13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1115

FAILURE MODES APPROACH TO SAFETY EVALUATION OF DAMS

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SUMMARY

Seismic performance of concrete dams is currently conducted on the basis of stress checks combined with engineering judgments. The paper introduces a failure modes approach for assessment of seismic safety of dams. Although this approach still uses magnitudes of stresses as a cursory measure of the performance, safety of the dam is assessed on the basis of potential modes of failure that could occur. The goal of the seismic safety evaluation is therefore to conduct appropriate analyses and evaluations that could demonstrate whether or not certain failure modes can develop.

The paper first discusses overstressing, sliding, joint opening, and other modes of failure that could affect stability of a concrete dam and its foundation. It then proposes new performance evaluation criteria that demonstrate whether overstressing means some joint opening and cracking or could lead to failure. The proposed performance evaluation involves both linear-elastic and nonlinear analyses including the dam-water and dam-foundation interaction effects. Also built into the evaluation process is the rational that sensitivity analyses may be required to account for the effects of many modeling, material, and seismic input assumptions. The acceptance criterion is therefore not based on stress checks alone; rather it examines stress demand-capacity ratios, accumulated duration of overstress excursions, spatial distribution of stresses, and other factors to determine whether or not nonlinear response in the form of cracking and joint opening could lead to failure mechanisms. With respect to sliding failures, reference is made to potential sliding in the dam, at the dam-foundation interface, or in the foundation and abutment; these may require response history analyses leading to estimation of the cumulative sliding displacement. Finally two examples are provided to validate the proposed performance criteria and to illustrate its application to arch dams.

INTRODUCTION

Seismic safety of concrete dams is currently assessed on the basis of simple stress checks from the linear elastic analysis combined with engineering judgment, see NRC [1]. The acceptance criterion for compressive stresses is that they should be less than the compressive strength of the concrete by a factor of 1.5 for new designs, see USACE [2], and 1.1 for existing dams, see FERC [3]. These and other guidelines generally require that tensile stresses be less than the tensile strength of the concrete, otherwise cracking would occur. In practice up to five stress excursions above the tensile strength of the concrete

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have been considered acceptable based on engineering judgment and other considerations. This criterion neither puts limit on the magnitudes of stresses exceeding the tensile strength of the concrete nor offers any provisions regarding the spatial extent of such stresses. Rather it is left to experts or the analyst to judge how high the magnitudes of critical tensile stresses could reach and how large an area they could occupy.

In an attempt to overcome these shortcomings, this paper introduces a systematic approach for assessment of the seismic performance and probable level of damage using linear-elastic time history analyses and consideration of potential modes of failure. First potential modes of failure and observed performance for three types of concrete dams are described, and then analysis and new evaluation procedures for assessment of dam safety are described. The performance evaluation is geared toward assessment of the probable level of damage that may lead to failure of the dam. It is formulated based on magnitudes of demand-capacity ratios, cumulative duration of stress excursions beyond the tensile strength of the concrete, spatial extent of overstressed regions, and other considerations. Finally two examples are provided to validate the proposed performance criteria and to illustrate its application to arch dams.

POTENTIAL FAILURE MODES OF CONCRETE DAMS

The performance of a dam can be threatened by natural phenomena such as floods, rockslides, earthquakes, and deterioration of the heterogeneous foundations and construction materials. In this paper only potential failure modes due to earthquake shaking are considered. Analysis of the performances of the various types of concrete dams shows certain failures are more likely to occur under earthquake loadings. These are briefly discussed below for three common types of concrete dams, followed by analyses and evaluation procedures that enable the analyst to assess dam safety on the basis of potential modes of failure rather than the magnitudes of stresses alone.

Gravity Dams

Three major potential modes of failure of gravity dams include: 1) overstressing, 2) sliding along cracked surfaces in the dam or planes of weakness within the foundation, and 3) sliding accompanied by rotation in the downstream direction. A gravity dam may collapse in one or more sections which have been overstressed or their resistance against sliding and/or rotation has been exhausted. Many studies have shown that under severe ground shaking a typical gravity dam section may suffer tensile cracks at the base and/or near the upper change of slope. The upper cracks usually initiate from the upstream or downstream face of the dam and propagate horizontally or at an angle toward the opposite face. The consequence of cracking, if extended through the dam section, may lead to sliding or rotational instability of the separated block. The seismic performance of Koyna Dam during the 1967 Koyna earthquake, India, can be cited as an example, see Chopra [4]. The Koyna shaking induced horizontal cracks on both faces of the tallest non-overflow blocks at the elevation of the downstream change of slope. However, the dam did not fail and there was no flooding downstream. Subsequently, the dam was strengthened by the addition of buttresses on the downstream face of the non-overflow blocks. This observed performance clearly points to the type of overstressing failure that could occur in a gravity dam.

The prevailing mode of failure for gravity dams is probably sliding along the base of the dam or along planes of weakness within the foundation. Many dams have failed where the sliding hazard was ignored or given inadequate attention. Although, none of such failures was caused by earthquakes, an intense earthquake capable of breaking the bond between the foundation and concrete or overstressing joint sets and planes of weakness in the foundation can promote sliding or overturning of the dam. The possibility of sliding mode of failure must therefore be taken into account in the safety assessment.

Buttress Dams

The seismic response of a slab and buttress or multiple arch dam has many similarities to that of an arch dam because the system geometry is three-dimensional and it responds to three components of seismic input. The most important difference in a buttress dam response from that of an arch lies in the sensitivity of the buttresses to cross-stream earthquake accelerations. The design of older buttress dams generally has considered only the gravity and water pressure loads, and the buttress configuration is remarkably efficient in providing the resistance required for such loading. However, in the interest of efficiency, the buttresses were made very slender and thus they had very little strength for resisting cross-stream accelerations. Under strongest shaking, it is conceivable that an older slab and buttress or multiple arch dam designed in this manner may suffer significant cracking and fall in domino fashion through the successive collapse of its buttresses. The collapse of Vega de Tera Dam in northwestern Spain exemplifies such a failure mechanism, even though it was not caused by earthquake, see Jansen [5]. This 112-foot high slab-and-buttress structure collapsed suddenly during the night of 10 January 1959, due to poor construction practice that had resulted in poor bonding of the masonry used in the buttresses (Fig. 1). Failure was said to have started on a sloping foundation near the abutment at a joint between the masonry buttress and concrete slab, followed by the collapse of 17 buttresses in succession (Fig. 2).

