NONUNIFORM GROUND MOTION EFFECTS AT PACOIMA DAM

John F. Hall and Steven W. Alves

Department of Civil Engineering
California Institute of Technology
Pasadena, CA

Abstract

A magnitude 4.3 earthquake was recorded on an array of accelerometers at Pacoima Dam on January 13, 2001. The records are used for two purposes: (1) to analyze the effects that canyon topography has on the ground motion along the abutments, and (2) as input for a system identification study, leading to a calibrated finite element model of Pacoima Dam. The quantified amplification and time delay characteristics of the 2001 abutment motions serve as a basis for generating records to replace ones that went off-scale during the 1994 Northridge earthquake. These generated records were then used in the finite element model to verify that nonuniform ground motion caused by the topography has a significant impact on the dam response. Forced vibration tests were also conducted in July/August 2002.

Introduction

Pacoima Dam is a concrete arch dam located north of Los Angeles. The dam is 113 meters high, with thickness varying from about 3 meters at the crest to 30 meters at the base. The crest is approximately 175 meters in length. There is a spillway tunnel 18 meters below the crest. The dam has eleven contraction joints with keys that are 30 cm deep. A well known ground motion record obtained above the south abutment (referred to as the left abutment) during the 1971 San Fernando earthquake reached accelerations of 1.25g horizontal and 0.70g vertical which have been attributed to topographical amplification. After this event, a more extensive 17-channel accelograph array was installed and was in place during the 1994 Northridge earthquake. Nine of these channels were located on the dam-rock interface in order to capture the spatially nonuniform features of the seismic input, and the remaining eight channels were located either on the dam crest or at 80% height on the downstream face of the dam (Figure 1). Unfortunately, middle portions of most of the 1994 accelerograms recorded at stations above the base of dam contained off-scale high frequency motions which could not be digitized. These motions probably resulted from impacts in the contraction joints between the blocks of the dam and at the thrust block on the left abutment. Only channels 8-11 were able to be digitized (CSMIP, 1994), which included the three channels at the base. During the Northridge earthquake, the water level was about 40 meters below the crest. Movement of a rock mass occurred on the left abutment in both the 1971 and 1994 events, more severely in the latter. These movements opened and reopened a gap in the joint at the south thrust block. Repairs undertaken after both earthquakes included stabilization of the damaged abutment and filling the gap at the thrust block.

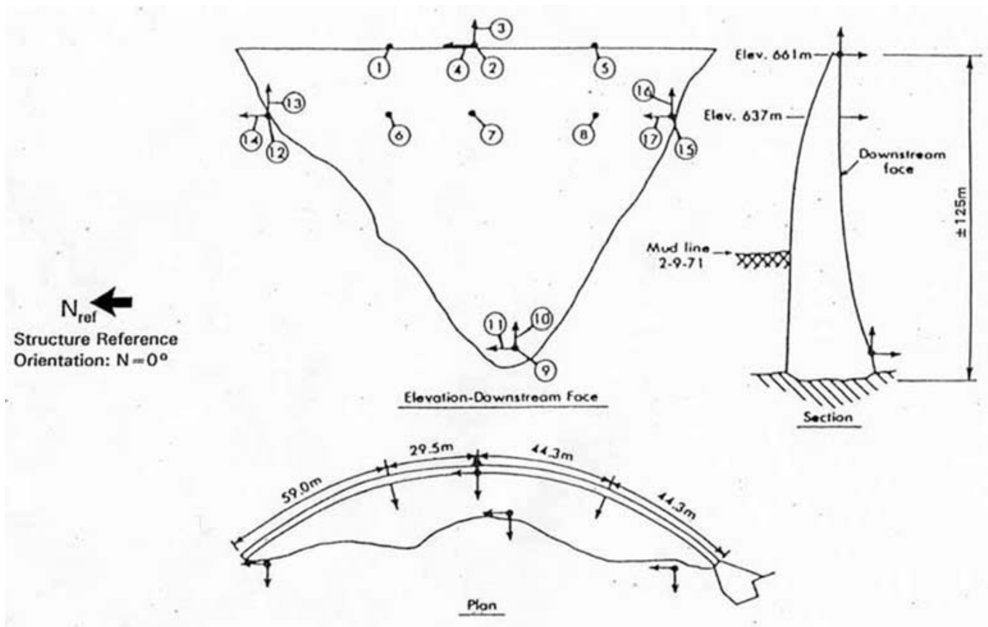

Figure 1. Location of the 17 recording channels at Pacoima Dam (CSMIP, 2001)

After 1994, the 17 analog channels were replaced by a digital array at the same locations. These digital stations recorded a magnitude 4.3 earthquake on January 13, 2001, centered about 5 km from the dam. During this event, the water level was about 41 meters below the crest. This complete set of records gave an opportunity to study the degree of nonuniformity in the ground motion and the effect that it has on the dam response. It is also of interest to determine the level of damping that is present in the dam system. These factors are important to consider for earthquake response of dams, and they have not been adequately quantified to date.

A system identification study was undertaken using the January 2001 records, in which parameters were fit for a 2-mode linear system. Even though the dam response should be predominantly linear at the excitation levels present, there were some uncertainties associated with the system identification study, and forced vibration testing was conducted to better understand the modal properties.

Using the parameters determined from both the system identification study and the forced vibration testing, a finite element model was calibrated. With this model, the effects of ground motion nonuniformity and larger amplitude motion were investigated. Some of the cases studied employed ground motions formulated to represent the seismic input from the 1994 Northridge earthquake. These motions were generated using the 1994 records at the base of the dam and information from the 2001 earthquake that quantified how the motions varied up the sides of the canyon from the base. The generated input motions replaced the unusable, off-scale records obtained on the abutments in 1994.

Description of January 2001 Records

The 17-channel array located on the downstream face of the dam is shown in Figure 1. Channels 1-8 are on the dam body: six of these channels are oriented radially, one channel is tangential, and one channel is vertical. Channels 9-17 are located at three stations near the dam-foundation rock interface. At each station, one channel is oriented in the east-west direction, one is vertical, and one is north-south. Channels 9-11 are at the base of the dam; channels 12-14 are at the north abutment (referred to as the right abutment); and channels 15-17 are at the south abutment (referred to as the left abutment), where the dam and the thrust block meet.

Acceleration time histories from the 2001 earthquake excluding vertically oriented channels are plotted in Figure 2. The highest accelerations at the interface and on the dam body are 0.10g and 0.16g, respectively. Since the level of shaking is much lower than during the 1994 Northridge earthquake, the acceleration records show none of the off-scale high frequency motions which characterized the Northridge accelerograms.

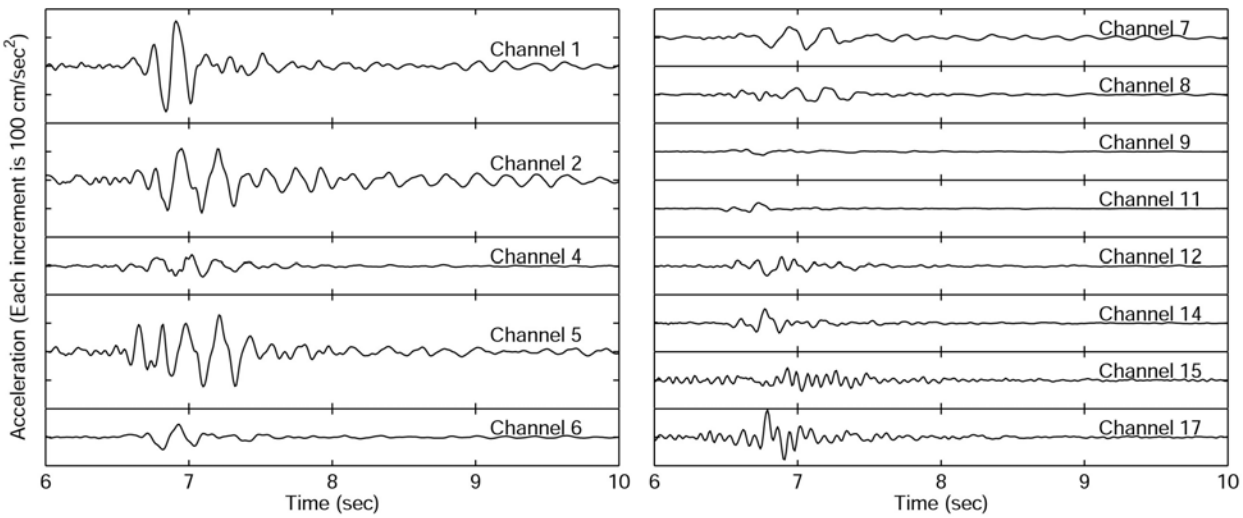

Figure 2. Hor. components of acceleration recorded at Pacoima Dam on January 13, 2001

Although the intensity of the input motion from the 2001 earthquake is not high, the amplitude and phase variations around the canyon which occur during earthquakes are probably more dependent on frequency than overall amplitude. Therefore, these characteristics of nonuniform input should be fully represented in the 2001 data and so can still be quantified and taken as indicative of larger events.

