EFFECT OF CONTRACTION JOINT OPENING ON PACOMIA DAM IN THE 1994 NORTHRIDGE EARTHQUAKE

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ABSTRACT

Pacoima Dam is constructed as cantilever monoliths separated by vertical contraction joints. A seismic safety evaluation of an arch dam often relies on a linear dynamic analysis assuming the dam is a monolithic structure. The CDMG network of strong motion accelerometers recorded the canyon and dam response in the 1994 Northridge earthquake. An analysis of dam models for the earthquake shows that joint opening occurrs and that it redistributes the stresses in the dam. The effects of non-uniform free-field motion were found to be important on the stresses and joint opening displacements.

INTRODUCTION

Concrete arch dams, such as Pacoima Dam in Los Angeles County, are constructed as cantilever monoliths separated by contraction joints. Since the joints cannot transfer substantial tensile stresses in the arch direction, the joints may open as the dam vibrates in response to earthquake ground motion. The earthquake response of arch dams is further complicated by the effects of dam–water interaction, dam–foundation rock interaction, and the spatial variation of the input motion along the interface between the dam and canyon.

A seismic safety evaluation of an arch dam often relies on a linear dynamic analysis assuming the dam is a monolithic structure and uses uniform free–field ground motion. The joint opening behavior is not represented, and hence a linear analysis can show unrealistically large tensile stresses in the arch direction. Design criteria have been developed to interpret arch dam behavior from stresses obtained from linear earthquake analysis. Although the effect of contraction joints has been recognized analytically and from shaking table tests of arch dam models, there has not been evidence of contraction joints opening during an earthquake. This situation changed on January 17, 1994, when Pacoima Dam was subjected to the Northridge earthquake. The contraction joints opened during the strong vibration. Some cracking and block offset occurred in parts of the dam body. The contraction joints closed under static forces after the earthquake, except for the left–most joint which had a permanent opening of two inches because the abutment thrust block slid downstream. The left abutment was damaged in a similar manner in the 1971 San Fernando earthquake, although the permanent opening was only 3/8–inch after that earthquake. The abutment was repaired after 1971 by post–tensioning the thrust block.

The strong motion response of Pacoima Dam in the 1994 Northridge earthquake was recorded by a network of California Division of Mines and Geology (CDMG) accelerometers (CSMIP, 1994). This data offers a unique opportunity to evaluate the contraction joint behavior in an arch dam subjected to a strong earthquake. The objective of this study is to examine the response of Pacoima Dam in the Northridge earthquake using the computer program ADAP–88 which has been developed at the University of California, Berkeley, for nonlinear analysis of arch dams considering joint opening effects (Fenves, Mojtahedi et al., 1992a; Fenves, Mojtahedi et al., 1992b). Seismic analyses of the Pacoima Dam are performed with the joints assumed closed as well as allowed to open. In view of the importance of the ground motion variation along the abutment of the dam, seismic analyses were performed considering non–uniform seismic input (based on simplifying assumptions) and the results were compared with seismic response which would have been obtained using uniform seismic input.

RESPONSE OF DAM IN THE NORTHRIDGE EARTHQUAKE

Pacoima Dam is a flood control structure constructed in the San Gabriel mountains in 1929 by Los Angeles County. The concrete arch dam is 365 ft high and has a crest length of 589 ft. The thickness of the dam at crown section varies from 10.4 ft at crest to 99 ft at base. A plan and profile of the dam are shown in Fig. 1. The water level was about two–thirds full at the time of the Northridge earthquake (234 ft above the base compared with the full operating water level of 300 ft above the base). Consequently, dam–water interaction is not particularly important because there is no hydrostatic pressure inducing compression in the upper arches, and the added mass of the water is important for reservoirs more than two–thirds full.

The epicenter of the 1994 Northridge earthquake (M_S=6.8) was approximately 11 miles southwest of the dam. Figure 2 shows the CDMG accelerometer locations on the dam and the images of the film records (CSMIP, 1994). The motion at a downstream location had a peak acceleration of 0.44 g. The accelerometer above the left abutment recorded a peak acceleration of 1.53 g, illustrating the topographic amplification of ground motion. At the base of the dam, the peak acceleration was 0.54 g in the radial direction and 0.43 g in the vertical direction. At the crest of the dam, peak acceleration in the radial direction reached 2.3 g at the left quarter point. The peak acceleration at 80% of the height was also 2.3 g. The motion of the dam shows higher frequency components than the downstream and base records, possibly because of higher mode contributions of the dam or impact caused by open contraction joints pounding closed. It was not possible for CDMG to digitize and process all the strong motion records because the traces became intertwined with the large acceleration peaks that exceeded the range of the instruments. Processed records are available for Channels 8 – 11 on the dam, the downstream site, and the left abutment site (see CSMIP Report Nos. 94–12A, 94–13, and 94–15A). Partial unprocessed records for Channel 1 – 6, 12, 13, and 15 – 17 are available in CSMIP Report 95–05.

In the 1971 San Fernando earthquake, the dam experienced strong motion close to the epicenter. The left abutment was damaged and several rockslides had to be repaired. An accelerometer above the abutment recorded a peak acceleration of 1.25 g, one of the largest recorded accelerations at the time. The topographic amplification in a canyon and the non–uniform support motion for an arch dam were recognized over twenty years ago in a study of Pacoima Dam in the San Fernando earthquake (Reimer, 1973). The computed base rock and dam accelerations from that study were remarkably similar to those in Northridge.

