Seismic Design and Evaluation of Concrete Dams – An Engineering Manual

by

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ABSTRACT

This paper provides an overview of the US Army Corps of Engineers' guidance for seismic design and evaluation of concrete dams, as presented in the Engineer Manual EM1110-2-6053. The requirements to design and evaluate concrete dams to have a predictable performance for specified levels of seismic hazard are discussed. The seismic input and performance levels associated with serviceability, damage control, and collapse prevention are defined. The analysis and evaluation procedures and acceptance criteria for each performance level are described. They consist of linear and nonlinear procedures for estimation of seismic response and acceptance criteria that use demand-capacity ratios, damage control thresholds, and irrecoverable level of movements and post-earthquake stability condition to assess dam safety. Finally, an example is provided to demonstrate the application of the manual to seismic evaluation of a concrete gravity dam.

KEYWORDS: Seismic design, seismic evaluation, concrete dams, gravity dams, arch dams, linear analysis, nonlinear analysis, joint opening, cracking, sliding displacement

1.0 INTRODUCTION

This paper provides an overview of the US Army Corps of Engineers' guidance for seismic design and evaluation of concrete hydraulic structures, as it relates to concrete dams. The Engineer Manual EM 1110-2-6053 (2007), which will be available shortly, covers seismic design and evaluation requirements for both the plain concrete structures such as dams and for the reinforced concrete structures such as navigation locks, intake/outlet towers, gravity walls, powerhouses, and spillway structures. The structures may be founded on rock, soil, or pile foundations and may or may not have backfill soil. However, only concrete gravity and arch dams will be discussed in this paper.

The manual introduces procedures that show how to design or evaluate a hydraulic structure to have a predictable performance for specified levels of seismic hazard. It states that the traditional design and evaluation procedures may still be used for feasibility and screening purposes. However, seismic design and evaluation of critical facilities should follow the procedures presented in the manual. The manual sets forth a set of requirements to prevent sudden collapse even though the structure may suffer severe damage, to limit damage to a repairable level, or to maintain functionality immediately after the earthquake.

2.0 DESIGN AND EVALUATION CRITERIA

The overall process of seismic design and evaluation of concrete dams consist of the following steps: development of design earthquakes and associated ground motions, establishment of performance levels and performance goals, analysis methodology for computation of seismic response, and interpretation and evaluation of results to assess dam safety. These steps are briefly described below followed by an example problem.

2.1 Design Earthquakes

Earthquake ground motions for the design and evaluation of the US Army Corps of Engineers' concrete hydraulic structures are the Operating Basis Earthquake (OBE) and the Maximum Design Earthquake (MDE) ground motions. Seismic forces associated with the OBE are considered unusual

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loads. Those associated with the MDE are considered extreme loads. Earthquake loads are to be combined with other loads that are expected to be present during routine operations.

2.1.1 Operating Basis Earthquake

The OBE is a level of ground motion that is reasonably expected to occur within the service life of the project, that is, with a 50-percent probability of exceedance during the service life. (This corresponds to a return period of 144 years for a project with a service life of 100 years).

2.1.2 Maximum Design Earthquake

The MDE is the maximum level of ground motion for which a structure is designed or evaluated. As a minimum, for other than critical structures, the MDE ground motion has a 10 percent chance of being exceeded in a 100-year period, (or a 1000-year return period). For critical structures, the MDE ground motion is the same as the maximum credible earthquake (MCE) ground motion. Critical structures, by ER 1110-2-1806 definition, are structures that are part of a high hazard project and whose failure will result in loss of life. The MCE is defined as the largest earthquake that can reasonably be expected to occur on a specific source, based on seismological and geological evidence.

2.2 Performance Levels

Three performance levels are considered for evaluation of earthquake responses of dams. These include serviceability performance, damage control performance, and collapse prevention performance, as shown in Figure 1.

2.2.1 Serviceability Performance

The dam is expected to be serviceable and operable immediately following earthquakes producing ground motions up to the OBE level.

