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SYNOPSIS: Stafford Dam, an approximately 79-ft high compacted earthfill founded on stream alluvium approximately 40 feet thick, was shaken by the 1989 Loma Prieta Earthquake. Records of the earthquake motions were obtained from seismographs located at the dam crest and at the right abutment on rock. Two-dimensional dynamic finite element analyses were performed to calibrate a model of the dam using the recorded motions. Excellent agreement between the recorded and calculated response was obtained by appropriate adjustments to material parameters based on shear wave velocity measurements. Various deconvolution methods for obtaining input bedrock motions to calculate the dam response are discussed.

INTRODUCTION

Stafford Dam is an approximately 79-foot-high compacted earthfill founded on stream alluvium approximately 40 feet thick. The dam is located on Novato Creek approximately 4 miles west of the City of Novato in northern Marin County, California. It is owned and operated by the North Marin Water District (District). Construction of the dam and related facilities was completed in 1951. A large portion of the downstream shell was removed and reconstructed in 1984 to replace the toe drain. The dam was subsequently raised 8 feet in 1985.

The dam was shaken by the October 17, 1989, Loma Prieta Earthquake (M_S = 7.1) which was centered approximately 80 miles south of the dam. Records of the earthquake motions were obtained by seismographs installed in 1978 at the dam crest and at the right abutment on rock, approximately 1/4 mile downstream of the dam crest.

These recorded motions provide an excellent opportunity to: (1) calibrate a dynamic analysis model of the dam by comparing the recorded response with the response predicted by the model, and (2) evaluate the effects of various deconvolution procedures for obtaining the input bedrock motions on the dynamic response of the dam.

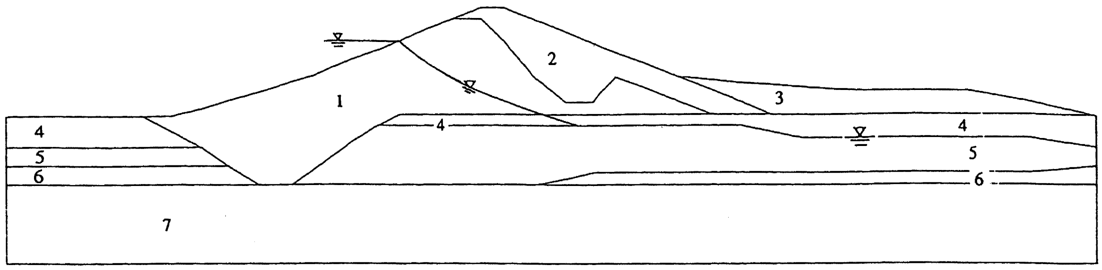
EMBANKMENT AND FOUNDATION MATERIALS

A significant amount of information and data on the embankment and foundation materials is available from previous engineering studies of the dam (Lee and Praszker 1978, 1986; Harlan Miller Tait 1985a, 1985b, 1986; Miller Pacific Engineering Group 1992). These data, which included cyclic triaxial laboratory test data, were reviewed to evaluate the dynamic properties of the materials. A representative idealized cross-section of the dam is shown in Figure 1(a). The embankment and foundation alluvium consist of four main soil types (refer to Figure 1a):

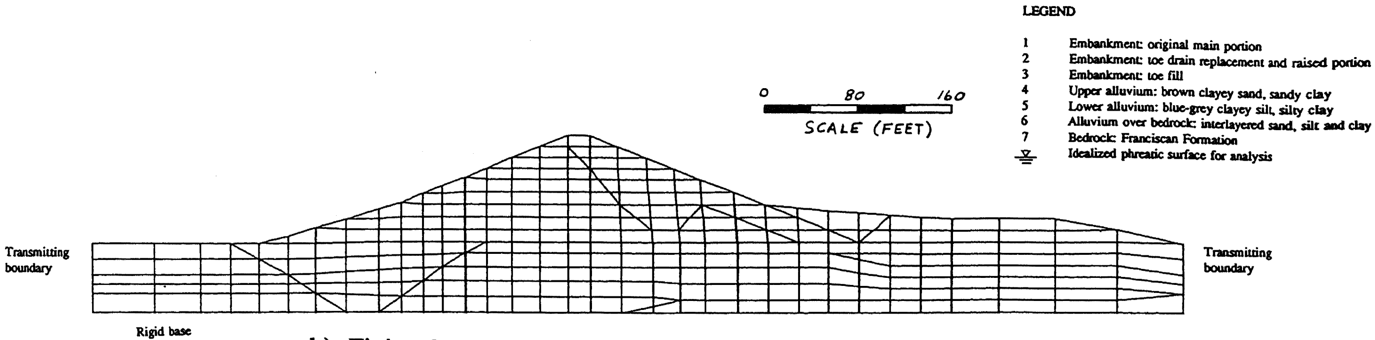
(1) A relatively homogeneous embankment of compacted fill, including old and new portions (zones 1, 2 and 3). The fill varies primarily between a clayey sand with gravel, a clayey gravel with sand, and a silty gravel with sand. The percent of materials passing the No. 200 sieve ranges from 8 to 55% (with an average of about 32%. Typical liquid limits are between 36 and 42, and the plasticity index is about 18.
(2) Brown sandy clay and clayey sands immediately underlying the dam (zone 4).
(3) Blue-grey clay, silty clay and clayey silt (zone 5; referred to as "blue clay") underly the brown sandy clay and clayey sand. This clay has between 70 and 98 percent passing the No. 200 sieve, an average liquid limit of about 39, and an average plasticity index of about 19.
(4) A discontinuous zone of interlayered sands, silts and clays (zone 6) directly overlying Franciscan bedrock (zone 7) and underlying the blue clay. These interlayered soils contain up to 33 percent gravel-size particles.