Fig. 1 Vega de Tera Dam before collapse

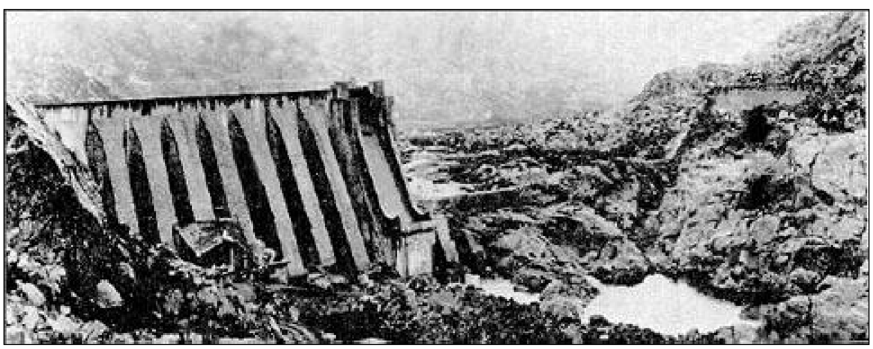


Fig. 2 Vega de Tera Dam after collapse

The two recent buttress dams with some seismic provisions that have suffered significant earthquake damage are Hsinfengkiang in China and Sefidrud in Iran. Hsinfengkiang Dam, see Shen [6], was completed in 1959. It is a diamond-head buttress dam 344 ft tall and has 19 buttress blocks in the central portion with gravity section located on either side. On 19 March 1962, Hsinfengkiang Dam was shaken by a magnitude 6.1 earthquake located very close to the dam and developed horizontal cracks at a change of section in the non-overflow blocks on each side of the spillway. Subsequent analysis demonstrated that cracking was to be expected at this location on the blocks during an earthquake of this intensity, and the dam was then strengthened so that it could resist even more severe earthquakes in the future.

Sefidrud Dam, see Ahmadi [7], a gravity buttress dam completed in 1962 in the northern Iranian province of Gilan, is 1,394 ft long and 348 ft high. It includes 7 gravity monoliths and 23 massive head buttress units, with heads 46 ft wide and buttress webs 16.4 ft thick. It was designed using a seismic coefficient of 0.25 g, but on 21 June 1990 it was damaged by the magnitude 7.3 Manjil Earthquake with the epicenter at less than 20 miles but the fault rupture much closer to the dam. The principal damage to the central monoliths was cracks at lift joints extending from the dam face through the buttress face and web. These occurred adjacent to a change of slope near the crest of the dam, and were accompanied by a 0.8-in shear displacement toward downstream. Although no catastrophic release of the reservoir occurred and the stability of the dam was not a matter of concern, severe leakage through the cracks led to lowering of the reservoir. Subsequently the dam was repaired using epoxy-grouting for water tightness with post tensioning strands to restore shear strength in the cracked sections. It is interesting to note that Hsinfengkiang diamond-head buttress dam and Sefidrud gravity buttress dam were able to resist cross-stream accelerations and that the observed cracking in the upper portion of the dams resembles performance similar to that of gravity dams discussed previously.

Arch Dams

Arch dams are usually built as independent cantilever blocks separated by vertical contraction joints. Contraction joints may have keys to provide increased shearing resistance; thus when the joints and keys are grouted, the structure will act as a monolithic system. The cantilever blocks are built by placing mass concrete in lifts. The integrity and strength of the concrete therefore depends in a large extent to the proper preparation of construction joints before placing fresh concrete upon the lift line surfaces.

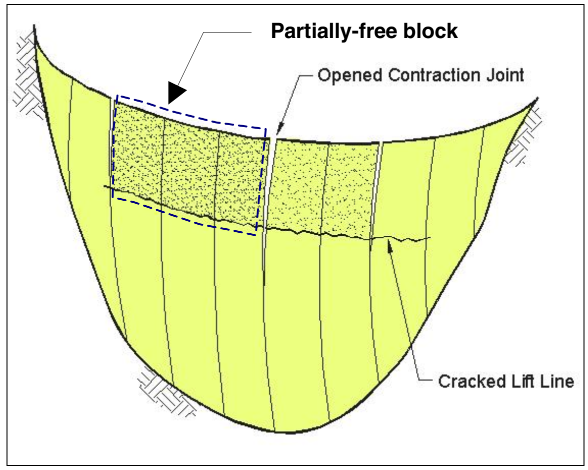


Fig. 3 Free blocks created by opened joints and cracked lift lines

Potentially an arch dam may fail as a result of: 1) excessive contraction joint opening combined with cantilever tensile cracking, 2) movements of the abutment rock wedges formed by rock discontinuities, and 3) in certain cases sliding along the gently sloped dam-abutment interface. Vertical contraction joints are known to possess very little or no tensile resistance and may repeatedly open and close during intense earthquake shaking when seismic tensile arch stresses exceed the static compressive arch stresses. The contraction joint opening releases tensile arch stresses but increases tensile cantilever stresses. The increased cantilever stresses may exceed tensile strength of the concrete or lift lines, causing horizontal cracks. The resulting partially-free blocks bounded by the opened contraction joints and cracked lift lines may become unstable leading to failure of the dam, see Fig. 3.

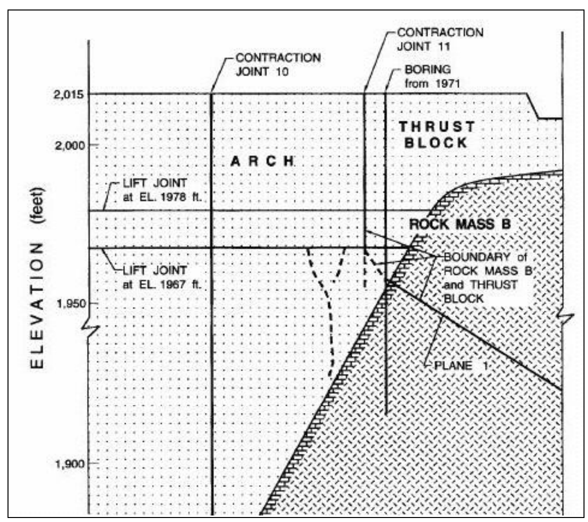
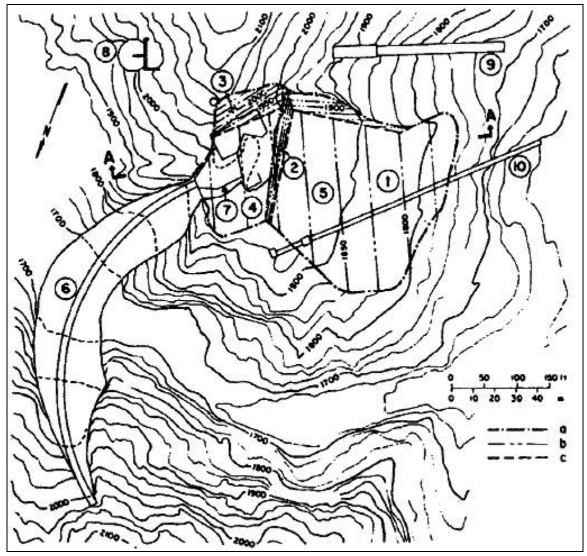