2.2.2 Damage Control Performance

Certain parts of the dam can deform beyond their elastic limits (non-linear behavior) if non-linear displacement demands are low and load resistance is not diminished when the dam is subjected to extreme earthquake events. Damage may be significant, but it is generally concentrated in discrete locations where cracking and joint opening occur. The designer should identify all potential damage regions, and be satisfied that the structure is capable of resisting static loads and if necessary can be repaired to stop further damage by non-earthquake loads. Except for unlikely MCE events, it is desirable to prevent damage from occurring in substructure elements, such as foundation, and other inaccessible structural elements.

2.2.3 Collapse Prevention Performance

Collapse prevention performance requires that the dam not collapse regardless of the level of damage. The dam may suffer unrepairable damage with nonlinear deformation greater than those associated with the damage control performance but should not result in uncontrolled release of water. If the dam does not collapse when subjected to extreme earthquake events, its resistance can be expected to decrease with increasing displacements. This can affect the post-earthquake stability condition and should be checked. Collapse prevention performance should only be permitted for unlikely MCE events. Collapse prevention analysis should be evaluated using nonlinear dynamic procedures discussed later.

2.3 Performance Goals

Both strength and serviceability must be considered in the design of dams. For concrete dams, the consequences of inadequate strength can be failure by shear, flexure, tension, or compression. Lack of adequate strength can result in loss of life and severe economic loss. Dams must also be serviceable under sustained and frequent loads. Serviceability for usual static load conditions is a matter of limiting structural displacements. For the OBE loading, the serviceability requirement is to assure the dam will function without interruption, with little or no damage.

The plain concrete structures such as dams generally show limited-ductile behavior in flexure with brittle behavior in shear. Limited ductile behavior is characterized by an elastic range and limited plastic range that may include strain hardening or softening, followed by a complete or partial loss of strength to the residual level (see Figure 1). To properly assess this behavior it is necessary to understand the loading history, the changes in system stiffness and damping as cracking and joint opening occur, resisting loads redistribute, and response mechanisms change from the initial elastic deformation state to nonlinear sliding and rocking limit states. Under the MDE ground motions, limited ductile structures should have sufficient strength to assure performance will remain within the inelastic region where strength increases with an increase in strain (i.e. damage control region in Figure 1).

Under the MCE ground motions the dam may perform in the collapse prevention region, provided that it can be shown that the dam will remain stable without uncontrolled release of water. However, this requires the use of an appropriate nonlinear dynamic analysis with parameter sensitivity studies to assure that the dam will remain stable during and after the earthquake shaking.

2.4 Design Requirements

2.4.1 Strength Design

Strength design for dams subjected to earthquake ground motions is achieved by reducing the probability of structural collapse to an acceptable level. This is accomplished by selecting a representative design basis earthquake event to be used in combination with specific design and evaluation procedures that assure the structure will perform as intended. Seismic design and evaluation is most often based on linear-elastic response-spectrum or time-history analysis procedures, although nonlinear analysis procedures can be used for evaluation of certain nonlinear mechanisms. The design basis earthquake event used for strength evaluation is the Maximum Design Earthquake (MDE).

2.4.2 Serviceability Design

Serviceability design for dams subjected to earthquake ground motions is achieved by reducing the possibility of structural damage to a negligible level. As for strength performance, this is accomplished by selecting an appropriate design basis earthquake event to be used in combination with appropriate design and evaluation procedures. Evaluation is based on linear-elastic response spectrum analysis or time history analysis procedures. The design basis earthquake event used for serviceability evaluation is the Operating Basis Earthquake (OBE).

2.4.3 Loading Combinations

The following loading combinations establish the ultimate strength and serviceability requirements for the design and evaluation of concrete dams. The loading combinations represent the total demand (dead load + live load + earthquake) for which the structure must be designed or evaluated.