INSTRUMENTATION DATA

Instrumentation data for the dam includes piezometer data, drainage system discharge data, and survey monument data. Piezometer levels showed no measurable response to shaking during the Loma Prieta Earthquake. Discharge flow rates from the toe drain and abutment drain also showed no measurable response to shaking during the



a) Representative idealized cross section of dam for finite element analysis



b) Finite element mesh for dynamic analyses

Fig. 1. Representative Cross Section and Finite Element Mesh

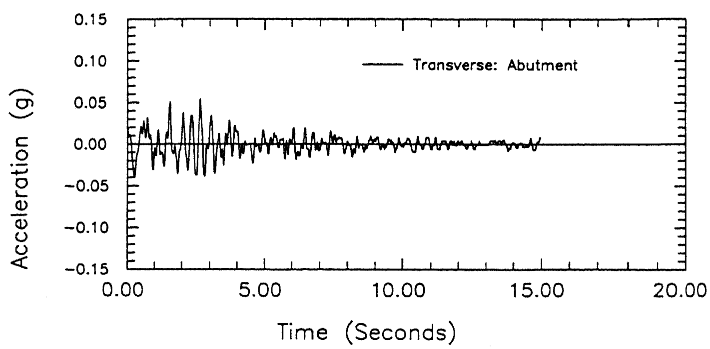
Loma Prieta Earthquake.

The monument survey data indicates progressive settlement and downstream movement of the dam crest since the dam was raised in 1985. The measured settlements and displacements are within the range expected due to raising of the dam, and movements caused by the earthquake appear to have been small.

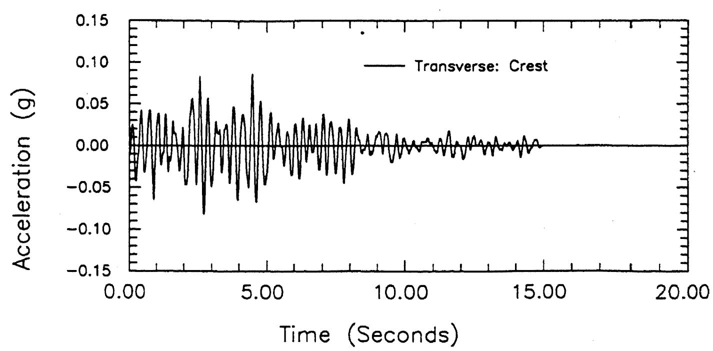
The analog film records for all components of ground motion (longitudinal, transverse, and vertical) recorded with both instruments at the site during the Loma Prieta Earthquake were digitized and baseline and instrument corrected (Agbabian Associates 1991). The corrected acceleration time histories for the motions transverse to the dam axis are presented in Figure 2, and the corresponding response spectra are presented in Figure 3. A summary of recorded motion parameters is given in Table 1.

Table 1. Recorded Loma Prieta Earthquake Motions

Location	Component	Peak Acceler-ation (g)	Maximum Spectral Acceler-ation (g)	Period at Maximum Spectral Acceler-ation (sec)
Crest	Longitudinal	0.040	0.170	0.320
	Vertical	0.020	--	--
	Transverse	0.086	0.479	0.320
Abutment	Longitudinal	0.054	0.243	0.300
	Vertical	0.017	--	--
	Transverse	0.039	0.164	0.257