Examining the response of the dam in the 1994 Northridge earthquake, Fig. 3 shows the transmissibility function between radial motion at the base (Channel 9) and radial motion of the dam at 80% height (Channel 8). The transmissibility function is computed from estimates of the power spectral density functions. It indicates response peaks below 15 Hz at 4.0, 6.25, 8.25, 10.1, and 14.5 Hz. More analysis is required to identify vibration mode shapes from the limited processed strong motion data.

FINITE ELEMENT MODEL OF DAM

The finite element modeling concepts of ADAP–88 program are described in Fenves (1989). For the present study, the modeling was improved in three ways: (i) 3–D solid elements can now be used for the dam body instead of thick shell elements; (ii) nonlinear joint elements can be used for opening of lift joints and at the dam–foundation interface; and (iii) non–uniform free–field motion can be specified instead of the previous restriction to uniform ground motion (Mojtahedi and Tseng, 1994).

The finite element model used in the analysis of the Pacoima Dam is shown in Fig. 4. A total of 564 eight–node 3–D elements are used for modeling of the dam body. Four contraction joints are included in the model (the dam has eleven joints); three are located at the quarter points along the crest and the fourth joint represents the interface of the dam with its thrust block at left abutment. The four joints included in the model is adequate for representation of joint opening effects for uniform free–field ground motion; peak stresses can be expected to remain unchanged with increasing of the number of joints in the model. Three joint elements through the thickness are used for simulating opening–closing at the contraction joints and at the dam–foundation interface. This paper does not address opening of lift joints or cracking of the concrete, nor does it consider slippage at the contraction joints or dam–foundation interface. The material properties of the dam concrete were obtained from previous evaluations of the dam (based on core samples). The elastic modulus is 2400 ksi and Poisson's ratio is 0.20. The unit weight is 150 lb/ft^3, as used in a previous study of the dam (County, 1983).

For analysis with uniform free–field motion, a foundation rock region of depth equal to the height of the dam is included in the model to account for dam–foundation interaction effects. To suppress stresses caused by propagation of seismic waves, the foundation rock is assumed to be massless and ground motion is specified at the rigid base of the foundation model. For analyses with non–uniform motion, free–field ground motions are specified for nodes along the dam–foundation interface and the same foundation model which is included in the analysis for uniform seismic input, is used to determine the impedance matrix of the foundation rock domain. In view of complexity of the foundation geometry and uncertainty in the foundation material properties, a prismatic shape is assumed for the canyon and a coarse mesh is used for finite element modeling of the foundation. A total of 220 3–D solid elements are used in discretization of the foundation rock domain. The material properties used for foundation rock are: modulus of elasticity = 2000 ksi, Poisson's ratio = 0.20, again based on previous investigations.

The hydrodynamic pressure of the impounded water acting on the dam is represented by an added mass matrix neglecting compressibility of water (Kuo, 1982). Compressibility of water in a full reservoir can have an important effect on the earthquake response of arch dams (Fok and Chopra, 1987). However, the frequency analysis required to include compressibility cannot be combined with the time domain analysis required to account for the nonlinear joint opening effects. For computing the added mass for all analyses, the water is assumed to be bounded by a cylindrical surface obtained by translating the dam–reservoir interface in the upstream direction. A total of 280 8–node 3–D elements is used for computing the added mass of the partially full reservoir.

The first ten free vibration frequencies of the Pacoima Dam, considering dam–reservoir–foundation interaction effects, are: 4.3, 4.4, 6.2, 7.5, 7.8, 8.5, 9.1, 9.8, 10.4, and 11.3 Hz. The first two analytical modes appear to have a slightly higher frequency with the broad peak in Fig. 3 at 4.0 Hz. The analytical model also shows more distinct modes which are not separated in the transmissibility function in Fig. 3. Considering the uncertainties in the material properties, the correspondence is judged to be adequate. The limited processed data may make it difficult to obtain a closer correspondence between the identified and model vibration properties.

Rayleigh damping was assumed for the dam–foundation system with the parameters selected to produce 10% damping at 5 Hz and 20 Hz. The damping was selected after several trial solutions. It is an acceptable value considering that energy dissipation in not modeled in the foundation, there was slight damage to the dam body, and the broad peaks in the transmissibility function (Fig. 3).

DAM RESPONSE ASSUMING CLOSED JOINTS

Three cases are considered to examine the response of the dam with the contraction joints prevented from opening. This is the typical assumption used in the seismic safety evaluation of arch dams. For Cases 1 and 2 uniform free–field motion is specified for three components of the ground motion. Case 3 uses an assumed spatial variation of the ground motion to account for the topographic amplification. Case 1 uses the recorded acceleration from the downstream instrument (in the canyon). Case 2 uses the acceleration from the upper left abutment, reduced by one–third to represent the motion of the canyon. The free–field motion for the two cases is very different because of the topographic amplification of the canyon, but they have been proposed as bounds for the response of the dam to the actual spatially varying ground motion.

The problem of determining the spatial variation of the free–field motion in the canyon is difficult because it depends on complicated wave propagation effects for seismic waves emanating from the extended source and scattering on the canyon. The instrumentation for the dam is not sufficient to define the spatial variation of input motion, and even if it were sufficient, the recorded motion of the abutments would include dam–foundation interaction effects. Since the principal objective of this study was to examine the contraction joint behavior, a very simple assumption for the non–uniform motion is made. For Case 3, the free–field motion is based on an interpolation of the recorded motion at the dam base and the recorded motion at the left abutment. The interpolation function should represent the dynamic characteristics of the canyon. In this study a linear interpolation function is selected in lieu of a detailed model of wave propagation effects. Consequently, each component of the free–field motion at an elevation is a linear interpolation between the toe and left abutment recorded motions.