The recorded motions from the 2001 earthquake on the right and left abutments are of higher amplitude than those at the base. The amplification is shown in Figure 3 as a function of frequency where plots of ratios of 5% damped response spectra computed from the respective components of the abutment and base motions are presented. At the fundamental frequency of the dam, which is shown to be about 5 Hz in the following section (actually two frequencies near 5 Hz), the most amplification is seen in the north-south component (cross-stream direction) at the left abutment, which is where the damage occurred in previous earthquakes. However, the other two components on the left abutment are amplified about the same, or even less, than the respective ones on the right abutment. At 5 Hz, the amplification on the left abutment in the north-south direction is about 3.5, and for the other channels, the amplification ranges from 2 to 3. Amplification factors above 4 occur for two channels.

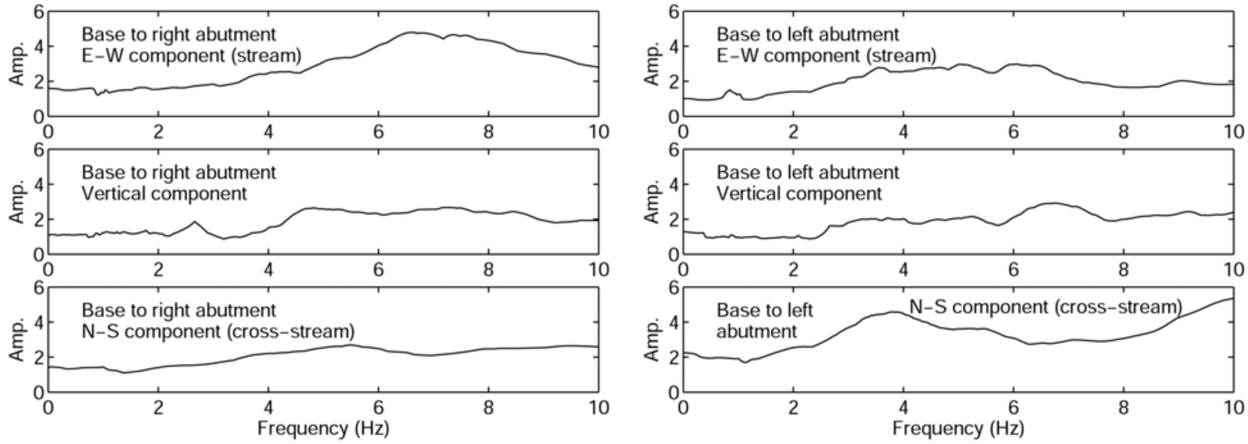

Figure 3. Amplification on the abutments of Pacoima Dam referred to motion at the base of the dam in terms of ratios of response spectra (5% damping)

Another aspect of the nonuniformity in the input is time delay. This quantity can be found between any two motions by integrating their product as a function of time shift between the two. The shift for which this correlation integral is maximized is the time delay, which is listed in Table 1 for respective components of the motions from the base station to the two abutment stations. The delays were computed using the recorded accelerations; the velocities or displacements may also be used and give somewhat shorter delays. As seen, the abutment motions in the horizontal directions lag (positive time delay) the base motions by times ranging from 40 to 66 milliseconds. These delays are a significant fraction of the fundamental period of the dam, which is about 200 milliseconds. Time delays for the vertical component are less, which could be due to the presence of an increased fraction of faster travelling compression waves for the vertical component of ground motion.

	E-W	Vertical	N-S
Base to right abutment	0.050 sec	0.024 sec	0.048 sec
Base to left abutment	0.040 sec	-0.008 sec	0.066 sec

Table 1. Time delays from the base station to the stations on right and left abutments for E-W (stream), vertical and N-S (cross-stream) components of the 2001 earthquake records

A long range goal of collecting ground motion data at the base and sides of canyons, as at Pacoima Dam, is to develop rules for prescribing nonuniform seismic input in safety assessment analyses of dams. Based on the data presented here, one could propose that time delay be a function of elevation and shear wave speed in the rock to account for travel time of seismic waves. For Pacoima Dam, there is about an 84 meter elevation difference between the base and abutment recording stations, and a shear wave velocity for rock of 1200 to 2300 m/sec can be assumed, which is based on a range of previously determined rock properties (Hall, 1988). Using these properties and assuming an upward propagating shear wave result in a time delay of 36 to 70 milliseconds, which includes the range found for the horizontal components of ground motion (Table 1). For the vertical component of ground motion, a time delay in this range could also be appropriate because the shear wave amplitudes are still larger than the compression wave amplitudes (CSMIP, 2001). An additional rule expressing amplification as a function of frequency and elevation could be formulated by averaging the results shown in Figure 3. Such rules would be applied to components of a reference motion to generate a suite of motions around a canyon. A major difficulty is how to select the reference motion. Should it be considered representative of a bottom site, in which case it would be amplified and time delayed up the canyon sides, or a site near the crest of the dam, in which case it would be attenuated and time forwarded down the canyon sides, or somewhere in between? If the reference motion is to be selected by current standard procedures that are used to produce a uniform motion to be applied to the dam, this becomes an interesting question.

System Identification

System identification was performed using the computer program MODE-ID (Beck and Jennings, 1980; Werner, Beck and Levine, 1987), which models a structure as a linear system with classical normal modes excited by spatially nonuniform ground motion. To run MODE-ID, the user specifies the number of modes to be included and supplies the accelerations recorded on the structure to be used as output response and the accelerations recorded around the structure to be used as input. For Pacoima Dam, channels 1-8 were the output and channels 9-17 were the input. The program uses these time histories to determine the frequencies, damping, shapes, and participation factors for each mode, as well as the pseudo-static response matrix, which produce the best fit to the recorded structural motions. The best fit is determined by minimizing, in the least-squares sense, the mean-square error between the measured and modeled output responses. Theoretically, MODE-ID computes the response of a system which is fixed at the locations of the input, i.e., foundation interaction effects are not included. This is not so clear in the present case of Pacoima Dam where the input is significantly nonuniform and measured at relatively few locations.

MODE-ID allows all parameters, including the pseudo-static matrix, to be adjusted to obtain the best fit, or some parameters can be fixed to predetermined values. Each entry in the pseudo-static matrix corresponds to the response at one of the output channels if one of the input channels is moved a unit amount while the others are held fixed. In the case of Pacoima Dam, allowing the pseudo-static component to be freely adjusted yields results that do not make physical sense. For this reason, the pseudo-static matrix was computed using the finite element model that will be described in a later section. Using this prescribed pseudo-static component, a two-mode model was fit to the 2001 earthquake records. A solution with modal frequencies of 4.74 Hz and 5.05 Hz with damping of 6.2% and 6.6% of critical, respectively, was found. The first mode has a nearly symmetric shape, and the second mode is predominantly antisymmetric. The estimated mode shapes at crest level are shown in Figure 4 with the undeformed crest shown for reference. Shown in Figure 5 are the output accelerations produced by MODE-ID for the horizontal channels at the crest, along with the recorded accelerations. The agreement is good.

Since the first two modal frequencies are closely-spaced, MODE-ID may have difficulty distinguishing some of the modal properties. For example, the mode shapes fit by MODE-ID, as shown in Figure 4, appear to violate orthogonality. The antisymmetric shape may include a component of the symmetric mode. The difficulty could also hold for damping, and, in fact, the measure-of-fit has been shown to be relativley insensitive to the damping ratios, because of a trade-off between participation factors and damping (Beck and Jennings, 1980). However, the modal frequencies are believed to be accurate.