a) Time history at abutment



b) Time history at dam crest

Fig. 2. Acceleration Time Histories Recorded Transverse to Dam Axis

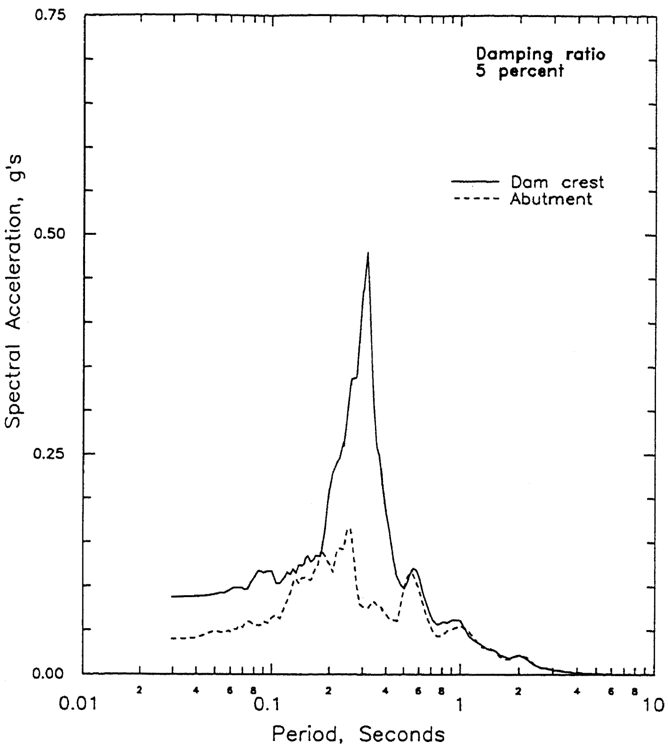


Fig. 3. Response Spectra for Motions Transverse to Dam Axis

FINITE ELEMENT CODE AND MESH

The dynamic finite element analyses were performed using the computer program FLUSH developed at the University of California at Berkeley (Lysmer et al. 1975). FLUSH is a two-dimensional finite element program for the dynamic response analysis of earth structures such as earth and rockfill dams, and the analysis of dynamic soil-structure interaction. The program uses the method of complex response to solve the equations of motion of a soil-structure system in the frequency domain. The nonlinear dynamic behavior of soils is modeled using the equivalent-linear method as proposed by Seed and Idriss (1970). The program allows the use of a finite element model with energy-transmitting lateral boundaries (Lysmer et al. 1975).

Figure 1(b) shows the finite element mesh developed for the analyses. The reservoir is not included in the finite element model since the effects of hydrodynamic forces on the response of the dam are likely to be small.

DYNAMIC PROPERTIES

Table 2 contains a summary of the unit weights, Poisson's ratios, damping ratio relationships, and shear modulus degradation relationships used in the analyses. Representative shear modulus degradation relationships were selected from the curves recommended by Sun et al. (1988) for clayey soils, by Seed et al. (1984) for sandy soils, and by Schnabel et al. (1972) for rock. Damping relationships were selected from the curves proposed by

Table 2. Dynamic Properties for Finite Element Analyses

Soil Type	Unit Weight γ (pcf)	Poisson's Ratio, ν Unsaturated	Poisson's Ratio, ν Saturated	Shear Modulus Degradation Relationship	Damping Ratio Relationship
Embankment	140	0.35	0.45	C1(1)	PI = 5(4)
Brown clayey sand/sandy clay alluvium	140	0.35	0.48	C1(1)	PI = 5(4)
Blue-grey clay, silty clay, and clayey silt	130	N/A	0.49	C2(1)	PI = 15(4)
Sand alluvium over bedrock	140	N/A	0.48	Sand(2)	Sand(2)
Bedrock	140	N/A	0.40	Rock(3)	Rock(3)

(1) From Sun et al. (1988)

(2) From Seed et al. (1984): degradation relationship depends on mean confining stress.

(3) Per Schanbel et al. (1972)

(4) From Vucetic and Dobry (1991) for indicated PI.

Vucetic and Dobry (1991) for clayey soils, by Seed et al. (1984) for sandy soils, and by Schnabel et al. (1972) for rock.

For the embankment materials (zones 1, 2 and 3 in Figure 1), the brown sandy clay and clayey sands (zone 4), and the interlayered soils (zone 6), the shear modulus, G_{max} (units of ksf), was assumed to depend on the mean effective stress, σ_m' (units of psf), as follows:

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

where σ_m' was taken as 65 percent of the vertical effective stress, and $K_{2,max}$ is a constant.

For the blue clay layer, the shear modulus was assumed to depend on the undrained shear strength, S_u, as follows:

$$\frac{G_{max}}{S_u} = \text{constant} \quad (2)$$

It was also assumed that:

$$\frac{S_u}{p'} = \text{constant} \quad (3)$$

where p´ is the vertical effectives stress. A lower limit was placed on the value of shear wave velocity, V_s, calculated from G_{max}.