Fig. 4 Downstream view of the left part of Pacoima Dam and thrust block showing cracking

Another failure mode especially critical to the stability of arch dams involves abutment rock wedges that are kinematically capable of movements. The abutment rock movements and contraction joint opening were observed at the 365-ft-high Pacoima Arch Dam during the 1971 San Fernando Earthquake and the 1994 Northridge Earthquake. In 1994 the contraction joint between the arch dam and thrust block on the left abutment opened 2 inches at the crest level (Joint 11 in Fig. 4). The opening continued downward, decreasing to ¼ of an inch 60 feet below the crest level. After passing through two cracked lift lines at Elevations 1978 and 1967, see Fig. 4, the opening connected to a diagonal crack extended down diagonally through the lower part of the thrust block to meet the abutment rock. Other contraction joints also opened but to a lesser degree. Apparently the diagonal crack in the lower part of the thrust block was the extension of a slip plane beneath Rock Masses A and B, see Plane 1 in Figs. 4 and 5. The post-earthquake surveys indicated that Rock Mass B slipped about 2 to 3 inches horizontally and 2 inches down, thereby accounting for the opening in the contraction joint between the dam and thrust block. While Rock Mass A moved 16 to 19 inches horizontally and up to 14 inches down, which caused a separation of 1 to 1.5 feet between Rock Masses A and B (Plane 2 in Fig. 5). It appears the 35 tendons installed after the 1971 earthquake have played a significant role in limiting the movement of Rock Mass B during the 1994 shaking. Note that during the Northridge earthquake the water surface was 131 feet below the crest. Nevertheless, the observed damage indicates that the worst failure scenario for the dam will involve the upper part of the dam, either originating from a sliding of the rock masses A and B on the left abutment or excessive joint opening and cracking leading to unstable free blocks.



a Failure plane surface trace	4 Rock mass “B”
b Contours in failure plane	5 Rock mass “A”
c Intersection of failure planes	6 Top of dam
1 Plane 1	7 Trust block
2 Plane 2	8 Spillway intake
3 Plane 3	9 Spillway

Fig. 5 Plan view of Pacoima Dam & left abutment (County of Los Angeles [7])

In other situations, excessive cantilever stresses near the dam-foundation region may develop cracking at the base of the dam or relieved through movements and joint opening of the fractured foundation rock below. In either case the tensile resistance capability of the dam in this region will be lost. If the cracked or failed foundation region is small the dam may remain stable by bridging over the region. However, large cracked or failed regions, especially on gently-slopped abutments may trigger sliding of the cracked cantilever blocks that are capable of resisting forces by compression and shear, but not tension. In this situation the possibility of sliding of the dam must be taken into account in assessing safety of the dam.

METHOD OF ANALYSIS

Seismic safety evaluation of concrete dams relies heavily on the results of numerical procedures. For this reason appropriate methods of analysis and evaluation are required to accurately predict potential failure modes. Safety evaluation for intense and damaging ground shaking requires the use of time-history method of analysis with careful consideration of the dam-water and dam-foundation interaction mechanisms and reasonable selection of the material properties, seismic input, and load combinations. Sensitivity of the dam response to critical modeling and parameter assumptions is another important factor that needs to be considered as part of the overall safety evaluation process. This can be accomplished by varying critical parameters and using computer programs having different analytical capabilities and assumptions.

Concrete dams are made of plain concrete that possesses limited ductile behavior. This behavior is characterized by a stress-strain relationship composed of elastic and inelastic strain ranges followed by a complete loss of strength. The inelastic-strain range, designated as damage control range in this paper, provides only limited inelastic behavior, see Fig. 6. As discussed later, a linear-elastic time-history analysis combined with a systematic performance evaluation criteria can be used to assess the dam response in the damage control range. The dam response beyond the damage control range is governed by complete loss of strength, sliding, and nonlinear response behavior of discrete blocks bounded by opened joints and cracked sections. This behavior must be evaluated using nonlinear time-history analysis.

Both linear and nonlinear analyses should be based on mathematical models that adequately capture dynamic characteristics of the dam-water-foundation system. This requires accurate modeling of the dynamic response and interaction mechanisms as well as mass and stiffness distributions. Mathematical models of existing dams should account for the effects of existing cracks, deteriorated concrete, or any deficiencies that might affect the stiffness. Depending on its significance, the effects of dam-water interaction may be modeled by simple added mass coefficients, or by a more elaborate solution that includes the effects of water compressibility and boundary absorption. Idealization of foundation rock may range from a simple massless model to a more elaborate formulation involving both the inertia and damping properties of the foundation. Sliding of the abutment and foundation wedges impacting the dam response may require a coupled nonlinear analysis which includes both the dam and wedges.

Gravity Dams

Relatively long and straight concrete gravity dams are usually idealized using a 2-D finite-element model, but curved gravity dams and those built in narrow canyons generally require 3-D models. The 2-D dam-water-foundation model may be analyzed as three separate systems using the substructure method, see Fenves [9], or as a single composite model using the standard finite-element procedures. Idealization of the heterogeneous foundation rock as a homogenous, isotropic, viscoelastic half-plane should be fully examined to ensure that it produces realistic results with no artificial damping. A strong dynamic coupling between the dam and water may dictate a dam-water interaction model with water compressibility and reservoir bottom absorption effects. However, applicability of an idealized fluid domain of constant depth and infinite length and of the reservoir-bottom absorption coefficient should be checked against the actual reservoir geometry and sediment conditions. The standard composite model with a massless foundation and simple added mass coefficients can be used as a baseline case to assess the effects of the heterogeneous rock properties. Planes of weakness within the foundation may form sliding surfaces or wedges affecting stability of the dam. These should be included as part of the finite-element model of the dam-foundation system to assess sliding stability of the dam.