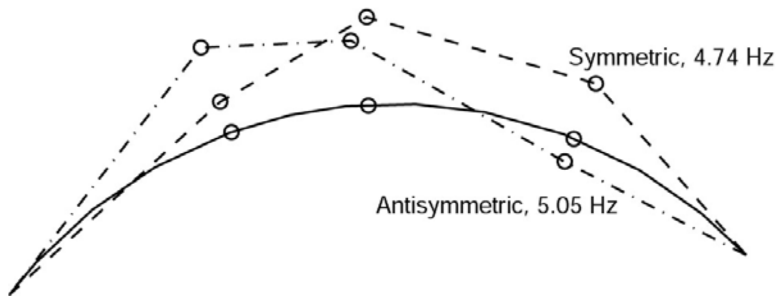

Figure 4. Symmetric and antisymmetric mode shapes computed using MODE-ID, the open circles are the locations of the crest level stations

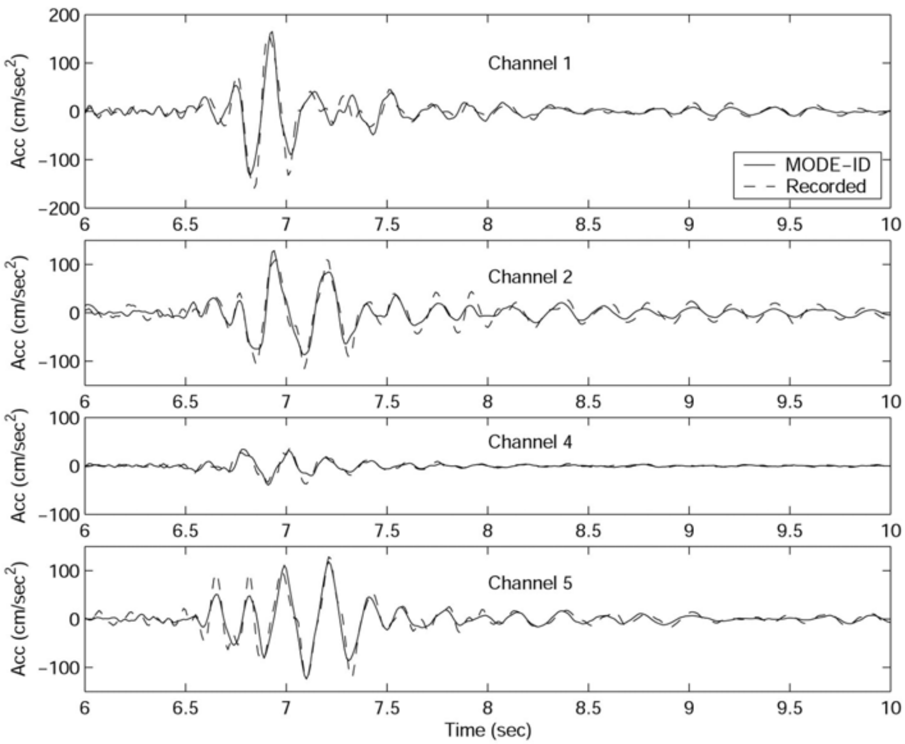

Figure 5. Comparison between the recorded accelerations at channels 1, 2, 4 and 5 and the best-fit accelerations from MODE-ID system identification study

Arch dams typically have two closely-spaced modal frequencies, and these correspond to mode shapes which can be classified as symmetric and antisymmetric. Previous forced vibration tests on Pacoima Dam in April 1980 revealed frequencies of 5.45 Hz and 5.60 Hz for symmetric and antisymmetric modes, respectively (ANCO Engineers, 1982), which are higher than the MODE-ID determined values. The water level during the 1980 tests was about 23 meters below the crest, 18 meters above that during the 2001 earthquake. At these levels, the reservoir should not significantly affect the frequencies, but the 1980 forced vibration determined frequencies would be expected to be even higher if the reservoir was at the 2001 elevation. Damping from the 1980 tests also exceeded that from MODE-ID, but data from those tests were of poor quality and this made it difficult to determine damping accurately. However, even the 6% to 7% damping estimated by MODE-ID seems on the high side compared to forced vibration results from other dams (for example, 1.4% to 4.0% at Morrow Point Dam and 1.8% to 3.1% at

Monticello Dam; see Hall, 1988). In addition, if the MODE-ID methodology is consistent with a rigid foundation, the frequencies with flexible foundation would be even lower than the determined values of 4.74 Hz and 5.05 Hz, increasing the discrepancy with the forced vibration frequencies determined in 1980. To investigate further, additional forced vibration tests were performed on Pacoima Dam.

Forced Vibration Testing

The testing was carried out over one week in July/August 2002. During the testing, the water level was 36 meters below the crest of the dam, 5 meters higher than during the 2001 earthquake. An eccentric mass shaker with force proportional to excitation frequency squared was used to generate the input. Frequency sweeps were conducted from 2.5 Hz to 11.0 Hz for shaking in both the stream and cross-stream directions.

Kinemetrics SS-1 Ranger seismometers were used to measure the motion at 5 locations in two perpendicular, horizontal directions. The Rangers have a response proportional to velocity at frequencies above their natural frequency, which is approximately 1 Hz. The Rangers were placed near the existing accelerometers at the three crest locations on the downstream side, oriented radially and tangentially, (center C, right third R, left quarter L), and at the two locations along the right and left abutments, oriented east-west and north-south, about 24 meters below the crest. The shaker was placed on the upstream side of the dam crest about 3 meters north of Ranger location C. Directions of shaking were radial and tangential at this point; see Figure 6.

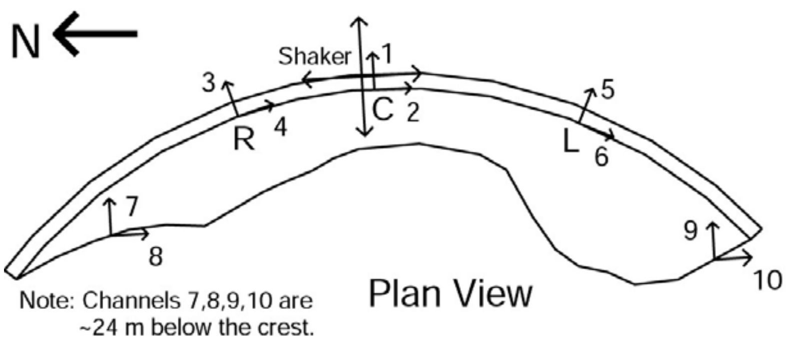

Figure 6. Ranger and shaker force locations and directions

For a perfectly symmetric dam with the shaker at the centerline, shaking in the stream direction excites only symmetric modes and shaking cross-stream excites only antisymmetric modes. This is because the motion of the dam centerline is in the stream direction for a symmetric mode and cross-stream for an antisymmetric mode. At Pacoima Dam, due to the lack of sufficient symmetry, the directions of motion at location C for the first symmetric and antisymmetric modes were both primarily in the stream direction with a difference of only 35 degrees between the two directions, compared to a difference of 90 degrees for a perfectly symmetric dam. As a result, there is considerable interference between the two modes for both directions of shaking, and this makes the determination of natural frequencies and damping difficult. Figure 7 shows the interfering resonances of the first symmetric and antisymmetric modes between 5 Hz and 6 Hz in the response of channel 1 under the stream shake. The amplitude scale in the figure is proportional to the displacement of the dam per unit shaker force.