In the analyses for the Loma Prieta earthquake motions, the maximum soil moduli, G_{max}, were varied until good agreement between the computed and recorded dam response was obtained. The "calibrated" soil moduli thereby compensate for approximations inherent in the finite element analyses. Shear wave velocity measurements along the centerline of the dam (Lee and Praszker, 1978) are compared in Table 3 with the values of V_s used in several of the analyses. In this comparison the latter have been corrected to correspond to the

Table 3. Comparison Between Model and Measured Shear Wave Velocities

Analysis Number	Material Type	Depth Interval (ft)	Measured Shear Wave Velocity, V_S (ft/s)	Model Shear Wave Velocity, V_S, (ft/s)
1,2,5	Original Main Embankment(1)	20-50	1200	1020-1200
		50-60	1700	1200-1240
	Brown sandy clay, clayey sand	60-70	1200	1180-1220
	"Blue Clay"	80-90	800	760-790
9	Original Main Embankment(1)	20-50	1200	1380-1620
		50-60	1700	1620-1680
	Brown sandy clay, clayey sand	60-70	1200	1300-1340
	"Blue Clay"	80-90	800	830-870
10,11,12	Original Main Embankment(1)	20-50	1200	1600-1880
		50-60	1700	1880-1950
	Brown sandy clay, clayey sand	60-70	1200	1450-1500
	"Blue Clay"	80-90	800	930-970

(1) The model parameters defining maximum shear modulus were increased 10 percent for materials within the toe drain replacement and raised portions of the dam, and were decreased 10 percent for materials within the toe fill portion of the dam.

effective stresses in the dam at the time of the measurements. Shear wave velocity measurements are not available for the zone of interlayered sand, silt, and clay overlying bedrock (zone 6) because this zone was not encountered by the borehole in which shear wave velocities were measured.

The Franciscan bedrock was assigned a uniform shear wave velocity of 2,500 feet/second (fps) for all analyses. This value was based on the available site data and a review of published values for similar geologic formations.

RESPONSE ANALYSES FOR LOMA PRIETA EARTHQUAKE

The dynamic response of the dam to the Loma Prieta Earthquake was computed using the abutment record as a rock outcrop input motion. A maximum frequency of 12 Hz was used in the analyses.

The motion on a bedrock formation beneath a soil deposit can differ significantly from the motions at a nearby outcrop of the same formation. The difference will depend on: (1) the impedance ratio between the deposit and the rock; (2) the damping in the deposit; and (3) the frequency content of the rock motion and the resonance frequencies of the deposit. In this study the difference between bedrock and outcrop motions was accounted for by deconvolving the recorded outcrop motion before inputting the motion at the base of the finite element model. Deconvolution is the process of calculating by wave propagation theory the motion at depth below a free surface, given the motion at the free surface.

The input motions at the base of the finite element model were calculated using three different approaches to account for bedrock compliance: (1) deconvolution to the base of the model using a soil column at the midpoint of the downstream face of the dam; (2) deconvolution to the base of the model using a soil column at the upstream toe of the dam; and (3) no deconvolution.

Deconvolution through the soil columns was performed using the program SHAKE (Schnabel et al. 1972). Because the base of the finite element model is assumed to be rigid while bedrock at the site is estimated to have a shear wave velocity of about 2500 fps, the deconvolution process consisted of two steps: (1) inputting the recorded motions as outcrop motions and propagating them to the top of the soil column overlying a half-space with a shear wave velocity of 2500 fps, and (2) deconvolving the motions calculated at the top of the soil column to a rigid base at the bottom of the column.

The computed response of the dam was compared to the recorded response in terms of acceleration response spectra, Fourier amplification ratios, and acceleration time histories at the dam crest. Model parameters were varied to obtain the best possible match in response. Shear wave velocity was found to be the dominating variable provided that other parameters were kept within a realistic range. Sensitivity analyses were performed in which Poisson's ratios, unit weights, modulus degradation relationships and damping relationships were varied. The results of selected analyses are summarized in Table 4, and the corresponding material parameters are summarized in Tables 2, 3, and 4.

Model Calibration

Analyses Nos. 1, 2, and 5 in Table 4 were performed using model parameters and soil properties developed based on engineering studies of the dam before the Loma Prieta earthquake. For analysis No. 5, the calculated acceleration time history at the dam crest is presented in Figure 4, the calculated acceleration response spectrum is compared with the recorded spectrum at the crest in Figure 5, and the calculated and recorded Fourier amplification ratios between the crest and abutment motions are presented in Figure 6. It may be seen that there is relatively poor agreement between the calculated and recorded response and that the finite element model appears to be "softer" than the dam.