Arch Dams

The complicated 3-D geometry of an arch dam requires a 3-D model of the dam, its foundation, and the impounded water for evaluation of its response to three components of the earthquake input. The arch dam-water-foundation system may be analyzed in the time domain using the standard finite-element procedures, see Ghanaat [10] or in the frequency domain using the substructure method, see Fok [11]. The standard method employs a massless foundation rock modeled as part of the dam finite-element mesh in conjunction with an incompressible liquid mesh representing the impounded water. Treating each system separately, the substructure method considers a dam model same as the standard method, a foundation model with the flexibility as well as the damping and inertia effects, see Tan [12], and a reservoir water model that includes water compressibility and reservoir boundary absorption effects. The standard method provides reasonable results for dams built on competent rock foundations having a deformation modulus equal or greater than that of the mass concrete and the impounded water having fundamental resonance frequency greater than twice the frequency of the dam alone. Otherwise, a more rigorous treatment of the dam-water-foundation interaction effects may be required, in which case all cautionary remarks regarding idealization of the dam-water and dam-foundation interaction effects discussed for gravity dams also apply to analysis of arch dams. In situations where movements of abutment wedges are coupled with movements of the dam, the wedge and the dam should be analyzed together as a coupled system.

PROPOSED PERFORMANCE AND DAMAGE EVALUATION

The linear time-history analysis is used to formulate a systematic and rational methodology for assessment of performance and qualitative estimate of the probable level of damage. In the linear time-history analysis, dam deformations and stresses are computed using mathematical models described in the previous section. Using acceleration time-histories as the seismic input, the linear time-history analysis computes both the magnitudes and time-varying characteristics of the seismic response. A systematic interpretation and evaluation of these results in terms of the stress demand-capacity ratios, cumulative overstress duration, spatial extent of overstressed regions, and other considerations form the basis for an approximate and qualitative estimate of damage. This evaluation is applied to the damage control range of strains shown in Fig. 6. If the estimated level of damage falls below the acceptance threshold for a particular dam type, the damage is considered to be low to moderate and the linear time-history analysis will suffice. Otherwise the damage is considered to be severe, requiring a nonlinear time-history analysis to determine whether or not it would lead to failure of the dam.

Load Combination Cases

Earthquake performance of concrete dams is evaluated for three or more sets of earthquake ground motions. For each set of two- (gravity dams) or three-components (arch dams) ground motions the effects of static loads and earthquake ground motions components are combined by multiplying each earthquake component by +1 or -1 to account for the most unfavorable direction of earthquake attack.

Demand-capacity Ratios

The demand-capacity ratio (DCR) for gravity dams is defined as the ratio of the calculated principal stress to tensile strength of the concrete. For arch dams where high stresses usually oriented in the arch and cantilever directions, DCR refers to ratio of the calculated arch or cantilever stress to the tensile strength of the concrete, but it can also be developed for the principal stresses. The tensile strength of the plain concrete used in computation of DCR is obtained from the uni-axial splitting tension tests or from

$$f_t = 1.7\, f_c'^{\,2/3}$$

proposed by Raphael [13], where f_c' is the compressive strength of the concrete. The maximum permitted DCR for linear analysis of dams is 2. This corresponds to a stress demand twice the static tensile strength of the concrete. As illustrated in Fig. 6, the stress demand associated with a DCR of 2 is the same as the so called "*apparent dynamic*" tensile strength of the concrete, a quantity proposed by Raphael for evaluation of the results of linear dynamic analysis.

Cumulative Inelastic Duration

The main problem with the traditional stress criterion is that the number of stress cycles alone is not adequate to assess damage. For example, the upper stress history in Fig. 7 includes fewer than 5 cycles exceeding the tensile strength of the concrete (i.e. DCR >1). Yet the damage potential of this stress history is by far greater than the lower stress history in the same figure, where more than five stress cycles exceed tensile strength of the concrete. Both magnitudes and duration of overstress cycles in the upper stress history are greater than those of the lower stress history, a factor that the number of cycles alone cannot show. For this reason the proposed damage criteria employ cumulative inelastic or overstress duration, which is a measure of energy and accounts for the magnitudes, as well as duration of stress excursions.