Figure 5 shows a comparison of the acceleration for Channel 8, the radial motion at about the 80% height in the left quarter of the dam (see Fig. 2 for the location). For Cases 1 to 3, the computed peak acceleration is reasonably close to the corresponding measured value. The computed histories for Case 3, however, differ in phase with the recorded motion.

Figure 6 shows the envelopes of maximum arch and cantilever stresses for Case 2 (positive is tension). All stress contour plots are viewed looking in the downstream direction. For Case 1 (not shown) with the uniform downstream motion the peak tensile stresses are 450 psi in the arch direction and 150 psi in the cantilever direction. For Case 2 with uniform motion based on the upper left abutment record, reduced by one–third, Fig. 6 shows that the peak tensile stresses more than double: 1100 psi in the arch direction (upstream) and 400 psi in the cantilever direction (downstream).

The stress distributions for the assumed non–uniform free–field motion Case 3 in Fig. 7 are very different than the stresses for the uniform free–field cases. The largest arch and cantilever stresses develop near the abutments, in contrast with Cases 1 and 2 for which the greatest stresses are at the center of the dam. The stress contours also have a very different pattern than the uniform ground motion cases. Very large tensile arch stresses, exceeding 3000 psi, are obtained at the upper right abutment. The cantilever stresses reach 1200 psi tension in the lower portion of the dam. The large stresses near the abutments are caused by the relative displacements of the non–uniform free–field motion at the interface; they tend to reduce towards the center of the dam where vibration contributes more to the stresses. From these results, the two uniform ground motion cases do not represent the response of the dam considering the assumed non–uniform ground motion.

DAM RESPONSE INCLUDING JOINT OPENING

As shown in Fig. 7, maximum tensile arch stresses of up to 1800 psi develop away from the abutment for Case 3 with the non–uniform free–field ground motion. Tensile stresses cannot be transmitted by the contraction joints, so it is expected that the joints will open during the earthquake preventing such tensile stresses from developing. To examine this issue, Case 4 allows the joints to open with the non–uniform free–field ground acceleration.

A comparison of the Channel 8 histories for Cases 3 and 4 (Fig. 5) shows that the agreement between the computed and recorded acceleration further improves when the joints are allowed to open. The main pulse in the recorded response is captured by the model with joint opening effects and non–uniform ground motion. After the main pulse, however, the analysis shows larger amplitude vibration than recorded. The overestimate of vibration is most likely caused by the lack of radiation damping in the foundation model.

As shown by the stress contours in Fig. 8 the maximum arch stresses in the dam body reduce to about 200 psi and there is a small increase in the cantilever stresses as seismic forces are transferred from arch action to cantilever action when the joints open. However, near the abutments the arch stresses are still large because of the pseudo–static effects of the relative input motion displacements. Figure 9 plots the joint opening history at the crest of each joint. The opening at the upstream and downstream faces are in–phase indicating complete separation of the joints. Also shown are the joint history plots for the pseudo–static joint displacements: the joint opening that would occur due to slow application of the free–field ground motion. Joint 1, near the right abutment, has the largest opening of 4 in., but most of that is due to the pseudo–static response. The vibration produces about 1 in. of the opening. Joints 3 and 4, the latter near the left abutment, have mostly pseudo–static opening. Joint 2 at the crown opens because of vibration, reaching a maximum opening of 3.5 in during the strong vibration response. Figure 10 shows the contours of maximum joint opening in each of the joints.

CONCLUSIONS

Examination of the limited CDMG processed records of Pacoima Dam in 1994 Northridge earthquake and comparison with analytical models indicates the following conclusions: (i) The contraction joints opened during the earthquake, and the effect of the joint opening is an important factor in the response, (ii) the non–uniform free–field ground motion caused by topographic amplification has a significant effect on the dam response; (iii) the computed response using uniform free–field ground motion does not provide an adequate representation of dam performance; (iv) damping due to foundation rock radiation appears to be important. The pseudo–static effects of the non–uniform ground motion cause high stresses. It is necessary to use more than three or four joints in the model to represent the effect of joint opening on the stresses, particularly near the abutments.