Computed crest acceleration time histories, crest acceleration response spectra, and Fourier amplification ratios for analyses numbers 9 and 10 are presented in Figures 7 through 12. Both analyses showed good agreement with the recorded response. Although analysis

Table 4. Dynamic Analysis Results for Loma Prieta Earthquake Motions

Analysis Number	Shear Wave Velocities (ft/s) Min V_s - Max V_s (1): Dam	Brown Sand/Clay	Blue Clay	Sand Over Bedrock	Bedrock	Maximum Acceleration (g): Abutment	Rigid Base	Crest	Predominant Period(2) (sec)	Location for Deconvolution of Abutment Motion	Match Between Calculated and Recorded Response Spectra
1	440-1380	610-1340	500-970	830-1160	2500	.039	.039	.13	.46	None	Poor
2	440-1380	610-1340	500-970	830-1160	2500	.039	.034	.093	.44	U/S Toe	Poor
5	440-1380	610-1340	500-970	830-1160	2500	.039	.034	.083	.44	D/S Face	Poor
9	590-1860	660-1460	700-1060	990-1380	2500	.039	.033	.092	.37	D/S Face	Good
10	690-2160	740-1640	800-1180	1040-1450	2500	.039	.032	.084	.33	D/S Face	Good
11	690-2160	740-1640	800-1180	1040-1450	2500	.039	.031	.11	.33	U/S Toe	Fair
12	690-2160	740-1640	800-1180	1040-1450	2500	.039	.039	.15	.34	None	Poor
Recorded	--	--	--	--	--	.039	--	.086	.32	--	--

(1)Shear wave velocities given for 10^{-4} percent strain. Maximum and minimum values refer to the range of values used throughout finite element model.
(2)Period for maximum amplification at dam crest from amplification ratio between crest and abutment.

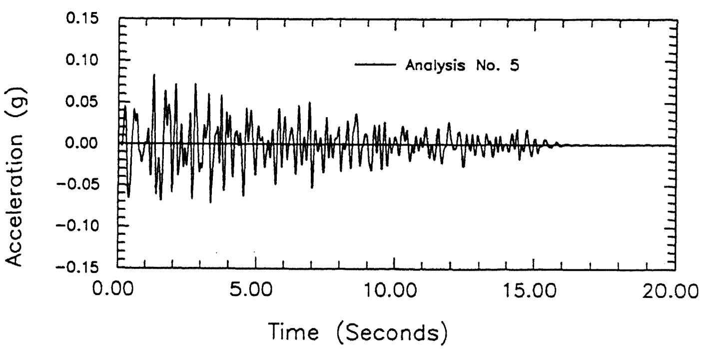


Fig. 4. Computed Acceleration Time History at Dam Crest - Analysis No. 5

number 10 produced a slightly better match than analysis number 9, it is likely that the dam material properties are somewhere in between the range defined by these two analyses since the analyses do not explicitly account for three-dimensional effects. Thus, it may be concluded that the two analyses "bracket" the range of soil model properties which best simulate the recorded response of the dam. As shown in Table 3 the soil properties are in reasonable agreement with the in situ shear wave velocity measurements.

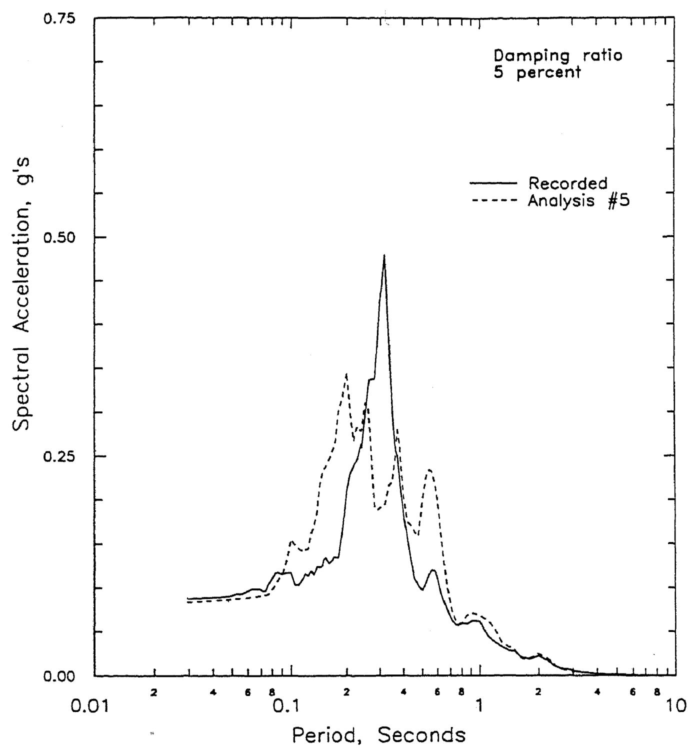


Fig. 5. Response Spectra at Dam Crest - Analysis No. 5

Effect of Deconvolution Method

The effect of the deconvolution method on the calculated response was investigated in analyses numbers 1, 2, and 5 for the set of model parameters leading to poor agreement with recorded response, and in 10, 11, and 12 for the calibrated model. Figure 13 shows the calculated response spectra for analyses 10, 11, and 12, all with the same model properties but using the three different deconvolution approaches previously described. It is clear that deconvolution using a soil column at the downstream dam face provides the best agreement with the recorded response, while the other two methods result in an over-estimation of the peak acceleration and spectral response at the dam crest.