Strength design loading combination:

$$Q_{DC} = Q_D + Q_L + Q_{MDE} \qquad (1)$$

Q_{DC} = Combined action of dead, live, and MDE loads for use in evaluating damage control performance

Q_D = Dead load effect

Q_L = Live load effect + uplift

Q_{MDE} = Earthquake load effect from MDE ground motions including hydrodynamic and dynamic soil pressure effects

The live load effect is the structure response to live loads such as hydrostatic, earth pressure, silt, and temperature loads. Live loads to be considered are those that are likely to be present during the design earthquake event. The earthquake load may involve multi-component ground motions with each component multiplied by +1 and -1 to account for the most unfavorable earthquake direction.

Serviceability loading combination:

$$Q_S = Q_D + Q_L + Q_{OBE} \qquad (2)$$

Q_S = Combined action of dead, live, and OBE loads for use in evaluating serviceability performance

Q_{OBE}= Earthquake load effect from MDE ground motions including hydrodynamic and dynamic soil pressure effects

Live loads to be considered are those that are likely to be present during the OBE earthquake event.

3.0 EARTHQAUKE GROUND MOTIONS

The earthquake ground motions for design and evaluation of dams are generally characterized in terms of response spectra and acceleration time histories. Information on development of response spectra can be found in EM1110-2-6050 (1999). Information on development and selection of earthquake acceleration time histories and time history dynamic analysis of concrete hydraulic structures can be found in EM 1110-2-6051 (2003). General guidance and direction for the seismic design and evaluation of all civil works projects are provided in ER 1110-2-1806 (1995).

3.1 Standard Response Spectra

The manual provides guidance for constructing standard acceleration response spectra based on the most recent national seismic hazard data. The standard response spectra are used as a starting point for developing conceptual designs and performing evaluations, determining if the earthquake loading controls the design, and establishing the need for more refined analysis and the impact the earthquake loading might have on construction costs.

3.2 Site-Specific Response Spectra

Earthquake ground motions depend on source characteristics, source-to-site transmission path properties, and site conditions. All of these factors can be considered in detail in a site-specific ground motion study for developing site-specific response spectra. There are two basic approaches to developing site-specific response spectra: deterministic and probabilistic. In the deterministic approach, one or more earthquakes are specified by magnitude and location with respect to a site. Usually, the earthquake is taken as the Maximum Credible Earthquake (MCE), and assumed to occur on the portion of the source closest to the site. The site ground motions are then estimated deterministically, given the magnitude and source-to-site distance.

In the probabilistic approach, site ground motions are estimated for selected values of probability of ground motion exceedance in a design time period or for selected values of the annual frequency or return period of ground motion exceedance. A probabilistic ground motion assessment incorporates the frequency of occurrence of earthquakes of different magnitudes on the various seismic sources, the uncertainty of the earthquake locations on the various sources, and the ground motion attenuation including its uncertainty. Guidance for developing site-specific response spectra and for using both the deterministic approach and the probabilistic approach can be found in EM 1100-2-6050 (1999).

3.3 Acceleration Time Histories

Acceleration time-histories of ground motion for dynamic analysis of dams are developed using procedures described in EM 1110-2-6051 (2003). The overall objective is to develop a set (or sets) of time-histories that are representative of site ground motions that may be expected for the design earthquake(s) and that are appropriate for the types of analyses planned for the dam. The following steps are included in this process:

- Initially selecting recorded time-histories that are reasonably consistent with the tectonic environment of the site; design earthquake (magnitude, source-to-site distance, type of faulting); local site conditions; and design ground motion characteristics (response spectral content, duration of strong shaking, and special characteristics, e.g. near-source characteristics).

- Modifying time-histories selected above to develop the final set(s) to be used in dynamic analysis. Two approaches that can be used in this process are simple scaling of time-histories (by constant factors) so that a set of time-histories has spectral values that, on average, are at the approximate level of the design response spectrum; and spectrum matching, which involves modifying the frequency content of a given time-history so that its response spectrum is a close match to the design response spectrum.