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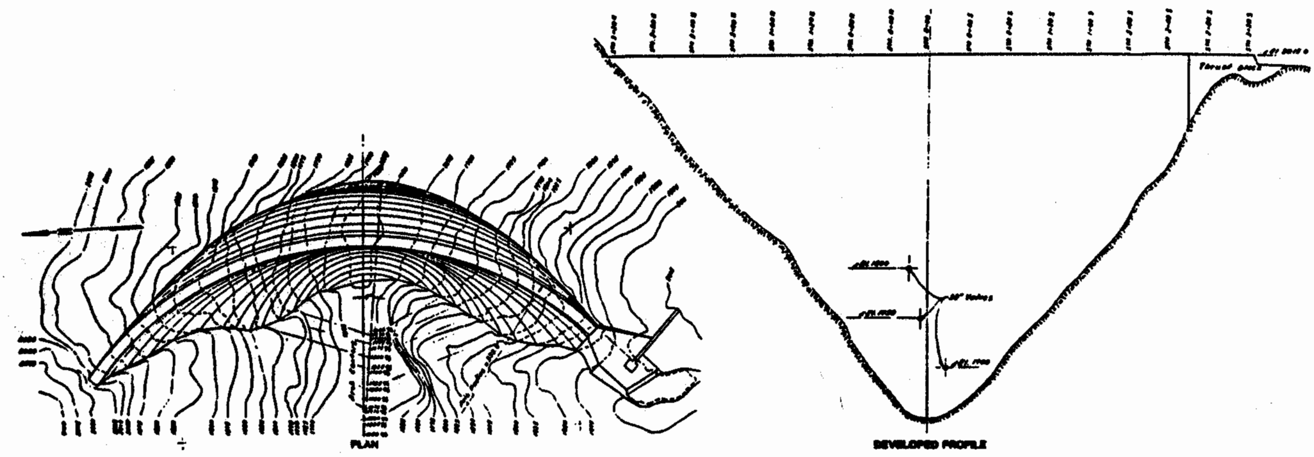


Fig. 1. Plan and Profile of Pacoima Dam (County, 1983).

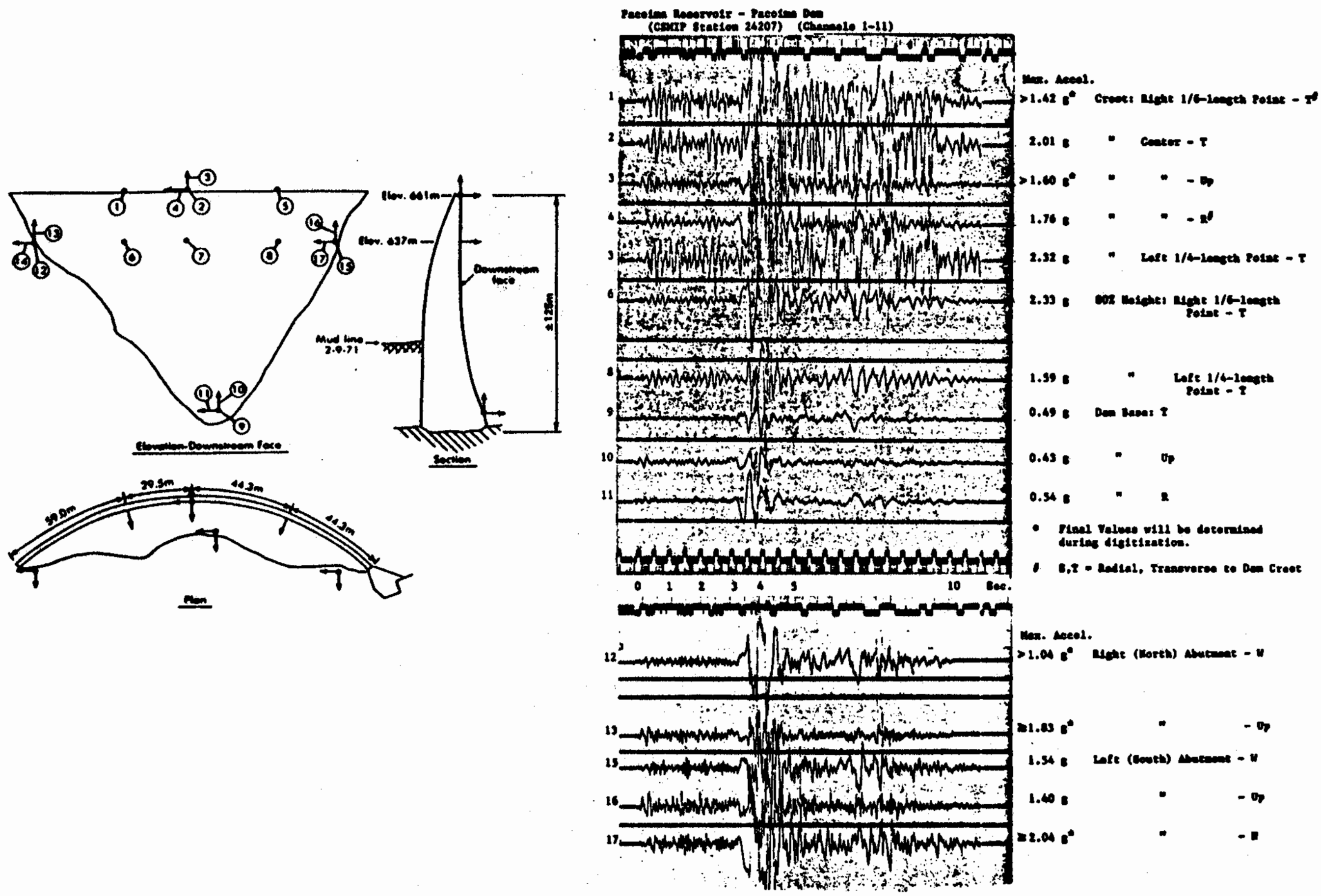


Fig. 2 CDMG Strong Motion Instrumentation for Pacoima Dam and Records from 1994 Northridge Earthquake (CSMIP, 1994).