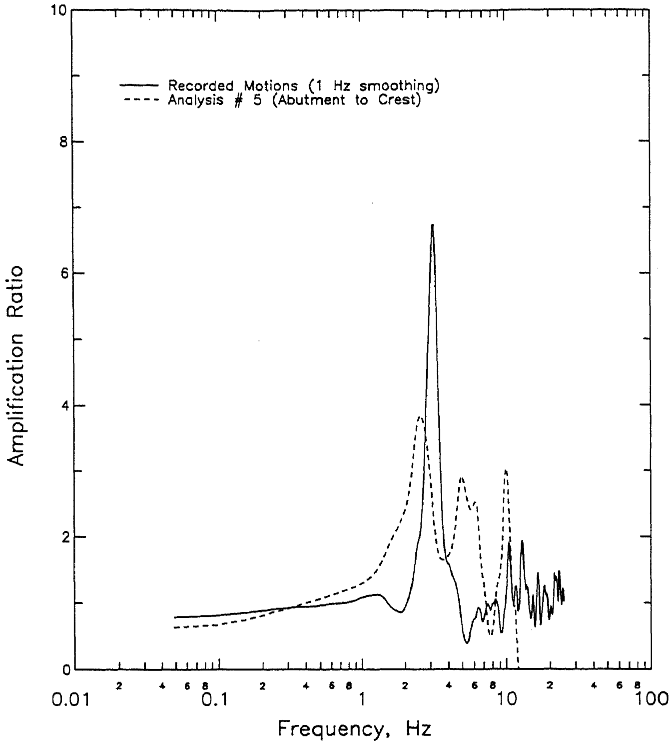


Fig. 6. Amplification Ratio to Dam Crest - Analysis No. 5

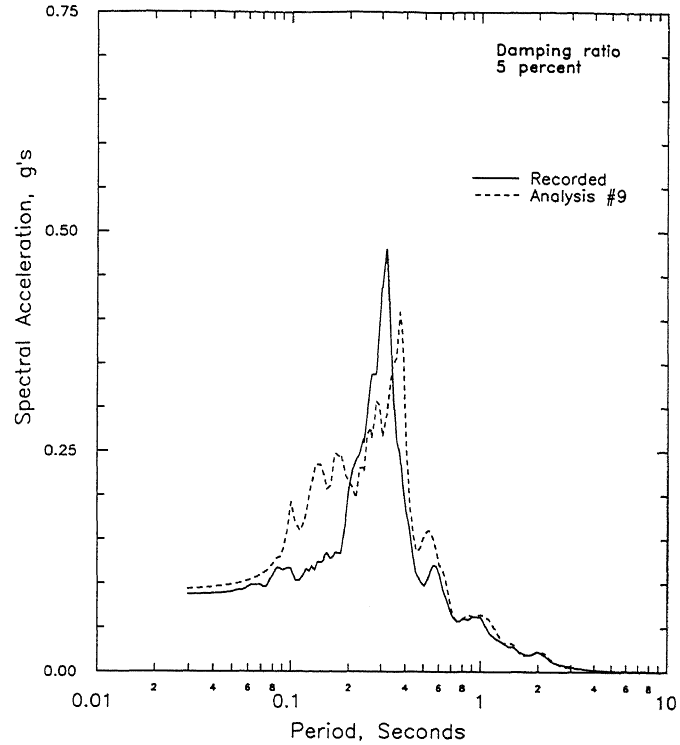


Fig. 8. Response Spectra at Dam Crest - Analysis No. 9

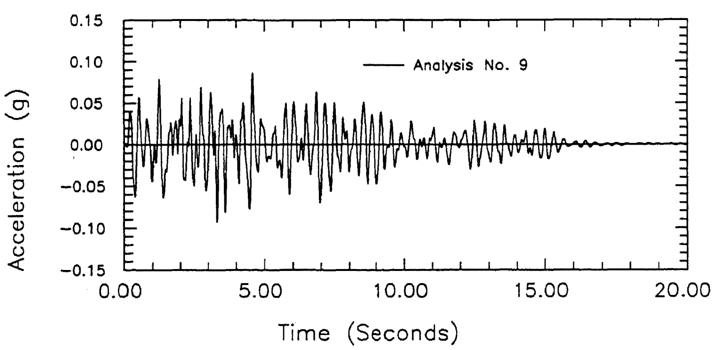


Fig. 7. Computed Acceleration Time History at Dam Crest - Analysis No. 9

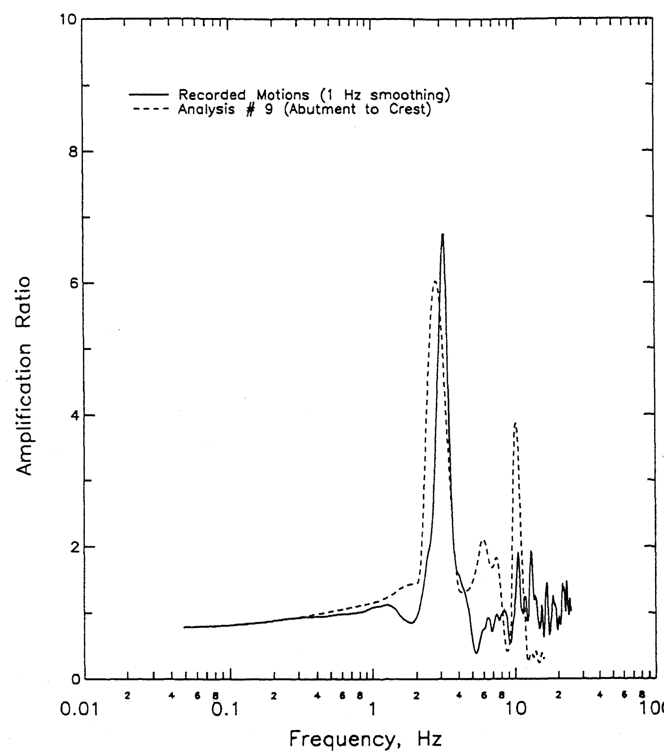


Fig. 9. Amplification Ratio to Dam Crest - Analysis No. 9

CONCLUSIONS

Stafford Dam performed satisfactorily during the 1989 Loma Prieta Earthquake. Peak ground accelerations transverse to the dam axis of 0.039 g and 0.086 g were recorded at the abutment and dam crest, respectively. Survey monument and piezometric data show no measurable effects of the earthquake on the dam. Good quality accelerograph records were obtained at the abutment and dam crest.