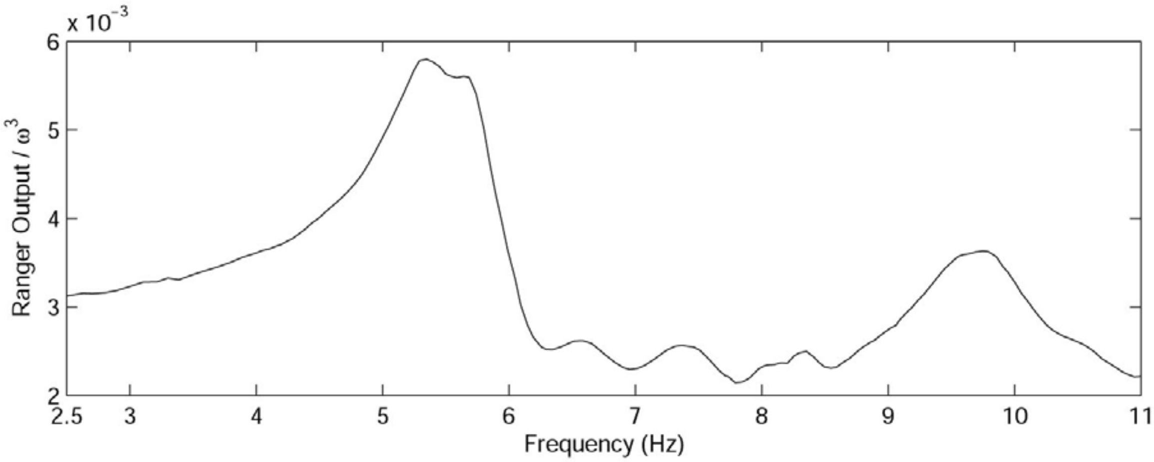

Figure 7. Frequency response curve for channel 1 at location C for the stream shaking test

One technique to eliminate interference between two modes is to align the direction of shaking perpendicular to the motion of one of the modes, which should eliminate the response of that mode, thus isolating the other one (Duron and Hall, 1986). For the Pacoima Dam data, this was done mathematically by combining the results of the two shaking directions vectorially. As a further enhancement, the pair of Ranger data channels at locations R, C and L were also combined vectorially in order to maximize the peak of the mode being isolated. This method yielded resonant frequency and damping for the antisymmetric mode of 5.70 Hz and 5.0% to 5.5%, respectively, with the damping determined by the half-power method. However, for the symmetric mode the resonant frequency could only be estimated to be between 5.30 Hz to 5.50 Hz with damping of 5.3% estimated only from location L.

A second attempt to isolate the first symmetric and antisymmetric modes was also made, based on the premise that for channels 3 and 5 (radial at locations R and L), the symmetric mode should be in phase and the antisymmetric mode should be out of phase. Using the stream shake, varying amounts of the two radial responses were added until the antisymmetric mode disappeared as much as possible, and varying amounts of the two responses were subtracted until the symmetric mode disappeared as much as possible. Values of natural frequency and damping were determined as 5.45 Hz and 4.0% for the symmetric mode and 5.70 Hz and 4.0% for the antisymmetric mode. A summary of the modal parameters appears in Table 2. The measured mode shapes for the first two modes are plotted in Figure 8.

Mode	Natural Frequency	Damping
Symmetric	5.45 Hz	$4.0\% < \zeta < 5.5\%$
Antisymmetric	5.70 Hz	$4.0\% < \zeta < 5.5\%$

Table 2. Estimated modal parameters

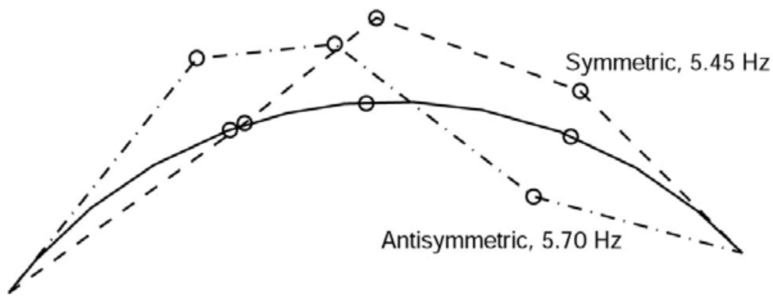

Figure 8. Symmetric and antisymmetric mode shapes determined from forced vibration testing, the open circles are the locations of the crest level stations

Comparison of Modal Properties Between Forced Vibration Tests and 2001 Earthquake

A summary of the frequencies and damping values for the first symmetric and antisymmetric modes of Pacoima Dam are presented in Table 3. Included are results from the April 1980 and July/August 2002 forced vibration tests, as well as the MODE-ID determined values from the January 2001 earthquake. In particular, the frequencies of the first symmetric and antisymmetric modes from the July/August 2002 forced vibration tests are 15% and 13% larger, respectively, than those from the earthquake.

Event	Date	Water Level	Freq 1st sym	Damping 1st sym	Freq 1st Anti	Damping 1st Anti
FVT	April 1980	-23 m	5.45 Hz	? 7.3% ?	5.60 Hz	? 9.8% ?
FVT	July/Aug 2002	-36 m	5.45 Hz	4.0%-5.5%	5.70 Hz	4.0%-5.5%
EQ	Jan 2001	-41 m	4.74 Hz	6.2%	5.05 Hz	6.6%

Table 3. Summary of determined modal frequencies and damping values of Pacoima Dam from two forced vibration tests (FVT) and the January 2001 earthquake (EQ)

The presence of nonlinear effects in structures during earthquakes typically causes resonant frequencies to decrease and effective damping to increase. However, the January 2001 earthquake produced fairly small amplitude motions of Pacoima Dam (peak acceleration and velocity on the crest of 0.16g and 6.2 cm/sec, respectively) which are thought to be still in the linear range. This is confirmed by a finite element simulation reported in the following section for the January 2001 earthquake for which no cracking in the concrete and only a very small amount of joint opening occurs.

Although some of the difference between the forced vibration tests and January 2001 earthquake may be attributed to errors in accurately determining the damping values, as discussed in previous sections, the frequencies are believed to be accurate, and so their differences are harder to explain. A decrease in the frequency of the first symmetric mode from 5.45 Hz during the July/August 2002 forced vibration tests to 4.74 Hz during the January 2001 earthquake requires a reduction in stiffness of the dam system of 32%, and the 5.70 Hz to 5.05 Hz decrease for the antisymmetric mode requires a stiffness reduction of 27%. These changes in stiffness are quite large and would be hard to justify. Nor can the lower frequencies exhibited during the January 2001 earthquake be attributed to difference in water level. First, the levels (Table 3) were all low enough to have only a minor effect on the frequencies. Second, the lowest water level occurs for the January 2001 earthquake, while a higher level is needed to explain the lower MODE-ID determined frequencies. Thus, the discrepancy between the modal frequencies observed during the forced vibration tests and identified from the January 2001 earthquake response is an interesting feature of the responses of Pacoima Dam, but it remains unexplained.

SCADA Finite Element Model Calibration

A finite element model of Pacoima Dam, massless rock foundation, and incompressible water reservoir was constructed with the computer program SCADA, Smeared Crack Arch Dam Analysis (Hall, 1996). Shell elements are used for the dam, solid brick elements for foundation, and pressure brick elements for the water (Figure 9). Rayleigh damping is employed using the stiffness and mass matrices of the dam and the stiffness matrix of the rock to construct a proportional damping matrix. The foundation model is connected only to the dam, and for modeling purposes the thrust block at the left abutment is considered to be part of the foundation. Nodes of the water mesh are fixed down to the surface elevation of the reservoir. SCADA uses the smeared crack method to model opening, closing and sliding nonlinearity associated with contraction joints and cracks in the dam, or it can operate in a linear mode. The nonlinear model has eleven contraction joints, which is consistent with the actual dam.