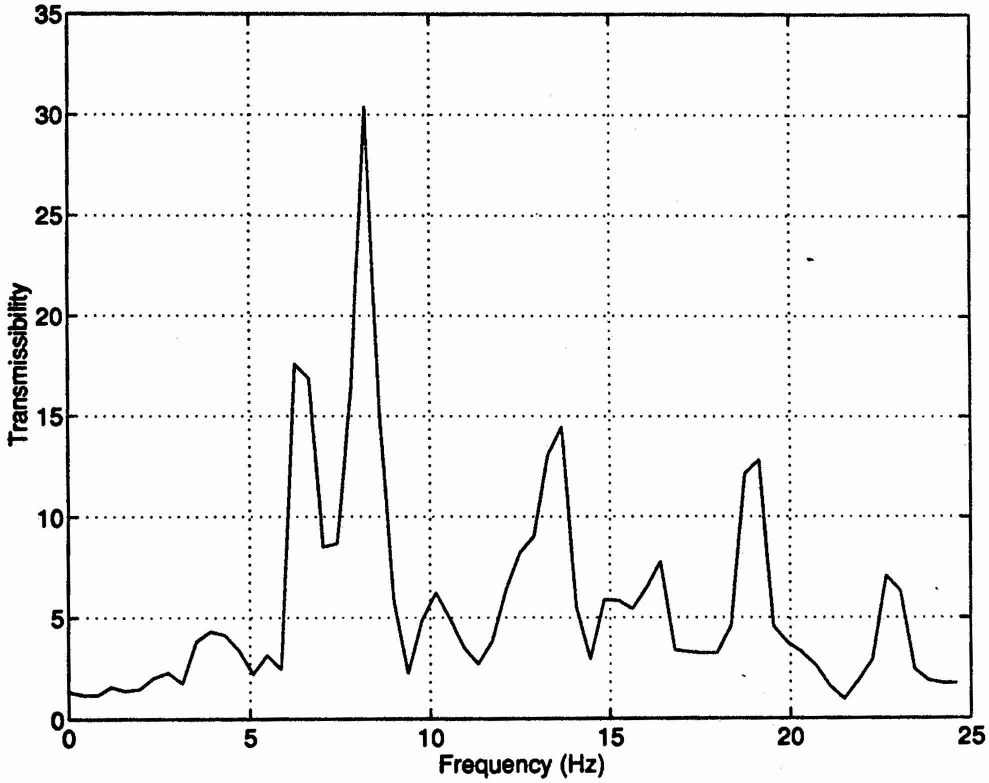


Fig. 3 Transmissibility for Radial Response of Pacoima Dam.

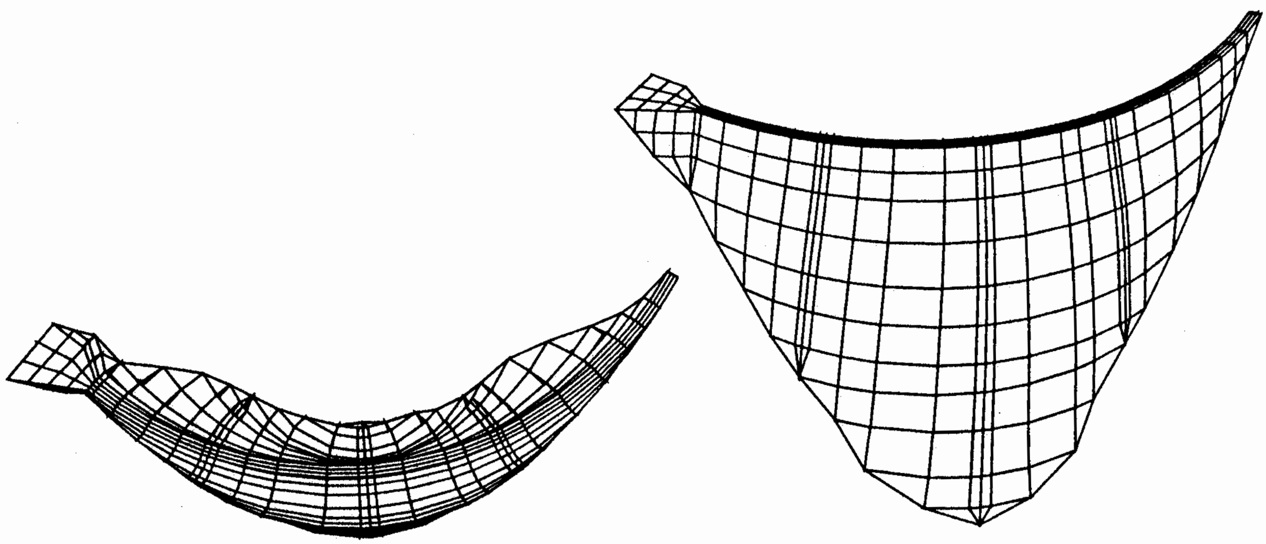


Fig. 4 Finite Element Model of Pacoima Dam Including Vertical Contraction Joints.