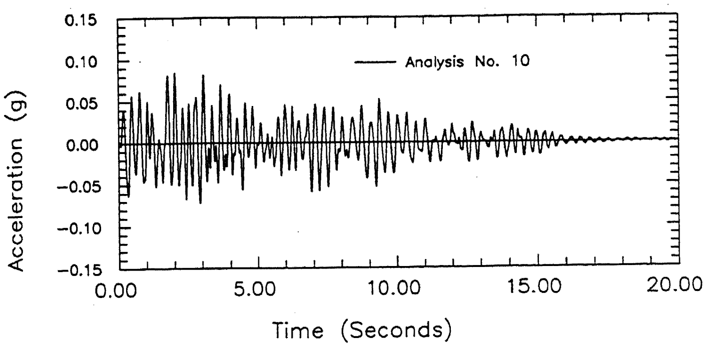


Fig. 10. Computed Acceleration Time History at Dam Crest - Analysis No. 10

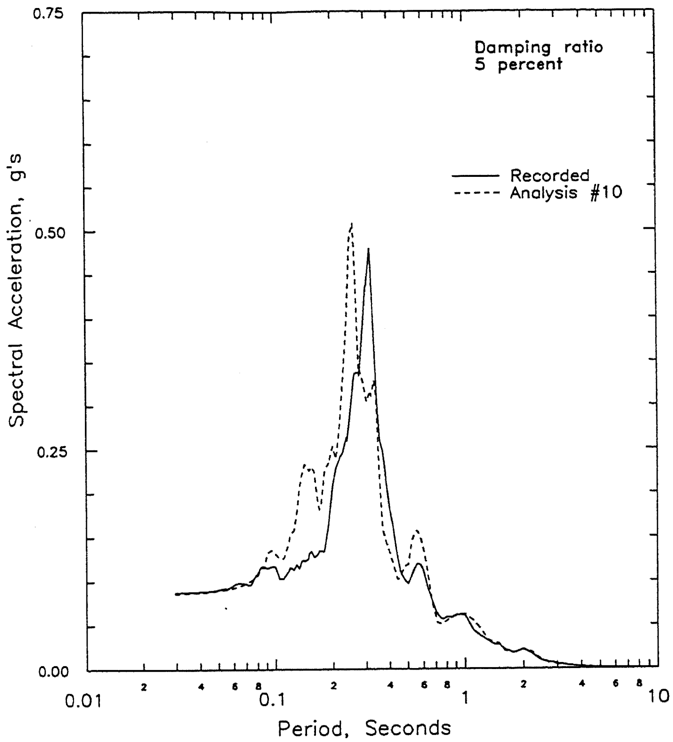


Fig. 11. Response Spectra at Dam Crest - Analysis No. 10

Two-dimensional dynamic finite element analyses were performed using the Loma Prieta Earthquake records to calibrate the soil properties in a numerical model of the dam. Good agreement was obtained between computed and recorded response using soil properties significantly stiffer than those used in previous engineering studies that did not have the benefit of model calibration with recorded motions. Model calibration with the recorded Loma Prieta Earthquake motions had a significant effect on the calculated response to the stronger design earthquake motions and thus proved to be a valuable design tool.