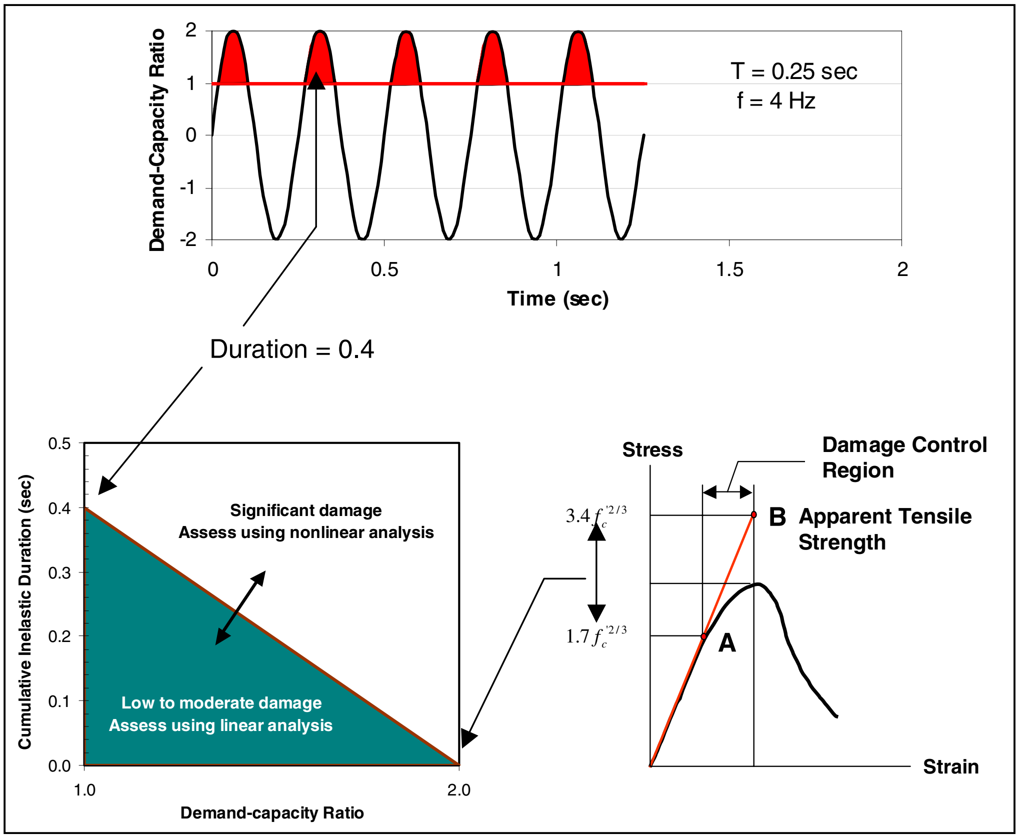


Fig. 6 Illustration of seismic performance and damage criteria

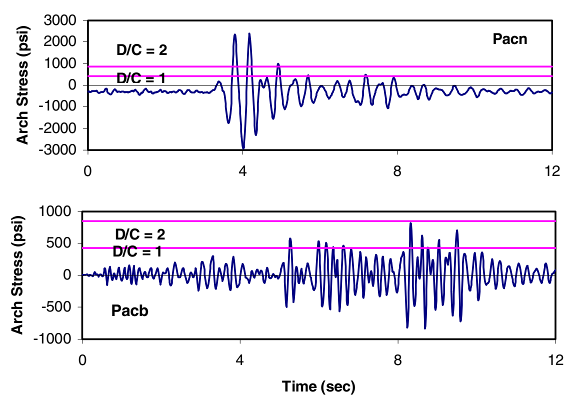


Fig. 7 Stress cycles exceeding D/C of 1 and 2

The cumulative inelastic duration of stress excursions is defined in Fig. 6. It refers to the total duration of stress excursions above a stress level associated with a DCR ≥ 1. For example, a cumulative duration of 0.4 sec at DCR = 1 indicates the total duration of stress excursions above the tensile strength of the concrete, see Fig. 6. Similarly, a cumulative duration of 0.2 sec at DCR = 1.5 is the total duration of stress excursions above a stress level 1.5 times the tensile strength of the concrete. The cumulative inelastic duration may be obtained approximately by multiplying number of stress points exceeding a certain stress level by the analysis time-step. The higher the cumulative duration, the higher is the possibilities for more damage. For arch dams, the allowable cumulative duration is taken equal to the duration of five harmonic stress cycles having a magnitude twice the tensile strength and an oscillation period equal to 0.2 seconds (Fig. 6). This results in a cumulative duration of 0.4 seconds for a DCR of 1. The cumulative duration for a DCR of 2 is assumed zero. For gravity dams a lower cumulative duration of 0.3 is assumed, mainly because gravity dams resist loads by cantilever mechanism only, as opposed to arch dams that rely on both the arch and cantilever actions.

Performance Criteria for Gravity Dams

The earthquake performance of gravity dams is evaluated on the basis of load combination cases, DCR's, and the cumulative duration described above. The performance is formulated for the maximum design earthquake (MDE). The MDE is defined as the maximum level of ground motion for which a structure is designed or evaluated. For dams the MDE is usually taken equal to the maximum credible earthquake (MCE). Three performance levels are considered:

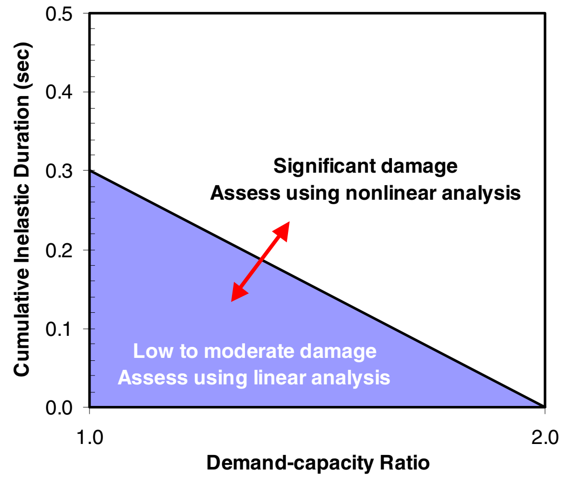


Fig. 8 Performance/damage acceptance for gravity dams

1. *Minor or no Damage*. The dam response is considered to be within the linear elastic range of behavior with little or no possibility of damage if DCR $\leq$ 1.

2. *Acceptable Level of Damage*. The dam will exhibit nonlinear response in the form of cracking and joint opening if the estimated DCR > 1. The level of nonlinear response or cracking is considered acceptable with no possibility of failure if DCR < 2, overstressed regions are limited to 15 percent of the dam cross-section surface area, and the cumulative duration of stress excursions for all DCR's between 1 and 2 falls below the performance curve given in Fig. 8.

3. *Severe Damage*. The damage is considered severe when DCR > 2, or cumulative overstress duration for all DCR's in the range of 1 to 2 falls above the performance curves given in Fig. 8. In these situations a nonlinear time-history analysis may be required, especially if the fundamental period of the dam falls in ascending region of the response spectra.

Performance Criteria for Arch Dams

The earthquake performance of arch dams is evaluated on the basis of combined static and seismic stresses in accordance with the load combination cases, demand-capacity ratios, and the cumulative inelastic duration described above. The proposed arch dam performance levels for the MDE include:

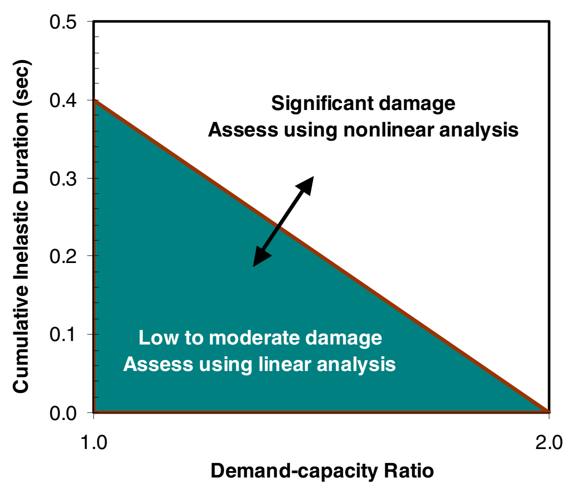


Fig. 9 Performance/damage acceptance for arch dams

1. *Minor or no Damage*. The arch dam response is considered to be within the linear elastic range of behavior with little or no damage if computed stress DCR's are less than or equal to 1. At this level of demands the contraction joint may still open, but the amount of opening will be small with no effects on the overall performance of the dam.

2. *Acceptable Level of Damage*. If estimated DCR's exceed 1.0, the arch dam exhibits nonlinear response in the form of contraction joint opening and possibly tensile cracking at the lift lines and elsewhere. The amount of joint opening and cracking are considered acceptable if stress DCR < 2, overstressed region is limited to 20 percent of the dam upstream or downstream surface area, and the cumulative duration of stress excursions for all DCR's

in the range of 1 to 2 falls below the performance curve given in Fig. 9. As discussed previously, the relation between the fundamental period of the dam and peak of the response spectra should also be considered to determine whether the nonlinear response behavior would increase or decrease the seismic demands.