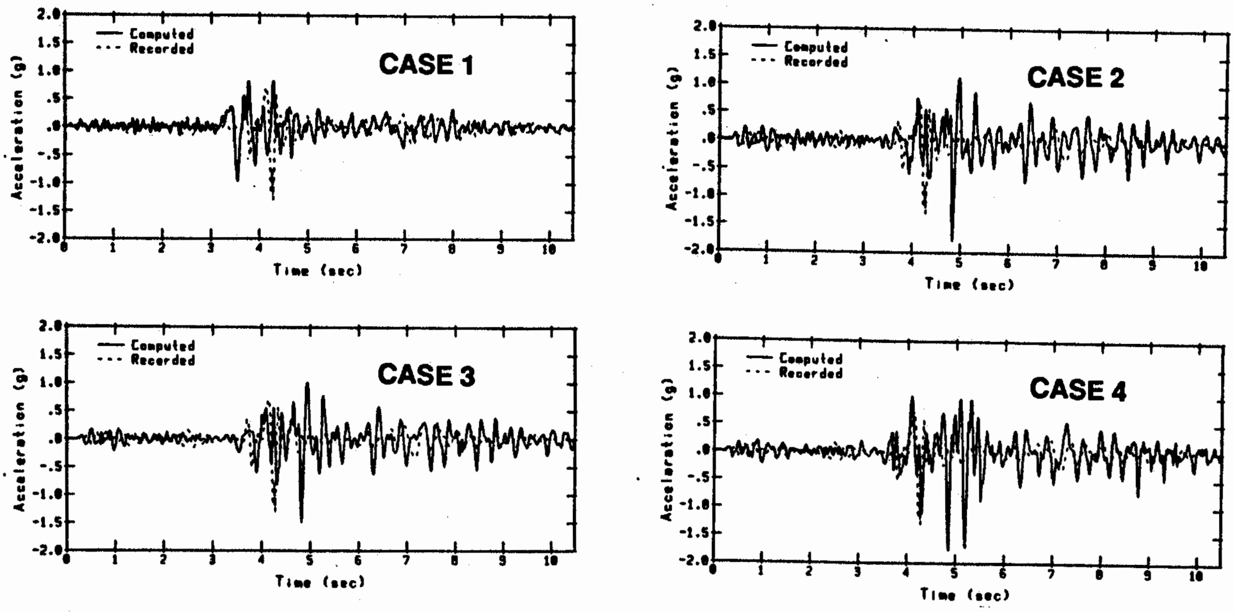


Fig. 5 **Computed Acceleration for Channel 8 for Case 1 (Uniform Downstream, Closed Joint), Case 2 (Uniform 2/3 Upper Left Abutment, Closed Joint), Case 3 (Non–uniform, Closed Joint), and Case 4 (Non–uniform, Open Joint), Compared with Recorded Acceleration.**

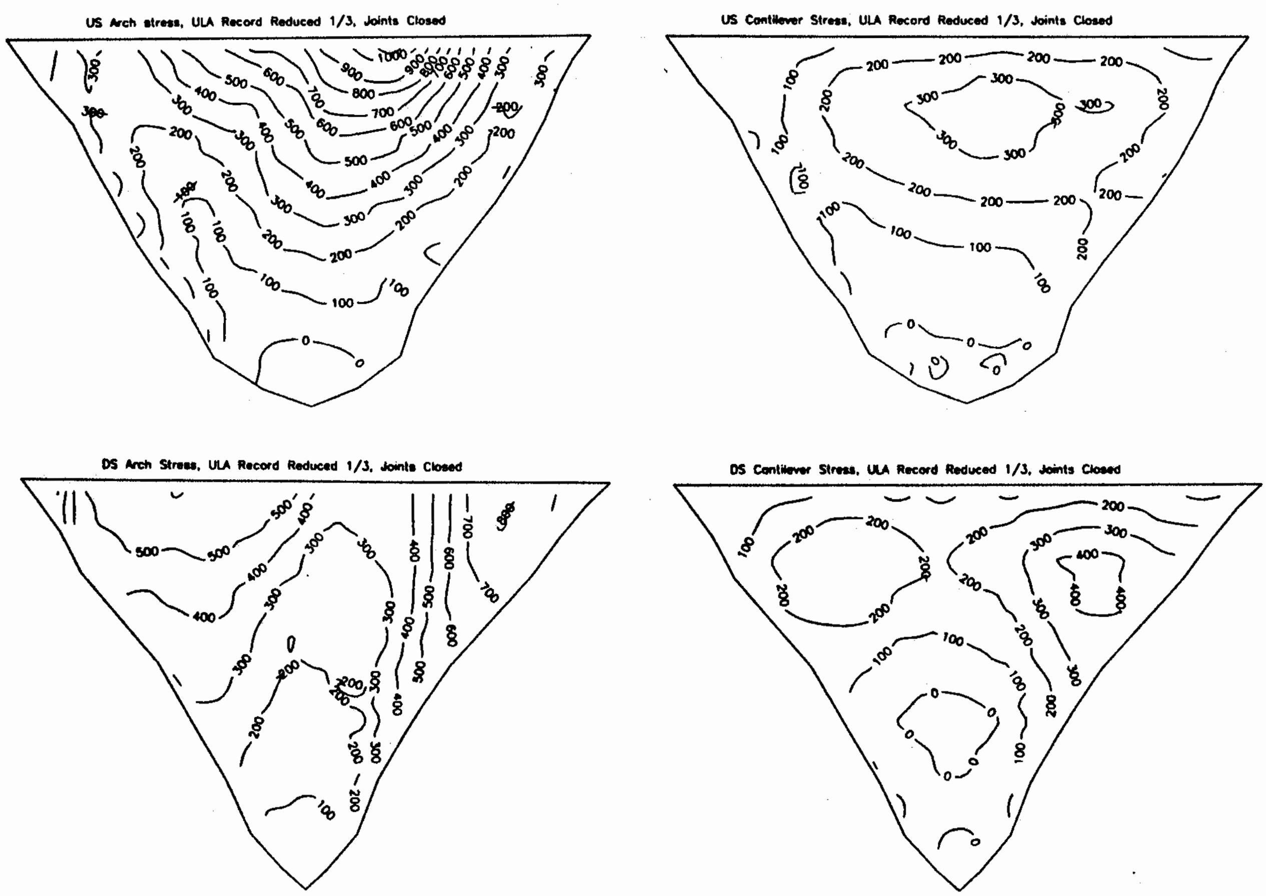


Fig. 6 **Contours of Maximum Principal Stress (in psi) for Closed Joint Model of Pacoima Dam. Case 2: Uniform Upper Left Abutment Free–field Ground Motion Reduced by One–Third.**