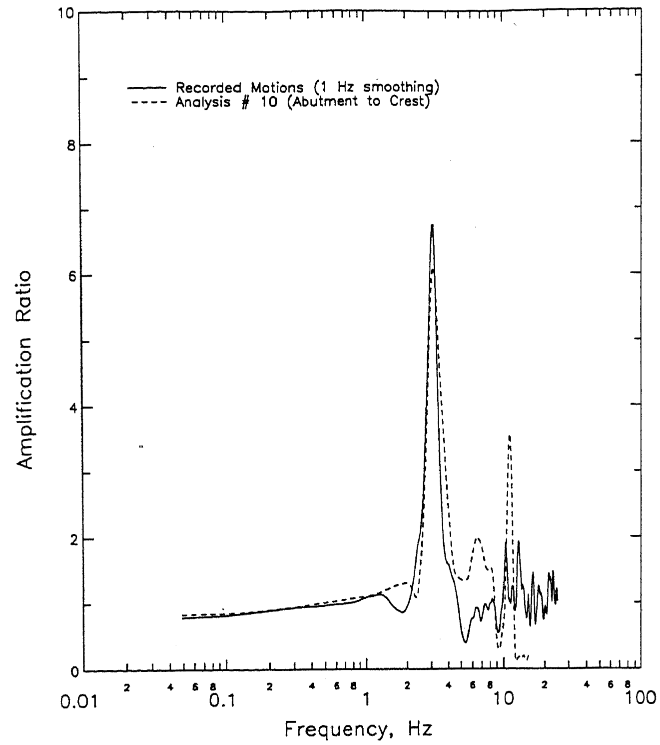


Fig. 12. Amplification Ratio to Dam Crest - Analysis No. 10

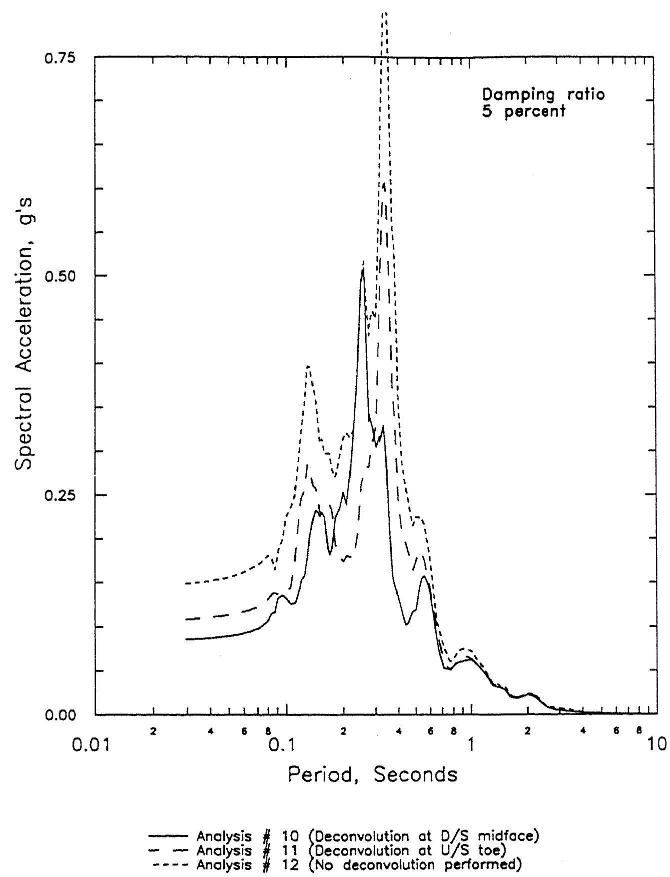


Fig. 13. Response Spectra at Dam Crest - Effect of Deconvolution

The method adopted for deconvolution of rock outcrop motions to input bedrock motions can have a significant effect on predicted dam response. The differences between peak crest accelerations for "analysis without deconvolution" and "analyses with deconvolution" were about 60 to 80 percent for the Loma Prieta Earthquake.

ACKNOWLEDGEMENTS

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