- Further modifying the time-histories for site response effects and spatial variations of ground motion, if it is desired to incorporate effects of site topography and wave passage and incoherence in the ground motions that would arrive beneath the dam.

3.4 Selection of Acceleration Records

Application of the above guidelines is straightforward when design earthquakes are expressed deterministically. However, the application of the guidelines is less straightforward when the design earthquake ground motions (typically the response spectrum) are derived from a probabilistic ground motion analysis (often termed a probabilistic seismic hazard analysis or PSHA). From this type of analysis, the design response spectrum for a design return period reflects the contribution of different earthquake magnitudes and distances to the probabilities of exceedance. Therefore, when the design response spectrum is probabilistically based, the PSHA should be de-aggregated to define the relative contributions of different magnitudes and distances to the ground motion hazard. Furthermore, the de-aggregation should be done for probability values or return periods that correspond to those of the design earthquake and for response spectral periods of vibration of significance for seismic structural response because the relative contributions of different magnitudes and distances may vary significantly with return period and period of vibration. The dominant magnitude and distance is then considered as representative in selecting time histories and defining strong motion duration.

For use in linear dynamic analysis, at least three time-histories (for each component of motion) should be used for each design earthquake. For use in nonlinear dynamic analysis, at least five time-histories should be used (for each component of motion) for each design earthquake. Fewer time-histories are required for linear dynamic analysis than for nonlinear analysis because the dynamic response of a linear structure is determined largely by the response spectral content of the motion, whereas the response of a nonlinear structure may be importantly influenced by the time domain character of the time-history (e.g., shape, sequence, and number of pulses) in addition to the response spectrum characteristics. Since these time domain characteristics may vary greatly for time-histories having similar spectral content, more time-histories are required for nonlinear analysis to capture the variability in response. If the nonlinear response is found to be significantly sensitive to the time-history characteristics for the records selected, then the set of time-histories should be expanded.

4.0 ANALYSIS METHODOLODY

The manual recommends progressive analysis methodology where the seismic evaluation is performed in phases in order of increasing complexity progressing from simple equivalent lateral force methods, to linear elastic response-spectrum and time-history analysis, to nonlinear methods, if necessary.

Gravity dams with simple geometry may initially be analyzed using the equivalent lateral force method, but generally 2D or 3D finite-element dynamic analysis will be required. Arch dams should always be analyzed using 3D finite-element idealization. For both gravity and arch dams, dynamic interactions with the foundation rock and the impounded water should be considered. Foundation rock may be idealized using simplified massless model, viscoelastic with inertia and damping effects, or a finite-element mesh with transmitting boundaries. The dam-water interaction effects may be represented by the Generalized Westergaard added-mass (Kuo, 1982), an incompressible fluid mesh (Kuo, 1982), or a compressible fluid mesh with energy loss capability at the reservoir bottom due to sediment accumulation (Hall and Chopra, 1980).

5.0 EVALUAITON PROCEDURES

Evaluation of seismic performance of concrete dams starts with utilization of a demand-to-capacity ratio (DCR) as a performance indicator, then progresses to the use of performance threshold curves using both DCR and cumulative inelastic duration, and finally continues with the extent of irrecoverable movements caused by sliding and rotation, as appropriate. For concrete dams, DCR is defined as the ratio of stress demands to static tensile strength of the concrete and is used to assess the results of response-spectrum analysis. The performance threshold curves, discussed later, are used to assess the results of linear time-history analysis. They provide a measure of severity of the nonlinear response in terms of amount of cracking and joint opening. Finally, irrecoverable movements which are obtained by conducting nonlinear time-history analysis are used to assess stability condition of the dam under severe earthquake ground shaking.

5.1 Acceptance Criteria for Response-Spectrum Analysis

A linear-elastic response-spectrum analysis is generally the first step in the evaluation process. The earthquake demands in terms of stresses are computed and compared with the stress capacity of the concrete to assess whether the resulting DCR ratios meet the allowable values listed in Table 1. In cases where DCR limits for tensile stresses are exceeded, a linear-elastic time-history analysis is generally performed and evaluated, as discussed next.

