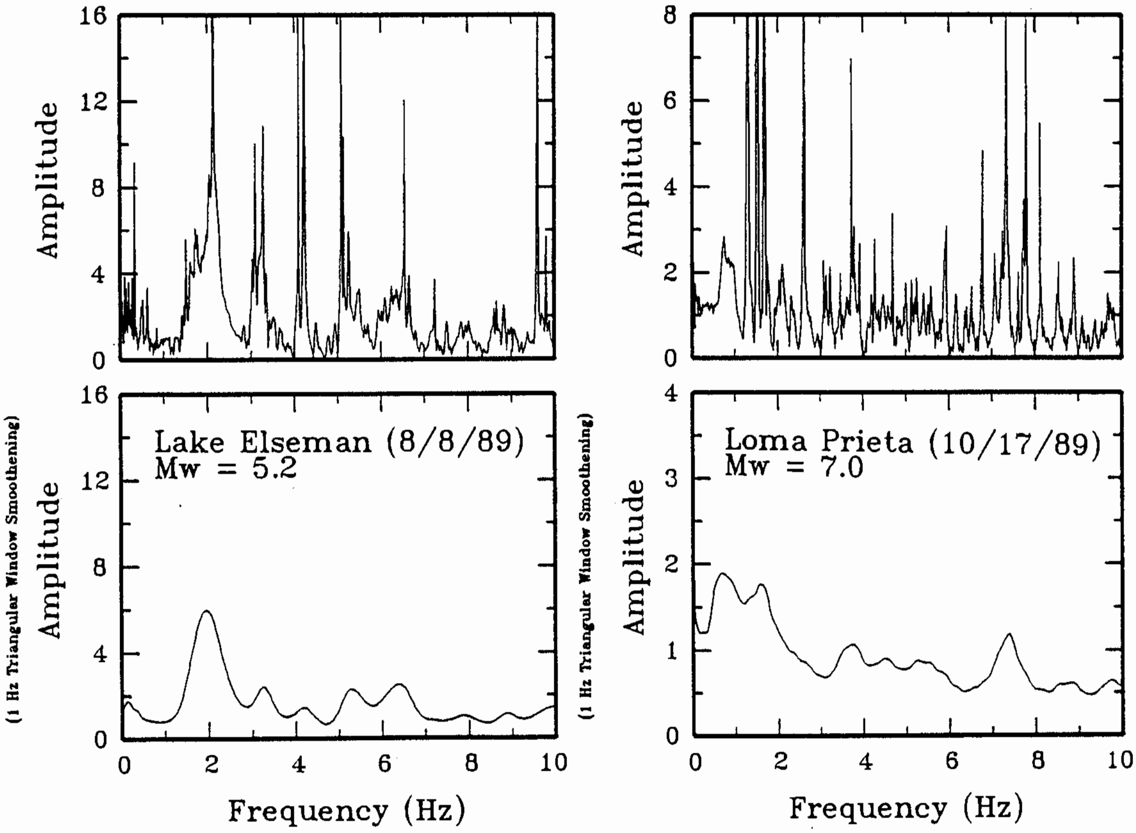

FIGURE 3. Fourier spectral ratios of crest to abutment recordings at Lexington Dam.

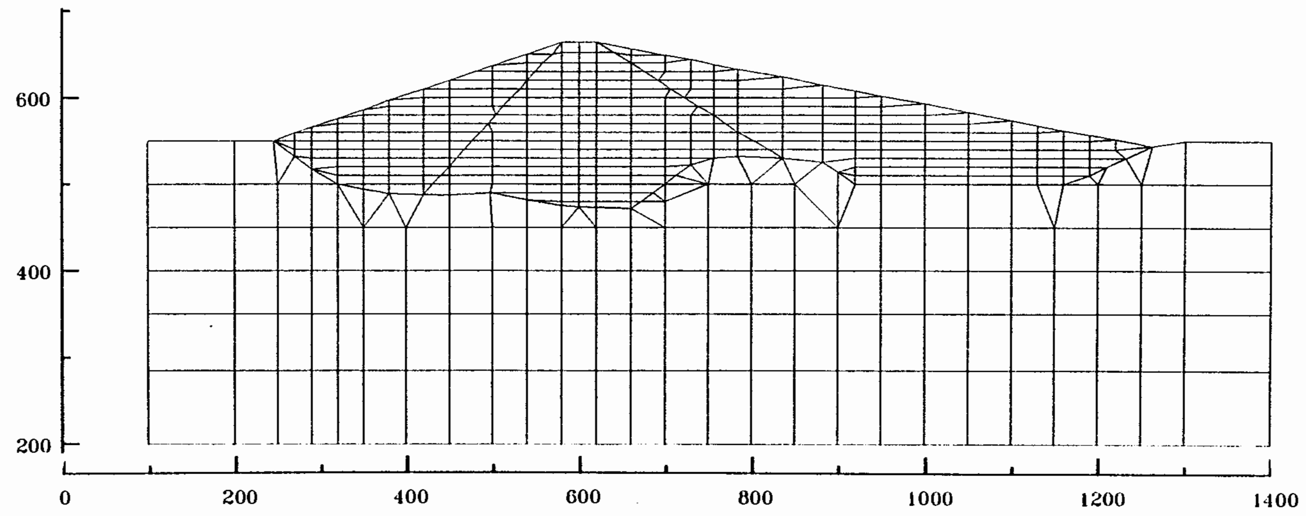

FIGURE 4. Finite element representation of Lexington Dam and its foundation.