3. *Severe Damage*. If the above performance criteria are not met, or met marginally and seismic demands would increase due to nonlinear behavior, then a nonlinear analysis is required to better estimate the level of damage and possible failure mode.

VALIDATION OF PERFORMANCE AND DAMAGE CRITERIA

Pacoima and Morrow Point Dams were analyzed to validate the proposed performance and damage criteria discussed above. Both dams were evaluated for six sets of earthquake acceleration time histories covering a wide range of ground motion parameters including frequency content, duration, pulse types and pulse sequencing. For each set the effects of static loads and earthquake ground motion components were combined as described previously. The linear analyses of example dams were conducted using the computer program GDAP [14]. The nonlinear earthquake response analyses with joint opening were carried out by computer program QDAP [15].

It is important to recognize that earthquake ground motions used in this paper may not be appropriate for safety evaluation of the example dams, especially in the case of Morrow Point Dam. Therefore the results and findings of this paper should not be used to draw general conclusion about the earthquake performance of these dams.

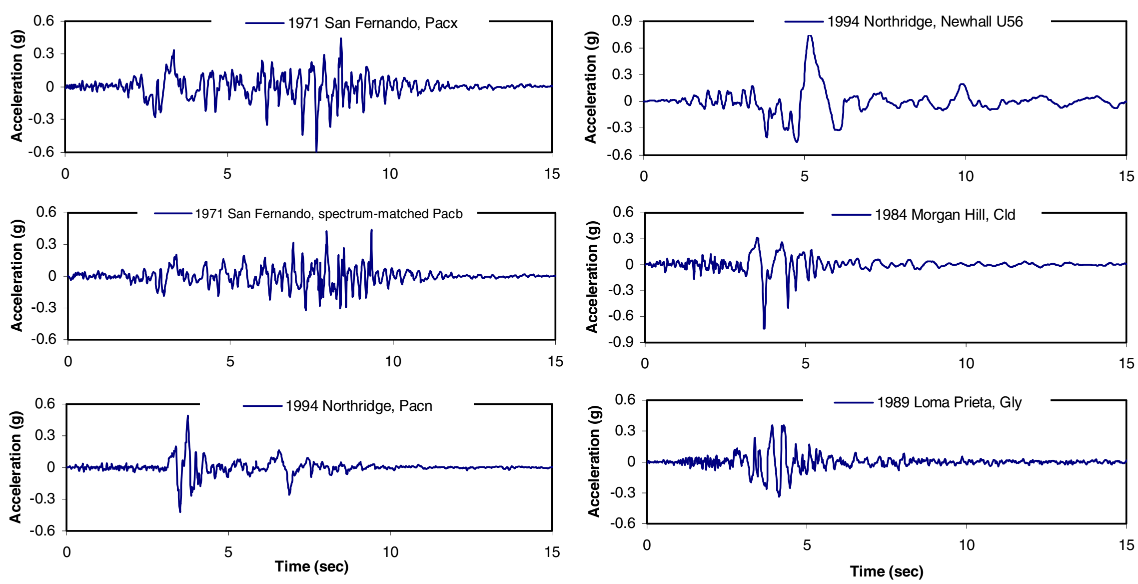


Fig. 10 Primary components of scaled acceleration time histories

Earthquake Ground Motions

For validation of performance and damage criteria, the example arch dams were subjected to the near field ground motions of a maximum earthquake event having a moment magnitude M_w of about 6-1/2. Five three-component sets of recorded acceleration time-histories from four recent California earthquakes were selected to account for sensitivity of the dam response to characteristics of earthquake ground motions. In addition, a three-component spectrum-compatible time-history derived using the 1971 Pacoima Dam record was also used. The smooth response spectra for the horizontal and vertical components of ground motion were constructed to be representative of median ground motions for an M_w 6-1/2 earthquake occurring at a distance of $R \approx 3$ miles. The primary components of the selected records are shown in Fig. 10. The acceleration time histories were scaled such that the sum of ordinates for the response spectra of each natural record would match the sum for the smooth response spectra in the period range of 0.1 to 0.4 sec, a period range that includes the most significant modes of vibration for the example dams.

Earthquake Response of Pacoima Dam

Located in southern California, Pacoima Dam is 365 ft high and 640 ft long at the crest level. Its thickness varies from 10.4 ft at the crest to 99 ft at the base. The finite-element model of the dam included three layers of solid elements through the dam thickness. The foundation model was constructed using solid elements arranged on semicircles having a radius twice the dam height. As a flood-control dam, the water level at Pacoima Dam is at 2/3 of the dam height. The dam-water interaction was represented by an incompressible fluid mesh that matched the concrete nodes on the upstream face of the dam and extended five water depths in the upstream direction.

The calculated natural periods of vibration for the 10 lowest modes vary from 0.233 to 0.088 sec. As expected, dam displacement and stress responses were different for different earthquake records, because the scaling had preserved characteristics of the recorded acceleration histories. The spectrum-compatible record (Pacb) induced the largest upstream displacement (2 inches), whereas the 1994 Northridge Newhall record (U56) produced the largest vertical (0.4 inches) and cross-stream (0.8 inches) displacements. Distribution of maximum stresses for all records is nearly the same (contours not shown), but the magnitude and location of peak stress values are different for each record. High tensile arch stresses in the upper central region and near the upper 1/4-point regions suggest low to moderate