5.2 Acceptance Criteria for Linear Time-history Analysis

The acceptance criteria for the linear-elastic time-history analysis of concrete dams are based on the use of performance threshold curves (Ghanaat, 2002 and EM 1110-2-6051, 2003). The dam response to the MDE is considered to be within the linear-elastic range of behavior with little or no damage if computed stress demand-capacity ratios are less than or equal to 1.0. The dam is considered to exhibit nonlinear response in the form opening and closing of contraction joints and cracking of the horizontal joints (lift lines) and the concrete if the estimated demand-capacity ratios exceed 1.0. Note that in the case of arch dams where the ability of contraction joints to resist tension is limited, the joints may open even if demand-capacity ratios are less than or equal to 1.0. However, the amount of contraction joint opening at a DCR $\leq$ 1, is expected to be small with negligible or no effects on the overall stiffness of the dam. For DCRs exceeding unity the performance is evaluated as follows:

5.2.1 Gravity Dams

The level of nonlinear response or cracking is considered acceptable if demand-capacity ratios are less than 2.0 and the percent of overstressed dam-section surface areas and the cumulative duration of stress excursions above the tensile strength of the concrete fall below the performance threshold curves given in Figure 2. Consideration should also be given to relation between the fundamental period of the dam and peak of the earthquake response spectra. If lengthening of the periods of vibration due to nonlinear response behavior causes the periods to move away from the peak of the spectra, then the nonlinear response would reduce seismic loads and improve the situation by reducing stresses below the values obtained from the linear time-history analysis. When these performance conditions are not met, then a nonlinear time-history analysis would be required to estimate the damage more accurately, as discussed in Section 5.3.1.

5.2.2 Arch Dams

The level of nonlinear response in the form of cracking and/or opening of contraction and lift joints is considered acceptable if DCR < 2 and the percent of overstressed dam surface areas and the cumulative inelastic duration of tensile stress cycles exceeding tensile strength of the concrete fall below the performance threshold curves given in Figure 3. 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 demand. If these performance criteria are not met, then a nonlinear analysis would be required for more accurate estimate of the damage, as described in Section 5.3.2.

5.3 Evaluation and Acceptance Criteria for Nonlinear Time-history Analysis

5.3.1 Gravity Dams

While it is possible to model material and other sources of nonlinearity in analysis of gravity dams, the required parameters are either not known or well defined. For this reason the nonlinear dynamic analysis of a gravity dam should focus on capturing the potential failure modes that would have the most impact on the stability of the dam. A typical gravity dam is built as individual monoliths separated by vertical joints. Furthermore, construction of each monolith involves placement of concrete in lifts that produces horizontal joints whose tensile strength could be less than that of the parent concrete. Consequently, in a major earthquake it is likely that the vertical joints would open and close repeatedly and tensile cracking would occur along the lift lines, at the dam-foundation interface, and at the change of slope in the upper part of the dam where stress concentration occurs. The nonlinear performance evaluation of gravity dams therefore starts with a linear-elastic time-history analysis to identify overstressed regions that would experience cracking, followed by nonlinear dynamic analyses incorporating slippage and rotation with respect to opened joints and cracked sections, as well as post-earthquake analyses for static loads and after-shock excitations.

The results of nonlinear analysis will include sliding displacement and rotation demands that must be sufficiently small not to jeopardize safety of the dam during the main event as well as during the after shocks. This means that after the level of damage has been established for the main event, the damaged structure should be tested against the probable aftershocks that could be one to two magnitudes smaller than the main shock. In addition, post-earthquake static stability analyses should be carried out so that the ability of the damaged structure to resist the operating loads and the potential increased uplift can be demonstrated.