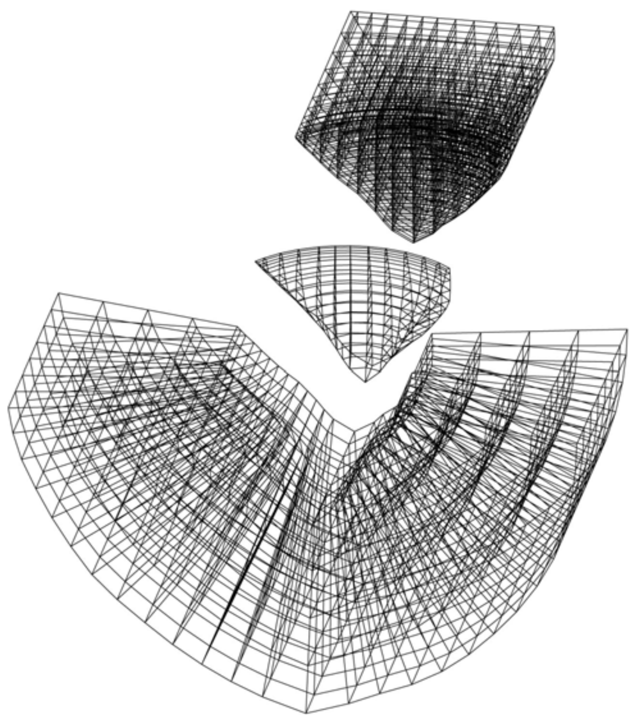

Figure 9. Finite element meshes for Pacoima Dam, rock foundation and water reservoir

The calibration was first performed by choosing values for the material properties so that the natural frequencies computed from the linear model for the first symmetric and antisymmetric modes matched those measured during the July/August 2002 forced vibration tests. In a second calibration, these moduli of the concrete and rock were scaled in proportion to match the MODE-ID determined natural frequencies. This second model was then used in a

SCADA analysis to see if the recorded motions on the dam during the January 2001 earthquake could be reproduced. For this latter exercise, SCADA was modified to accept earthquake ground motion input which was nonuniform along the abutments. All of the calibration studies set the water level to 38 meters below the crest, which was close to the level during both the 2002 forced vibration tests and the 2001 earthquake.

In the first calibration to match the forced vibration determined modal frequencies, the concrete and foundation rock material properties chosen were: Young's moduli of 26,200 MPa (3800 ksi) for concrete and 13,800 MPa (2000 ksi) for rock, Poisson's ratios of 0.20 for concrete and 0.25 for rock, and unit weight of 22.0 kN/m^3 (140.0 lb/ft^3) for concrete. The computed frequencies for the first symmetric and first antisymmetric modes are 5.45 Hz and 5.69 Hz, respectively, compared to the measured values of 5.45 Hz and 5.70 Hz, respectively. The computed mode shapes at these two frequencies are shown in Figure 10.

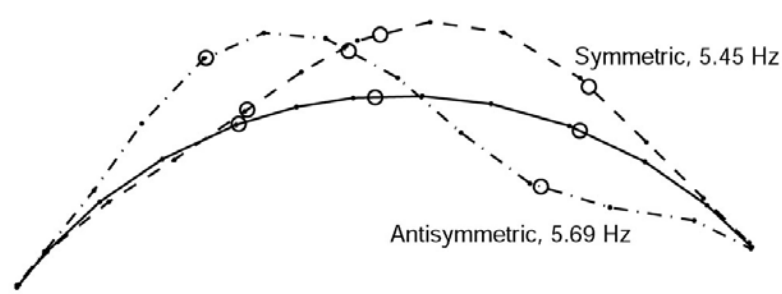

Figure 10. Symmetric and antisymmetric mode shapes computed from linear SCADA model calibrated to match forced vibration modal parameters, the open circles are the locations of the crest level stations

For the calibration to the MODE-ID determined frequencies, a 29% reduction in the Young's moduli to 20,300 MPa (2950 ksi) for the dam concrete and 10,700 MPa (1550 ksi) for the foundation rock reduced the frequencies of the first symmetric and antisymmetric modes computed from the linear model to 4.80 Hz and 5.01 Hz, respectively, close to the MODE-ID frequencies of 4.74 Hz and 5.05 Hz, respectively. The rock modulus is in the range of a rather large variation of field data (Woodward-Lundgren, 1971); it corresponds to a shear wave velocity of about 1300 m/sec. The concrete modulus is in the typical range for dam concrete. The computed mode shapes are similar to those shown in Figure 10.

Another factor to consider for this second calibration is whether the MODE-ID determined frequencies include the effect of foundation flexibility. Since the recorded input motions are on the dam-foundation interface, the theoretical answer is no; however, since the input is only sparsely sampled, this conclusion is questionable. If the finite element model is altered to have a much stiffer foundation, it is found that the symmetric mode is stiffened more than the antisymmetric mode to the point that the antisymmetric mode has the lower frequency. This is not consistent with either the MODE-ID or forced vibration results, and the forced vibration response includes interaction with the foundation. Therefore, it was concluded that the system being modeled by MODE-ID is closer to having a flexible foundation.

As mentioned above, the simulation of the 2001 earthquake response was run with a modified version of SCADA to incorporate nonuniform input. Like the original version of SCADA with uniform ground motion, the nonuniform input is free-field motion, i.e., that which would occur during the earthquake at the dam-foundation interface if the dam were not present. The earthquake is represented by a set of forces which, if applied to the foundation nodes at the interface of the dam with the dam mesh absent, would produce the desired free-field motions. These forces are computed from the nonuniform input displacement and velocity time histories by multiplying them by the foundation stiffness and damping matrices, respectively, and adding. In the analysis, these forces are then applied to the nodes of the interface of the dam and foundation with both meshes present. The water mesh is also included, and the excitation of the water from accelerations of the canyon bottom and sides is performed as in the original version of SCADA except that the distribution of acceleration can be nonuniform.

The records at Pacoima Dam from the base and the two stations on the abutments are not free-field records and so, theoretically, they should be applied with a rigid foundation. However, as mentioned above, the finite element model calibration to the MODE-ID determined frequencies is better with a flexible foundation, and so the simulation of the January 2001 earthquake will use this model with its flexible foundation and apply the recorded motions as if they were free-field motions. This is thought to be a reasonable approximation.

Since the January 2001 accelerograms to be used as input in the simulation were recorded only at the base and the two abutment stations, some interpolation and extrapolation is necessary. Motions at nodes on the north side of the canyon are interpolated from the right abutment and base records, similar for the south side of the canyon using the left abutment and base records. Interpolation is performed channel by channel, and the interpolation at a node is weighted according to the elevation of the node. Before interpolation, any time delay is eliminated by shifting, and then the interpolated record is appropriately re-shifted based on its nodal elevation. For nodes located higher than the abutment stations, larger amplitudes and time delays were extrapolated. Displacement and velocity time histories are integrated from the interpolated/extrapolated acceleration time histories, and used to compute the forces to be applied at the dam-foundation interface. The interpolated/extrapolated accelerations themselves are used for the water excitation.

The simulation of the January 2001 earthquake response used the material properties resulting from the MODE-ID study. With the MODE-ID estimates as a guide, stiffness proportional damping was specified to give modal damping of about 6.7% and 7.0% for the symmetric and antisymmetric modes, respectively. The tensile strength of concrete was set to 3.79 MPa (550 psi).

With nonlinear behavior allowed, the calibrated finite element model subject to the January 2001 earthquake input motions exhibited minimal joint opening, mostly near the crest that was limited to less than 0.03 cm, and there was no cracking. Keys were present to prevent sliding between the joints. The resulting displacement responses at locations corresponding to channels 1, 2, 4 and 5 are compared to the actual recordings in Figure 11. The agreement is good, although the computed response overestimates the displacement during the period of strongest shaking. The agreement between computed and recorded accelerations (not shown) is not as good. The computed accelerations generally overestimate those of the records, and there are some high frequency spikes that show up in the computed response for channel 4 that are not present in the record. This is due to banging at a joint in the model. However, the agreement is good enough to verify the calibration of the finite element model for use in the earthquake analyses of the next section.