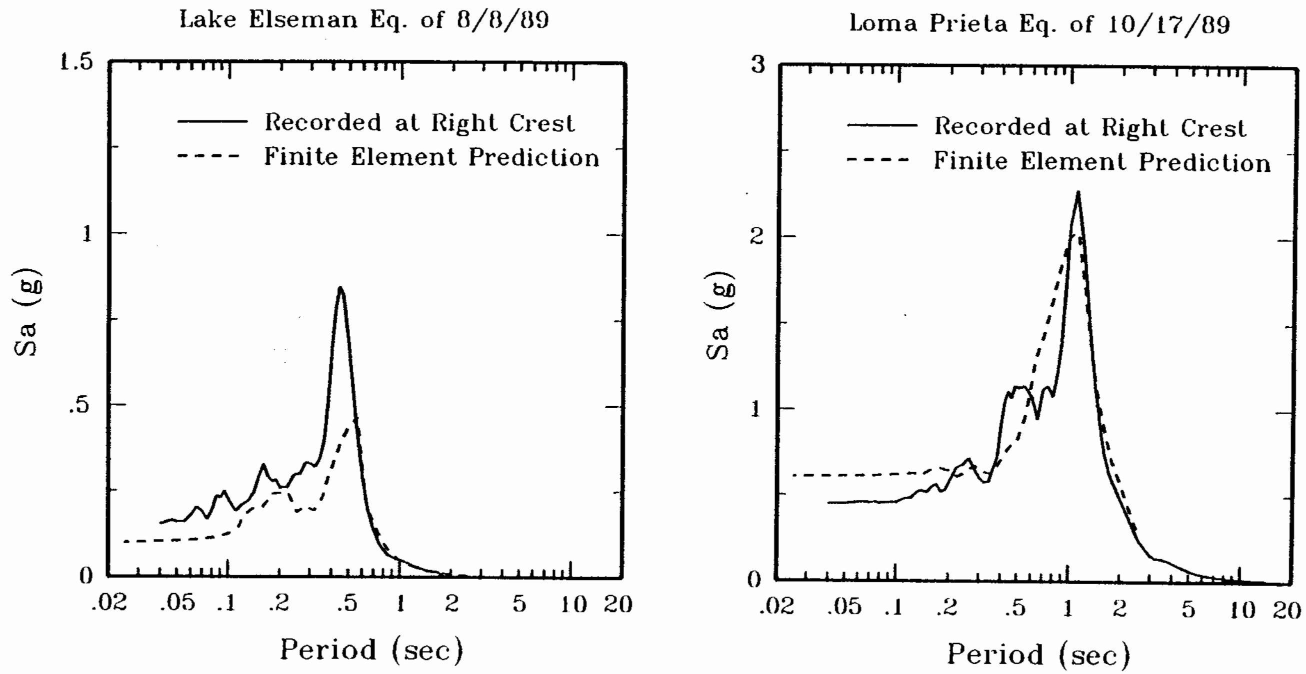

FIGURE 5. Comparison of computed and recorded response spectra at crest of Lexington Dam.