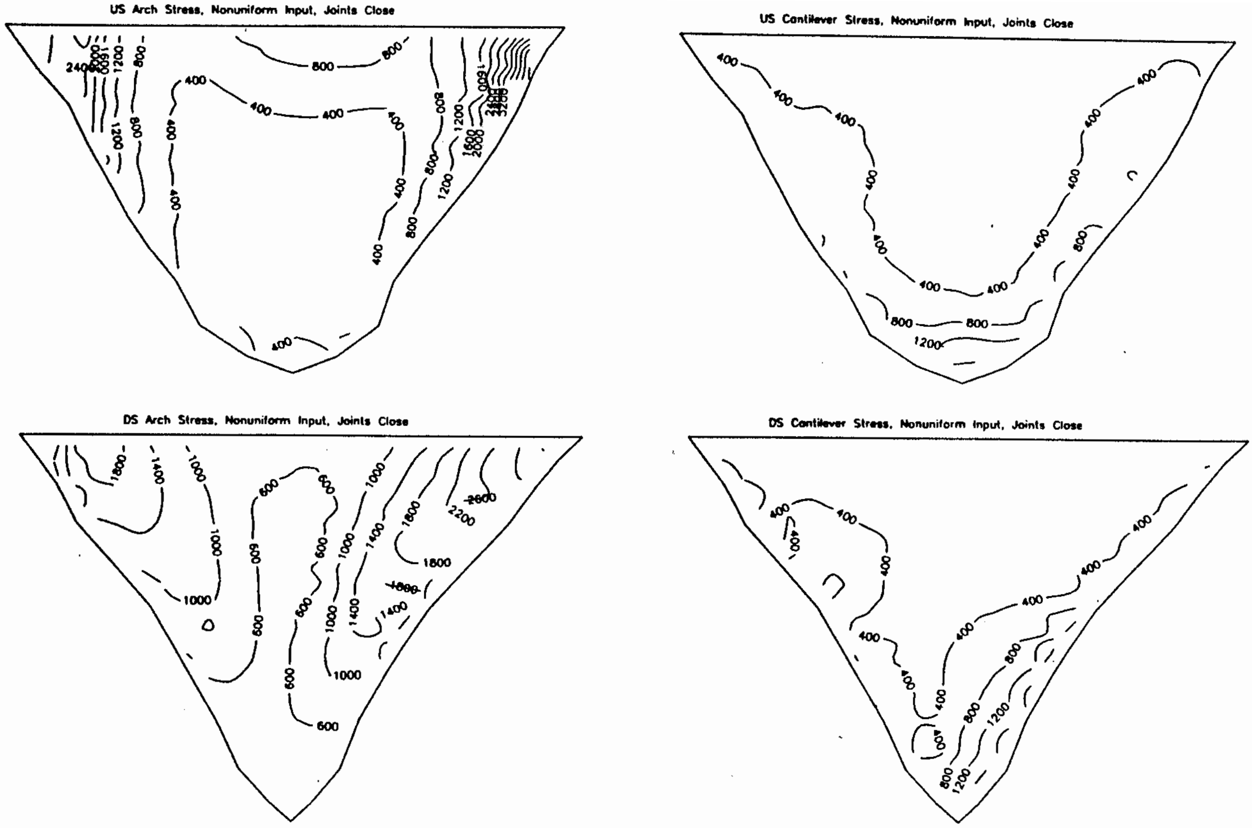


Fig. 7 **Contours of Maximum Principal Stress (in psi) for Closed Joint Model of Pacoima Dam. Case 3: Non–uniform Free–field Ground Motion.**