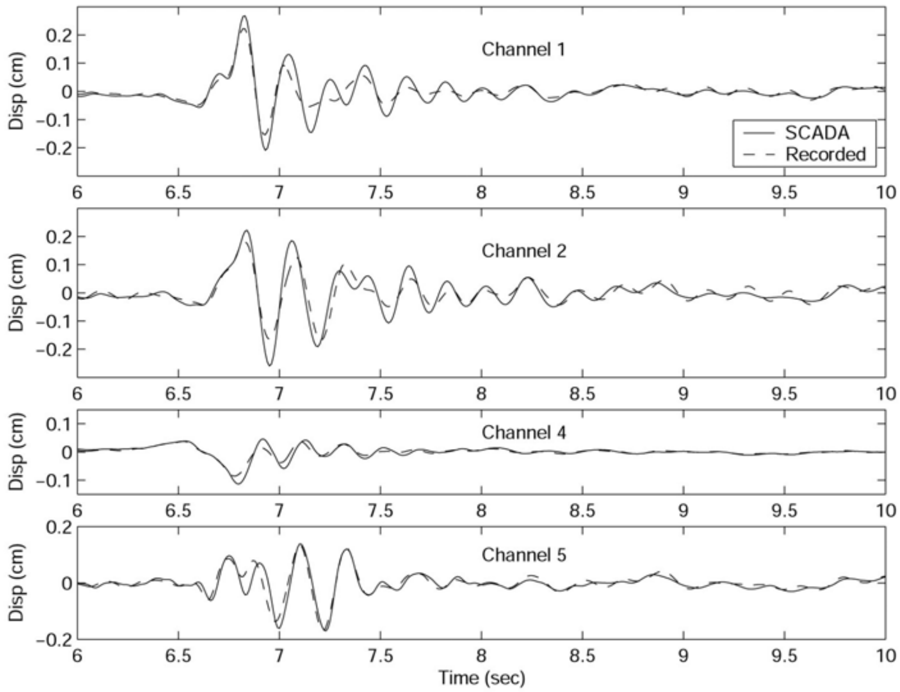

Figure 11. Comparison between the recorded displacements at channels 1, 2, 4 and 5 and the computed displacements from the SCADA finite element model

Effects of Nonuniform Ground Motion

The finite element model of Pacoima Dam calibrated to the MODE-ID determined properties was used to study the effects of nonuniform ground motion compared to the uniform motion assumption that is commonly used in dam engineering practice. For this purpose, the January 2001 earthquake was considered to be too small to lead to meaningful conclusions. Therefore, it was decided to use the motions from the 1994 Northridge earthquake. However, because of the off-scale problem, only the input records from the base of the dam are available. The motions at the two abutment recording stations had to be generated from these base motions, and this generation was based on the results of the analysis of the January 2001 records.

The amplification factors in Figure 3 were approximated using piecewise linear functions of frequency. These approximate amplification factors are shown in Figure 12. This amplication was applied to the base acceleration histories from the 1994 Northridge earthquake to generate the amplified records at the right and left abutment recording stations. The computation was done by Fourier transforming a record to the frequency domain, scaling it frequency by frequency, and then transforming it back to the time domain. The amplified records were then time shifted to be consistent with the time delays determined from the January 2001 earthquake records (Table 1). As previously argued, the time delay is assumed to be the same for all three components on each abutment. The records were delayed in time 48 and 54 milliseconds for the two recording stations on the right and left abutments, respectively. The generated abutment and base accelerations for the Northridge earthquake are shown in Figure 13. The maximum acceleration for these records is approximately 1.3g at channel 17. Another set of amplified records was also generated, but with no time delay in the right and left abutment records, so the importance of the effect of travel time could be evaluated.

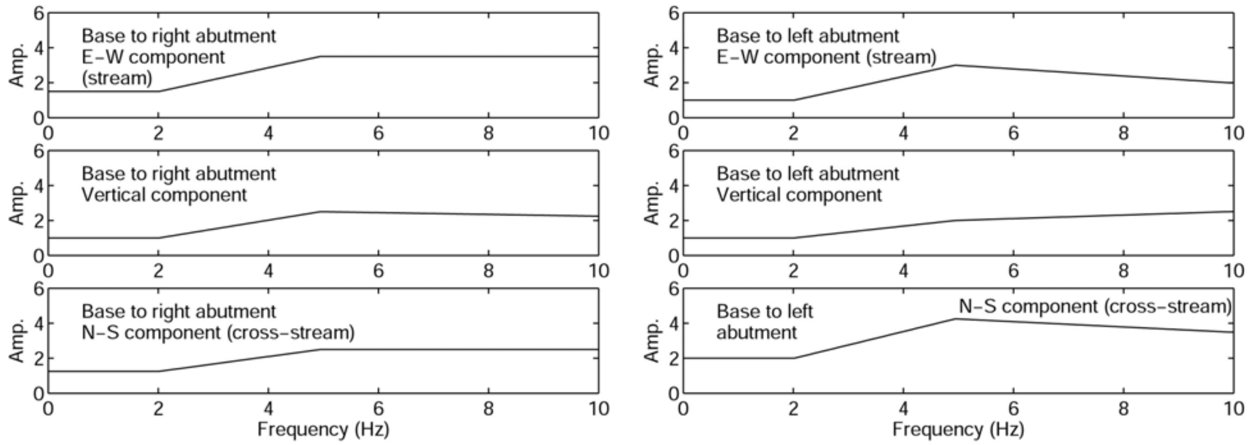

Figure 12. Amplification factors used to generate the motions at the abutment recording stations from the Northridge earthquake base records

The original Northridge records at the base of the dam and the generated records at the two abutment locations were interpolated/extrapolated to the nodes of the finite element model along the dam-foundation interface, as described in the previous section. The maximum acceleration on the crest increased to 1.5g after extrapolation from the channel 17 record. Time integration was performed on these acceleration histories to produce the velocity and displacement histories needed in the computation of the earthquake forces to be applied to the interface nodes. Any residual velocity or displacement was zeroed by subtracting a linear trend from the integrated records.

Using the generated records as input, several cases were run (Table 4). The complete nonuniform ground motion was input for three reservoir levels: 38 meters, 20 meters, and 5 meters below the crest. The first corresponds to a level near to the Northridge earthquake elevation; the second corresponds to the full condition at the invert of the spillway; and the third corresponds to a flood condition. These same reservoir elevations were also run with the records scaled up by a factor of 1.5. Uniform ground motion cases were run at the two lower reservoir depths (Northridge level and full) for three cases each where the ground motions for the base, right abutment and left abutment recording stations were applied uniformly. The nonuniform ground motion with no time delay was also run at these two reservoir levels. All of these cases allow fully nonlinear behavior except that keys are present to prevent sliding between the joints.

A summary of the results from these analyses is given in Table 5. Case 1 is an approximation of the conditions present during the 1994 Northridge earthquake. The main goal of the study was not to attempt to exactly duplicate the Northridge earthquake, but some comparisons can be made. The joint opening and cracking seem to be fairly consistent with observations. After the earthquake there was a 5 cm opening at the top of the joint between the dam and the thrust block at the left abutment. The opening extended 18 meters down the joint, at which point a large crack extended diagonally into the foundation (Hall, 1995). The residual opening is not present in the model results because the residual input displacement was zero, but the largest computed joint opening does occur at the left abutment and is on the order of the actual observed residual opening. Also, cracking in the model results is limited mostly to elements along the abutments, including elements on the left abutment near the location where the actual crack was observed. Cracking was not reportedly observed in most of the dam body after the Northridge earthquake.

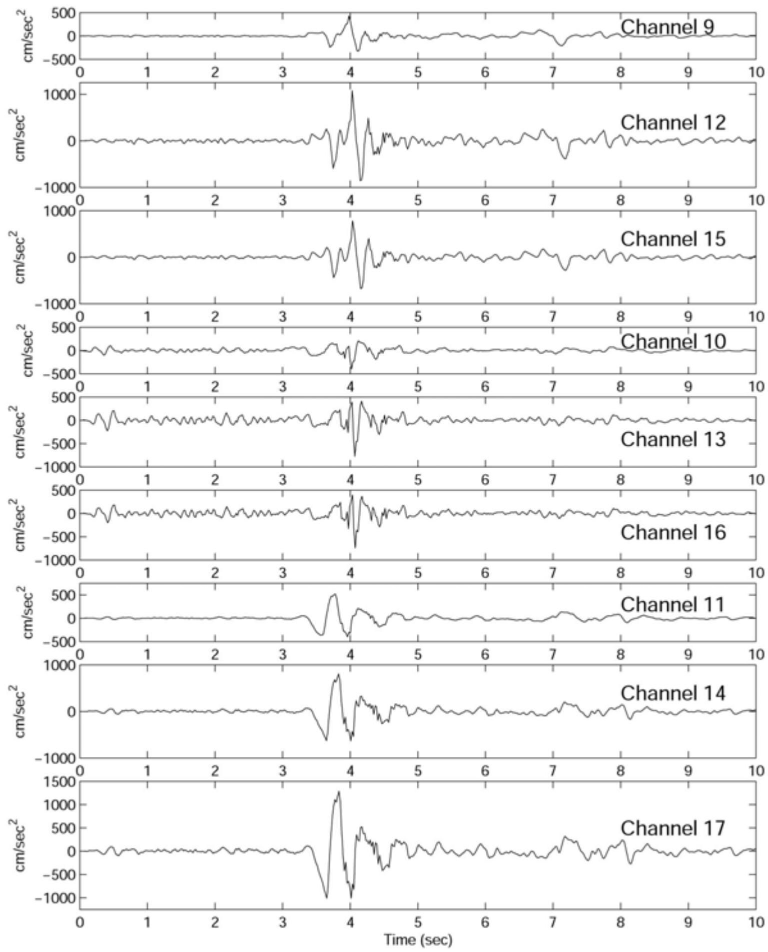

Figure 13. Northridge earthquake accelerations on the abutments (channels 12-17) generated from the existing base records (channels 9-11)