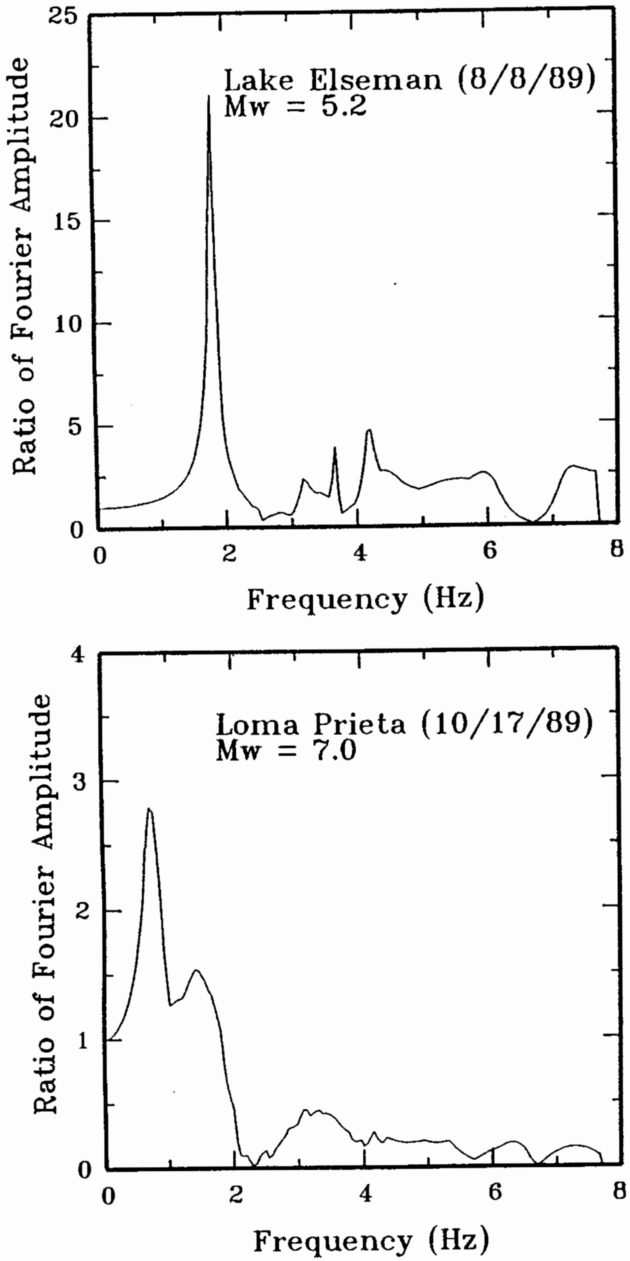

FIGURE 6. Computed Fourier amplitude transfer function (crest to abutment) at Lexington Dam.

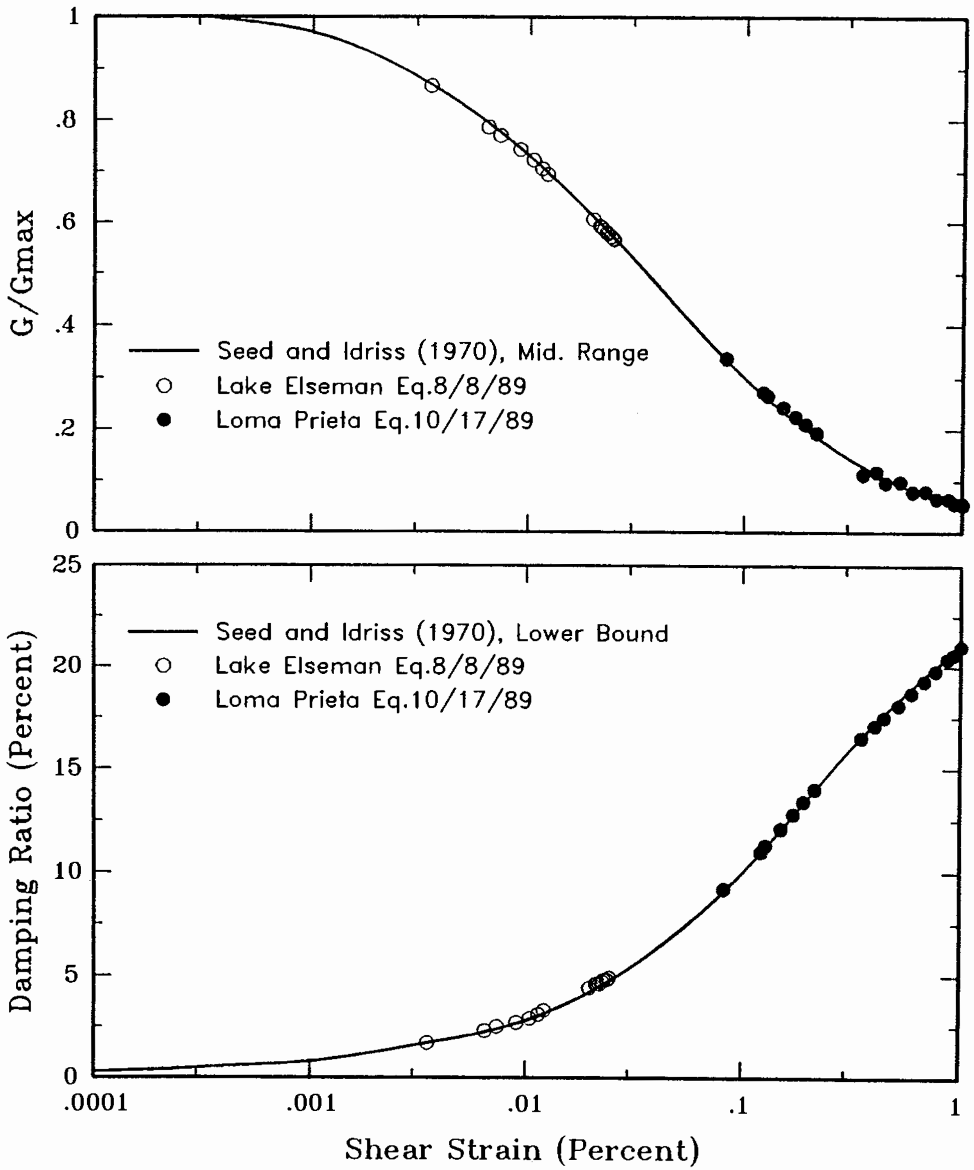

FIGURE 7. Variation of shear modulus and damping with strain for two events at Lexington Dam.