For example, a linear-elastic dynamic analysis may indicate that the gravity dam will experience high tensile stresses at the dam-foundation interface and that the dam does not pass the acceptance criteria set forth for the linear analysis. In subsequent nonlinear dynamic analyses gap-friction elements are introduced at the high tensile-stress region of the base to allow formation and propagation of cracks, which may extend through the entire base of the dam. The results may indicate that the dam would fully crack leading to sliding and rocking responses with a permanent displacement (offset) at the end of the shaking. The magnitude of the permanent sliding displacement is estimated and compared with operational and safety requirements. The performance of the dam is then considered satisfactory if the cracks and permanent sliding displacement do not lead to uncontrolled release of water, and that the post-earthquake stability of the dam under static loads is not compromised. A numerical example of linear and nonlinear time-history analyses and performance evaluation of a non-overflow gravity dam to illustrate this process is presented in Section 6.

5.3.2 Arch Dams

Arch dams are generally built as independent cantilever monoliths separated by vertical contraction joints. Since contraction joints cannot transfer substantial tensile stresses in the arch direction, the joints can be expected to open and close repeatedly as the dam vibrates in response to severe earthquake ground motions. Construction of arch dams also involves horizontal construction joints known as lifts that may exhibit lower tensile strength than the mass concrete. Consequently opening of contraction joints and cracking of lift joints are the most likely nonlinear mechanisms that could occur in arch dams. Such conditions can be modeled and analyzed using QDAP (Quest Structures, 2001) or other finite-element programs with nonlinear joint capabilities. As in the case of linear analysis the concrete arch and the foundation rock are discretized using standard 3D solid elements, but joints and fractures in the dam, at the dam-foundation interface, or within the foundation are represented by nonlinear 3D joint elements. Therefore the only nonlinear effects considered for the response of the dam are those associated with the opening, closing, and sliding along the joints and cracked sections. Since opening of the contraction joint and cracking of the lift joints relieve high tensile stresses, the traditional stress-based criteria will not be applicable to the nonlinear results. Instead the magnitude of compressive stresses, the extent of joint opening or cracking, and the amplitude of non-recoverable movements of concrete blocks bounded by opened joints would control the overall stability of the dam and should be used to assess dam safety.

The nonlinear dynamic analysis of arch dams should also assess stability of potentially moveable blocks in the abutments if there are adversely jointed rock blocks directly beneath the dam. This problem is best handled as a coupled dynamic problem in which the moveable blocks are modeled as part of the dam finite-element model to allow joint slippage in the abutments and the effects it might have on the stability of the dam. The block joints can be modeled using 3D joint elements discussed above which resist bearing and shear but not tension. The sensitivity of the results to shear strength of the joints and strength degradation with movement and uplift pressures should be investigated.

6.0 EXAMPLE GRAVITY DAM

The purpose of this example is to demonstrate the application of manual guidelines to earthquake response assessment of gravity dams. The example problem is a 74.17 m (243.33 ft) high non-overflow gravity dam with crest thickness of 9.75 m (32 ft) and a base thickness of 53.83 m (176.6 ft). On the lower two-third, the dam is sloped at 1/10 on the upstream and at 7/10 on the downstream face. The dam is first analyzed using linear time-history method to demonstrate the linear-elastic acceptance criteria discussed in Section 5.2, and then evaluated by nonlinear time-history method to assess potential sliding and rocking responses under earthquake shaking in accordance with Section 5.3.1.

6.1 Earthquake Ground Motions

The example gravity dam was assumed to be located in the near field of a maximum earthquake event having a moment magnitude M_w of about 6-1/2. Four sets of recorded acceleration time-histories from three California earthquakes were selected in accordance with Section 3.4. These included the Pacoima Dam record from 1971 San Fernando earthquake, the Gilroy Array No. 1 record from 1989 Loma Prieta earthquake, the Newhall record from 1994 Northridge earthquake, and the 1971 Pacoima Dam record modified to match the design response spectra. The smooth design 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 = 5 km (3.1 miles). The ground motions 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 0.06 to 0.3 sec (see Figure 4). Time-histories of the horizontal components of the records are plotted in Figure 5. This figure clearly demonstrates the pulsive type motions contained in the San Fernando and Northridge records.