The maximum arch compressive stresses listed in Table 5 for the cases involving nonuniform input are quite large and exceed somewhat the compressive strength of concrete. For example, the value for Case 1 is 29.66 MPa (4300 psi). Figure 14(a) shows that this large arch compression occurs in a localized region in the corner of the dam at the top of the left abutment. This situation is typical of the other cases. The primary cause of the large arch compressive stresses seems to be the variation of the amplified input displacement along the interface between the dam and foundation. The stresses are fully present in the pseudo-static component of the response, which is shown in Figure 14(b) and was computed by applying the earthquake loading very slowly so that inertial and damping effects become negligible.

Case 1	Nonuniform, water 38 meters below crest
Case 2	Nonuniform (scaled by 1.5), water 38 meters below crest
Case 3	Uniform (base), water 38 meters below crest
Case 4	Uniform (right abutment), water 38 meters below crest
Case 5	Uniform (left abutment), water 38 meters below crest
Case 6	Nonuniform (no time delay), water 38 meters below crest
Case 7	Nonuniform, water 20 meters below crest
Case 8	Nonuniform (scaled by 1.5), water 20 meters below crest
Case 9	Uniform (base), water 20 meters below crest
Case 10	Uniform (right abutment), water 20 meters below crest
Case 11	Uniform (left abutment), water 20 meters below crest
Case 12	Nonuniform (no time delay), water 20 meters below crest
Case 13	Nonuniform, water 5 meters below crest
Case 14	Nonuniform (scaled by 1.5), water 5 meters below crest

Table 4. SCADA analyses run for Pacoima Dam

Case	Arch Compression (MPa)	Joint Opening (cm)	No. of Elements Cracked	Crack Opening (cm)	Crack Sliding (cm)	Max. Ch. 2 Disp. (cm)
1	-29.66	4.28	16	0.68	0.71	-11.21
2	-43.69	5.49	37	3.42	5.34	-17.58
3	-4.52	1.47	0	0.00	0.00	-5.85
4	-13.01	8.42	24	5.40	-3.22	-14.62
5	-11.30	5.25	10	4.62	2.05	-10.59
6	-25.47	4.34	8	0.53	0.64	-11.46
7	-30.30	4.81	19	1.33	0.67	-11.32
8	-44.27	8.52	40	5.49	-3.35	-18.79
9	-4.97	1.21	0	0.00	0.00	5.94
10	-15.62	11.98	25	10.87	-5.75	-16.83
11	-11.18	8.90	14	4.20	6.08	-11.86
12	-26.03	3.68	13	0.89	0.63	-11.51
13	-30.97	5.20	24	3.71	-1.07	-12.62
14	-44.78	8.49	47	5.53	14.43	-25.42

Table 5. Maximum responses computed from the SCADA analyses

The amount of amplification for each displacement component is determined by the interpolated/extrapolated level of the corresponding amplification function shown in Figure 12 at the low frequency end. According to the figure, the highest amplification will occur for the north-south component on the left abutment and equals 2 at the elevation of the abutment recording station. Extrapolation to crest elevation gives an amplification of 2.3. This is the displacement component which is responsible for the large arch compressive stresses in the dam at the top of the left abutment.

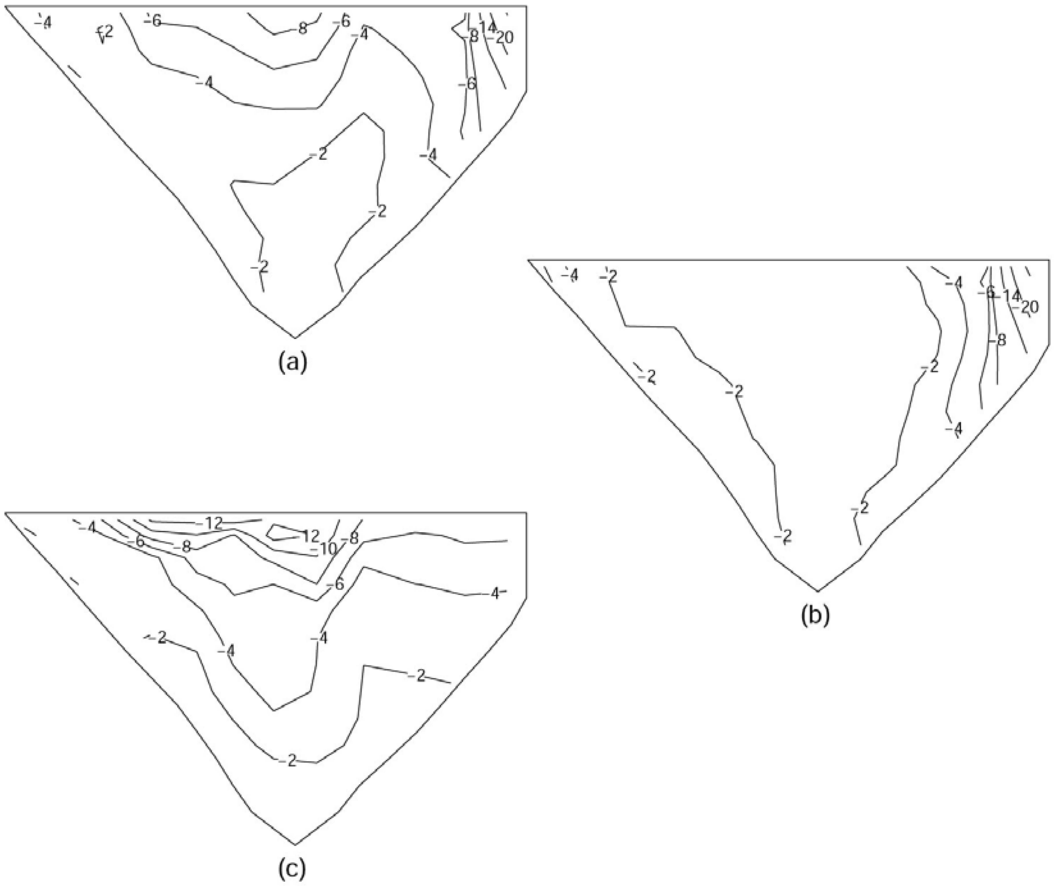

Figure 14. Contours of maximum arch compressive stress (MPa) on the upstream face for (a) Case 1, (b) Case 1 with inertial and damping effects neglected and (c) Case 4

Comparisons of the cases lead to several observations. In general, nonuniform ground motion causes more severe stresses, joint opening and cracking near the abutments (mainly the left one as discussed above) than those computed for the interior of the dam when the water level is 38 meters below the crest. As the water level is raised, the stresses, joint opening and cracking become more severe in the interior of the dam, especially near the upper center. This is due to the higher dynamic response. Of course, when the input motion is scaled up by a factor of 1.5, the response is also more severe. The number of cracked elements approximately doubles for each of the three water levels considered as the input is scaled by 1.5. The cracking is pronounced when the water is 5 meters below the crest and the scale factor is 1.5. For this case, severe cracking and crack sliding are seen in the row of elements approximately 35 meters below the crest. For Pacoima Dam, ground motion this large occurring at the same time as a flood is unlikely. However, if this extreme ground motion and reservoir elevation were the conditions used to assess the safety of a dam, this level of nonlinear behavior might be cause for concern, and more work would be necessary to examine the stability of the dam under these conditions.