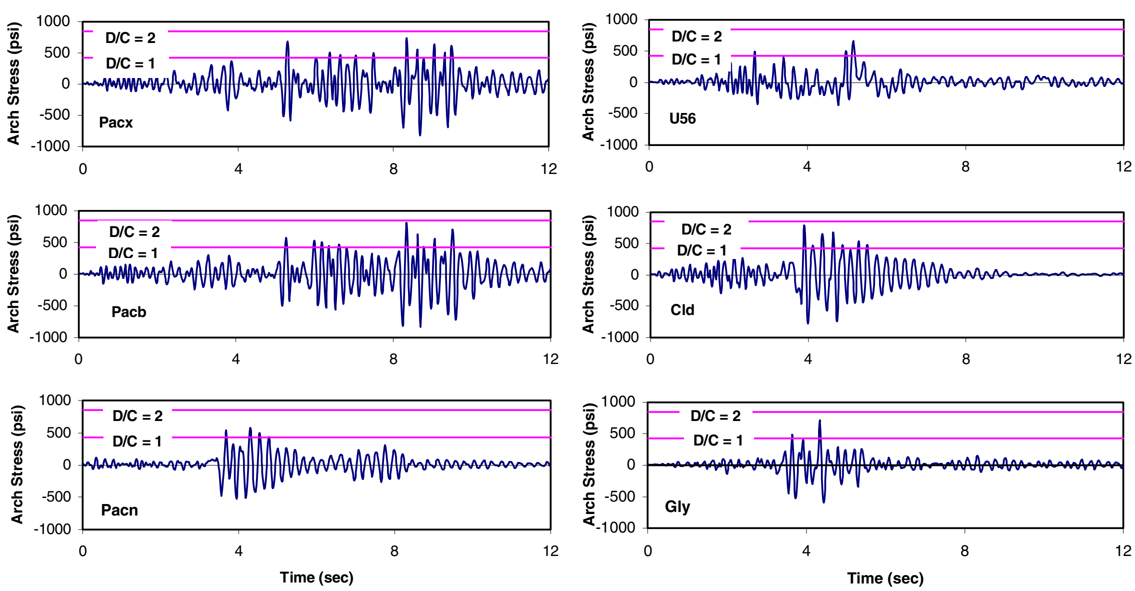


Fig. 11 Maximum stress histories for Pacoima Dam

contraction joint opening at these locations. Tensile cantilever stresses develop in the upper central region of the upstream face of the dam but their magnitudes are moderate and less than the tensile strength of the concrete (428 psi). This response behavior is consistent with the 1994 observed performance of the dam, where low to moderate contraction joint opening with minor lift line cracking occurred.

The calculated maximum arch stress histories in Fig. 11 show that arch stress peaks in excess of the assumed tensile strength of the concrete (428 psi) vary from 3 to 10 cycles for different earthquake records. Using this information, Fig. 12 is constructed to illustrate application of the proposed damage criteria to Pacoima Dam. The results show that DCR's for all earthquake records are less than 2, the cumulative overstress duration at all DCR's fall below the acceptance curve, and that overstressed regions are less than 20% of the dam-face surface area (not shown). It is therefore concluded that Pacoima Dam exhibits minor nonlinear response in the form of contraction joint opening. This performance prediction is consistent with the observed performance of the dam during the 1971 San Fernando and 1994 Northridge earthquakes where the damage was significant in the left abutment rock blocks but low to moderate within the body of the dam.

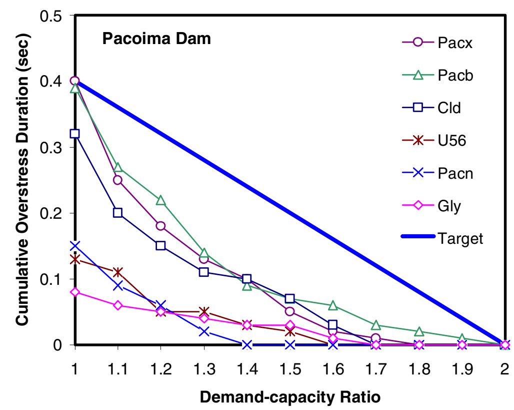


Fig. 12 Performance assessment for Pacoima Dam

Sliding of abutment wedges

With respect to sliding of the abutment wedges, the thrust block and Rock Masses A and B should be included in the dam-foundation model and analyzed as a coupled nonlinear problem. Such a nonlinear analysis should use nonlinear joint elements to represent the sliding Planes 1, 2, and 3 in the abutment wedge as well as several contraction joint in the dam including the one between the dam and thrust block. This will allow estimation of sliding displacement of the rock masses and opening and closing of the contraction joints during the ground shaking.

Earthquake Response of Morrow Point

Morrow Point Dam is a thin double-curvature arch structure in Gunnison, Colorado. It rises 468 ft above the foundation and spans a length of 724 ft at the crest. It is 12 ft thick at the crest and 52 ft at the base. Similar to the Pacoima Dam, the finite-element model of Morrow Point Dam also included three layers of solid elements through the dam thickness and a foundation model that was built on semicircles with a radius twice the dam height. The water level was assumed at the crest level to produce the most sever response. The dam-water interaction effects were represented using an incompressible fluid mesh.

The results show that the calculated periods of vibration of the 10 lowest modes vary from 0.338 to 0.104 sec. Displacement and stress response histories vary significantly for different input records in terms of the magnitudes and waveforms. Generally different earthquake records produce different maximum displacement and stress distributions with different peak values. This was to be expected because characteristics of the selected earthquake records varied significantly and were not modified by the scaling. The Gilroy record (Gly) induces the largest upstream displacement of 6.9 inches with the largest tensile arch stress of 3,000 psi at the center of the crest, both of which are 3.4 times larger than those obtained for Pacoima Dam. This is not surprising considering that Morrow Point Dam not only is thinner and taller the Pacoima Dam but also was analyzed with water at the crest level as opposed at 2/3 of the dam height in the case of Pacoima Dam. The results show that tensile arch stresses in the upper central-region and in the upper 1/4-point locations are very high and could lead to significant contraction joint opening in these locations. High tensile cantilever stresses also occur in the central and upper abutment regions but they are not concurrent with the maximum tensile arch stresses.