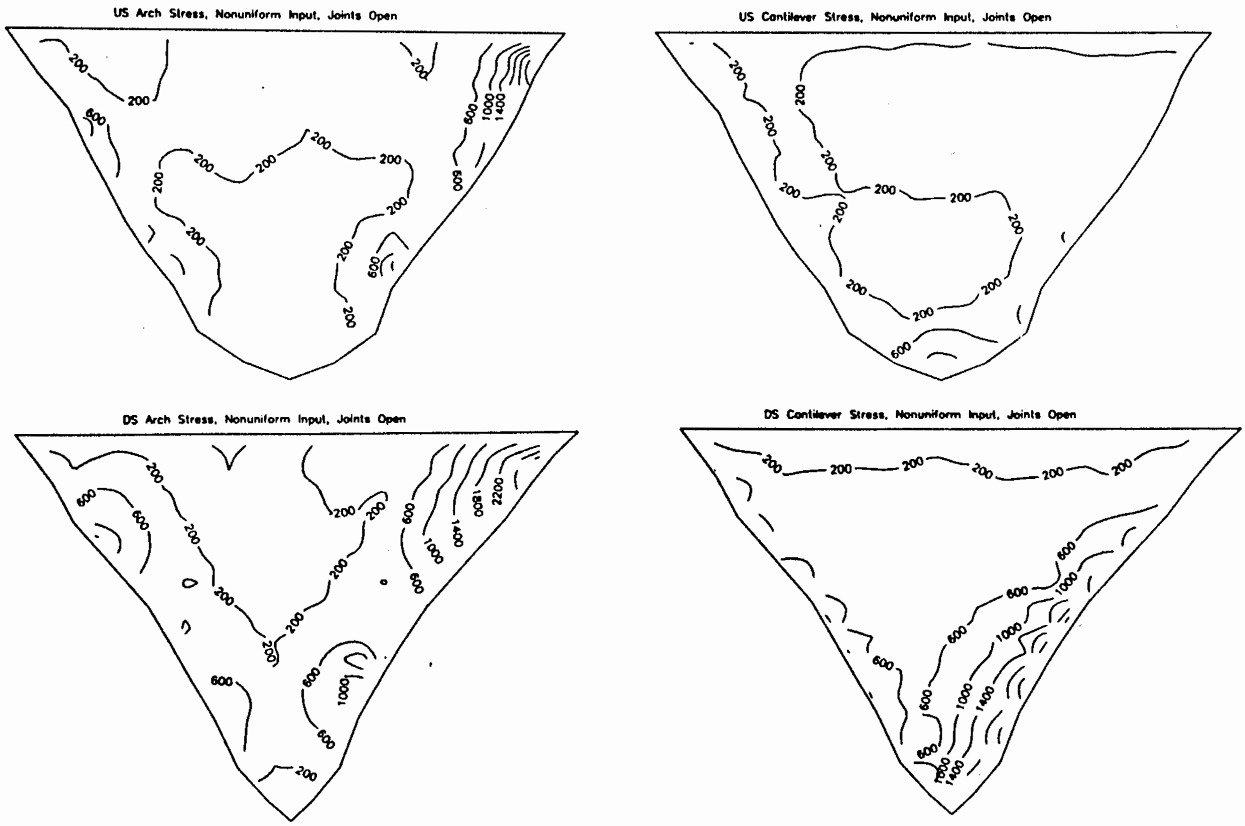
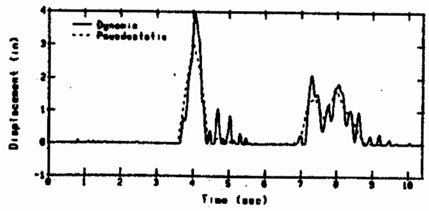
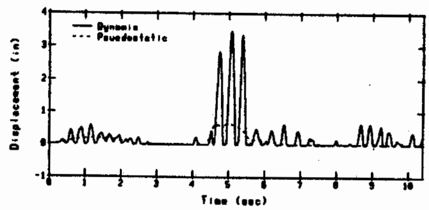


Fig. 8 **Contours of Maximum Principal Stress (in psi) for Open Joint Model of Pacoima Dam. Case 4: Non–uniform Free–field Ground Motion.**



JOINT 1



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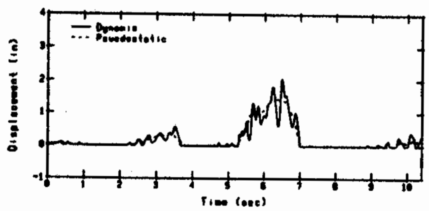

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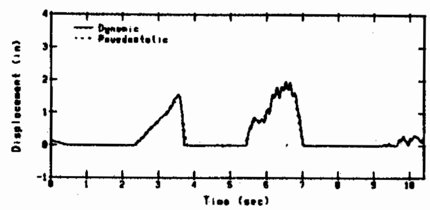
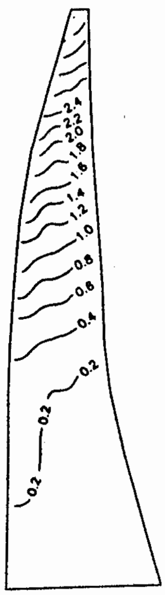


JOINT 4

Fig. 9 History of Joint Opening Displacement for Case 4, Open Joint Model and Non–uniform Free–field Ground Motion.

JOINT 1



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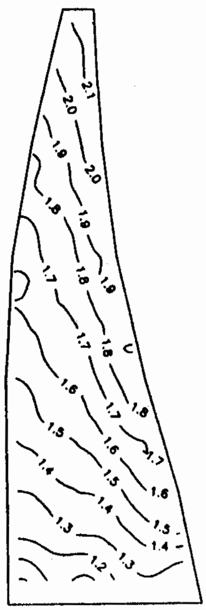

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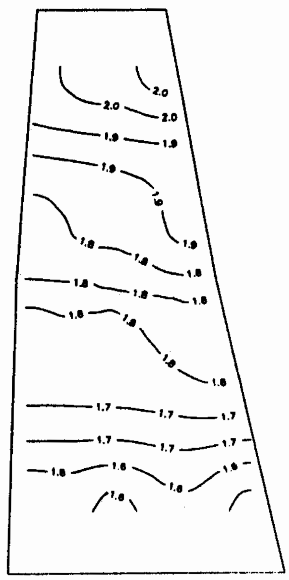

JOINT 4

Fig. 10 Contours of Maximum Joint Opening Dispalcement (in inches) for Case 4, Open Joint Model and Non–uniform Free–field Ground Motion.