The general character of the response under uniform input is quite different from that under nonuniform input. The stresses and levels of cracking and joint opening are relatively low in the region along the abutment, compared to the interior of the dam. When the base records are used as the uniform input, the response is quite mild, with no cracking at all. However, applying the right and left abutment records uniformly generates a more severe response in some ways than the nonuniform input. The joint opening and cracking in the dam interior are more severe than for any region of the dam under nonuniform motion, when corresponding cases are compared. However, the nonlinear behavior along the abutments is generally more severe when nonuniform input is used. The compressive stresses are higher in the upper interior part of the dam for the uniform input, but the left abutment stresses generated by the nonuniform motion are much larger than any stresses generated by the uniform motion (Figure 14(a) and (c)).

When nonuniform motion with no time delay is used, the stresses are generally less severe than when time delay is included. This is most pronounced near the upper part of the abutments. However, some elements near the interior of the dam are slightly more stressed when there is no time delay. The distribution of joint opening is different and the maximum opening is sigificantly larger when delay is included. The extent of cracking is less severe if time delay is omitted. Both the number and the size of cracks are decreased without the delayed input. These results are based on analysis with the water level at the full condition.

Summary and Conclusions

Pacoima Dam has been studied using records obtained from a relatively small magnitude 4.3 earthquake on January 13, 2001. The records from the two abutment stations are amplified and time delayed compared to those from the base of the dam. The complete set of records was used as input (base and abutment stations) and output (dam interior stations) for a system identification study using the computer program MODE-ID. A 2-mode model was used and the fit to the actual records was fairly good. As is often observed with concrete arch dams, the two modes identified have general symmetric and antisymmetric shapes and frequencies which are closely-spaced (4.74 Hz and 5.05 Hz). Damping for both modes was identified between 6% and 7% of critical. While the frequencies are believed to be accurate, the fit is less sensitive to the damping which means the actual range could be noticeably different.

Forced vibration tests were performed in an attempt to more precisely estimate these parameters. However, the two closely-spaced modes had motion in nearly the same direction at the shaker location, which again made it difficult to estimate damping accurately. After a considerable effort to isolate the modes from each other, the damping was determined to lie in a range between 4% and 5.5% for both modes, lower than what was computed by MODE-ID. While the mode shapes were similar to those determined by MODE-ID, the frequencies were significantly higher (5.45 Hz and 5.70 Hz). This discrepancy in the frequencies has not been explained because the dam system should have responded linearly to the small 2001 earthquake. One possibility is some nonlinearity in the slide-prone left abutment, but there is no evidence of such behavior.

The SCADA finite element model was calibrated to match the frequencies and damping determined from MODE-ID, to be consistent with the properties from earthquake excitation. The foundation rock was included in the model even though the input was recorded on the dam-foundation interface because the best fit to the identified frequencies was obtained with the foundation rock included. Simple interpolation rules were used to generate time histories to be input at each node of the SCADA model. The small January 2001 earthquake response was fairly well replicated in this way, using recorded and computed time histories for comparison, by the SCADA model. This analysis supports the assertion that the dam response was essentially linear.

The 2001 earthquake motion is not large enough to yield results that can be used to study the effects of ground motion nonuniformity on nonlinear aspects of the dam response such as joint opening and concrete cracking. The digitized records measured at the base of the dam during the Northridge earthquake provide much larger excitation, but the input is not complete because the other abutment records were off-scale and could not be processed. So, those other records were generated from the base motion using the frequency-dependent amplification and time delay data determined from the 2001 records. When this motion was input to the SCADA model with the water level the same as that during the Northridge earthquake, the model response was consistent with visual observations that were made after the event.

Several conclusions were drawn from results of the analyses run with the generated Northridge motion. For shaking as experienced during the Northridge earthquake, depending on the water level, joint opening and concrete cracking can be significant contributors to the dam response. As the water level is raised, there is a higher concentration of opening and cracking near the upper center portion of the dam. When the input motion is scaled up by a factor of 1.5, the response is, of course, more severe. With simultaneous earthquake and flood conditions and the Northridge motion magnified by 1.5, cracking is extremely severe with very high compressive stresses at the left abutment. To determine whether the dam would remain stable under these conditions, more work would be required, but these extreme conditions are unlikely for Pacoima Dam.

The SCADA analyses also demonstrated the importance of spatial nonuniformity in the ground motion. Taking the reproduced Northridge motion from a location along the abutment at 80% height of the dam, and applying it uniformly, overestimates the response in the upper part of the dam near the center. However, the response closer to the abutment is underestimated. It was determined that both topographic amplification and seismic wave travel times are important factors in the seismic response of Pacoima Dam, and therefore must be included in any seismic analysis of the dam. The seismic response, particularly near the abutment, receives a significant contribution from the pseudo-static component, which is directly related to the input displacement. This finding (also see Mojtahedi and Fenves, 2000) requires that the nonuniform input displacements be computed with care, for example, without significant integration errors.

In order to further investigate issues relevant to the seismic response of concrete dams, it is recommended that the number of instruments along the abutments at Pacoima Dam be increased. This will allow for the spatial nonuniformity of the motion to be better recorded in subsequent events. These recordings will provide data to support guidelines for generating a set of nonuniform motions from a single characteristic 3-component ground motion determined for a site. Also, if recordings from earthquakes that are even smaller than the 2001 event are obtained and analyzed, more might be learned to clarify the changes in frequencies and damping observed from forced vibration tests to small seismic events. This will be facilitated with more recording instruments because better characterization of the nonuniform input could lead to more accurate results from system identification studies.

References

ANCO Engineers, 1982. Dynamic Testing of Concrete Dams, report prepared for the National Science Foundation.

Beck, J. L. and P. C. Jennings, 1980. Structural Identification Using Linear Models and Earthquake Records, *Earthquake Engineering and Structural Dynamics*, Volume 8, Issue No. 2, pp. 145-160.

California Strong Motion Instrumentation Program, 1994. Processed Data for Pacoima Dam - Channels 8 through 11 from the Northridge Earthquake of 17 January 1994, Report OSMS 94-15A.

California Strong Motion Instrumentation Program, 2001. Processed Data for Pacoima Dam - Channels 1 through 17 from the M4.3 Earthquake of 13 January 2001, Report OSMS 01-02.

Duron, Ziyad H. and John F. Hall, 1986. New Techniques in Forced Vibration Testing, *Recent Advances in Structural Dynamics: Proceedings of a session sponsored by the Aerospace Division of the American Society of Civil Engineers in conjunction with the ASCE Convention in Seattle, Washington, April 10, 1986*, ASCE, pp. 16-33.

Hall, J. F., 1988. The Dynamic and Earthquake Behavior of Concrete Dams: Review of Experimental Behavior and Observational Evidence, *Soil Dynamics and Earthquake Engineering*, Volume 7, Number 2.

Hall, J. F., 1996. Efficient Nonlinear Seismic Analysis of Arch Dams - User's Manual for SCADA (Smeared Crack Arch Dam Analysis), Report No. EERL 96-01 (modified July 1997), Earthquake Engineering Research Laboratory, Caltech, Pasadena, CA.

Hall, J. F., ed., 1995. Supplement C to Volume 11: Northridge Earthquake of January 17, 1994 Reconnaissance Report, *Earthquake Spectra*, Volume 1.

Mojtahedi, S. and G. Fenves, 2000. Effect of Contraction Joint Opening on Pacoima Dam in the 1994 Northridge Earthquake, Data Utilization Report CSMIP/00-05 (OSMS 00-07), California Strong Motion Instrumentation Program.

Werner, S. D., J. L. Beck and M. B. Levine, 1987. Seismic Response Evaluation of Meloland Road Overpass Using 1979 Imperial Valley Earthquake Records, *Earthquake Engineering and Structural Dynamics*, Volume 15, Issue No. 2, pp. 249-274.

Woodward-Lundgren & Associates, 1971. Pacoima Dam - Determination of In Situ Dynamic Elastic Foundation Properties, report prepared for International Engineering Company.