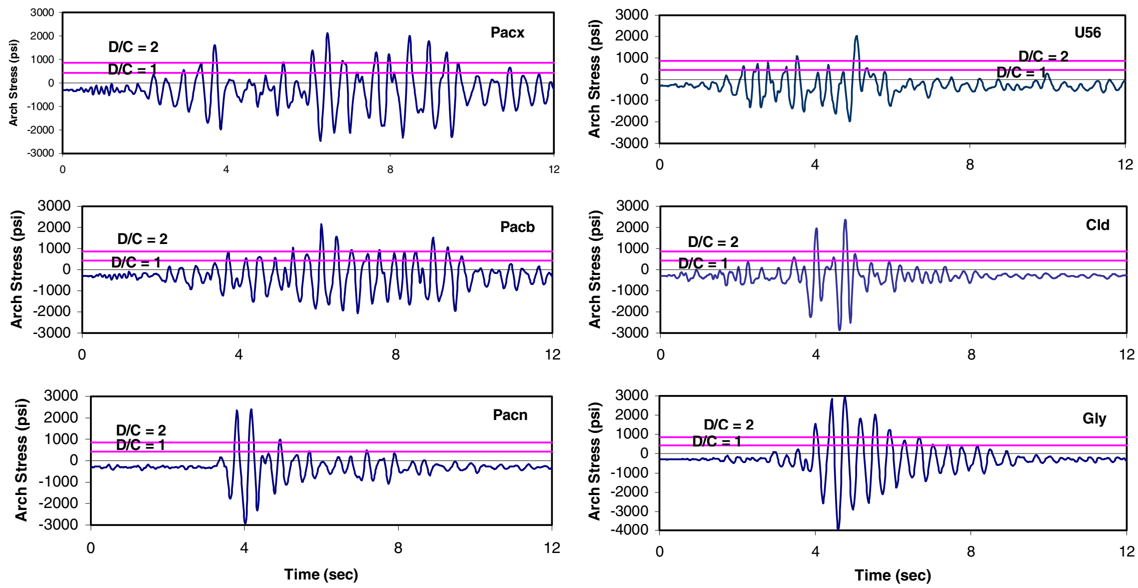


Fig. 13 Maximum stress histories for Morrow Point Dam

Fig. 13 displays maximum arch stress histories for all six earthquake records that were used to illustrate application of the damage criteria to Morrow Point Dam. The results in Fig. 14 show that the arch stress demand-capacity ratios for all six ground motions exceed 2 and that the cumulative inelastic duration, especially for Pacx, Pacb, and Gly records, are substantially greater than the acceptance level. This suggests that the selected ground motions induce significant nonlinear response in the form of repeated joint opening and closing and possibly tensile cracking. The actual amount of joint opening and whether or not it could lead to local failure is estimated by the nonlinear time-history analysis described next.

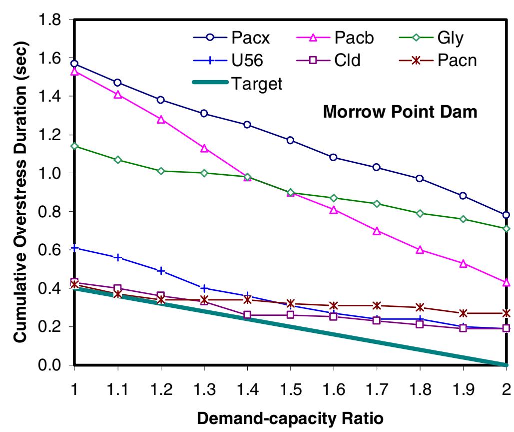


Fig. 14 Performance assessment of Morrow Point Dam

Nonlinear Earthquake Response of Morrow Point Dam

According to the proposed performance criteria, the selected ground motions produce significant nonlinear deformation in Morrow Point Dam requiring nonlinear analysis. To validate this aspect of the proposed damage criteria, nonlinear earthquake response of Morrow Point Dam was evaluated by permitting contraction joints to open and close during the ground shaking. For this purpose nonlinear joint elements were introduced at the crown section and at 1/4 points (Fig.15), where the linear analysis had indicated high tensile arch stresses. The nonlinear analyses were conducted using the computer program QDAP [15].

Fig. 15 illustrates deflected shapes at the time of maximum joint opening. The opening is the largest at the crown section and continues to mid height of the dam. The openings at the 1/4 span points are much smaller and penetrate less. Time histories of the maximum joint openings show that the contraction joints repeatedly open and close during the ground shaking, but they do not stay open continuously more than 0.5 sec, see Fig. 15. The 1994 Northridge Newhall record (U56) produces the largest joint opening reaching 3.3 inches at the mid crest.

The net effect of joint opening was that it released all high tensile arch stresses. However, the release of tensile arch stresses was not accompanied by unacceptable tensile or compressive cantilever stresses. At 3.3 inches, the joint opening is considered moderate. The cantilever blocks bounded by partially opened joints are expected to remain stable through shear-key interlocking. Consequently, excessive block movements are not expected to occur.

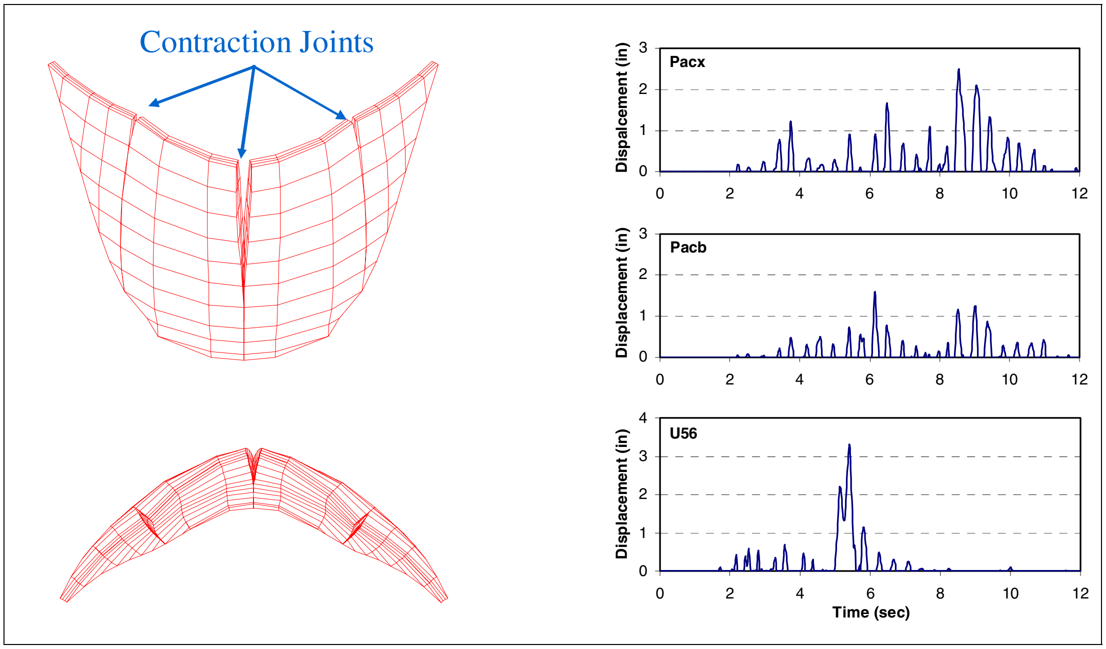


Fig. 15 Deflected shapes and time history of contraction joint opening

CONCLUSIONS

This investigation of the failure modes approach to safety evaluation of dams has led to the following conclusions:

1. The proposed failure modes approach provides a systematic methodology for assessment of the seismic performance and probable level of damage in the damage control range of behavior.
2. The acceptable performance is assessed using magnitudes of stress demand-capacity ratios, cumulative duration of stress excursions above various ratios of tensile strength of the concrete, as well as spatial extent of overstressed regions. Therefore, it eliminates shortcoming of the traditional evaluation procedures that rely on simple stress checks and vague consideration of other factors.
3. Threshold of performance acceptance and damage on the basis of the results of linear-elastic analyses is defined and the need for nonlinear analysis is determined.
4. Validation analyses indicate that the proposed approach is reliable and produces results that are consistent with the observed performance and level of damage.

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