6.2 Linear Elastic Response

6.2.1 Finite Element Model

The linear-elastic time-history analysis of the example gravity dam was carried out using a 2D model of the dam and foundation rock with a concrete modulus of elasticity of 40,679 MPa (5.9×10^6 psi), a Poisson's ratio of 0.19, and a unit weight of 158 pcf. The foundation rock was assumed massless but its modulus and Poisson's ratio were assumed to be respectively 39,990 MPa (5.8×10^6 psi) and 0.19. The inertia forces of the impounded water were represented by added hydrodynamic mass values in accordance with the generalized Westergaard method. The finite-element model is shown in Figure 6.

6.2.2 Evaluation of Linear Response

The gravity dam model was analyzed for the combined effects of static plus seismic loads. The static loads included the gravity, hydrostatic pressures due to a headwater depth of 73.15 m (240 ft), and uplift pressures. The earthquake loads consisted of the inertia forces generated by earthquake ground motions described above. A linear uplift pressure distribution from full headwater to zero tailwater was assumed. The uplift pressure was also assumed not to change during the earthquake ground shaking. The results of analyses include envelopes of maximum stresses, time history of stresses, time history of displacements, and time history of reaction forces at the dam-foundation contact. The envelopes of maximum stresses are used to assess severity and extent of overstressed regions (Figure 7). The stress time histories are used to compute cumulative duration of stress excursions for comparison with the acceptance limits (Figure 8). Finally, time histories of reaction forces are used to compute instantaneous factors of safety to assess stability condition of the dam (Figure 9).

The results of linear analyses indicate that high tensile stresses generally develop at the heel and toe of the dam as well as at upper elevations near the change of slopes, but they are largest at the heel of the dam (Figure 7). Recognizing that the tensile strength of the dam-rock interface or of the fractured rock immediately below the interface is lower than that of the intact concrete, the cracking is expected to initiate at the heel of the dam and propagate toward the toe. The instantaneous factors of safety for an assumed friction coefficient of unity (Figure 9) indicate that factor of safety repeatedly falls below unity during the earthquake ground shaking, an indication that the sliding might occur along the base.

The results of linear-elastic time-history analyses were also compared with the acceptance criteria established in Section 5.2.1. Figure 10 shows that the dam section overstressed areas exceed the acceptance limit for three of the earthquake records; only overstressed areas for the Loma Prieta record fall below the acceptance curve. Figure 11 compares cumulative inelastic duration of stress cycles at the heel of the dam with the acceptance curve. It is obvious that all cumulative duration for stresses at the heel of the dam are above the acceptance threshold set for the linear-elastic analysis. Similar results for stresses near the toe of the dam showed cumulative duration for stresses at this location fall below the acceptance threshold. These results suggest that the cracking would initiate from the heel of the dam but may or may not propagate through the entire base of the dam; thus nonlinear time-history analyses are needed to evaluate the crack propagation and the effects it may have on sliding of the dam, as discussed next.

6.3 Nonlinear Response Analysis

The results of linear analyses indicate that high tensile stresses develop at the base of the dam, on the upstream face of the dam, and on the downstream face near the change of slope. The magnitudes of stresses are greater at the base of the dam than they are at the upper elevations. For this reason and also because tensile strength of the dam-rock contact and the rock below is expected to be lower than that of the concrete, the nonlinear response in the form of tensile cracking is likely to start at the base. The nonlinear analysis of the dam was therefore formulated to capture this nonlinear mechanism. In this example, gap-friction elements are introduced at the base of the dam to simulate cracking and the sliding and rocking responses that might follow.

6.3.1 Nonlinear Finite-Element Model

Finite-element model for the nonlinear analysis consists of the dam monolith and gap-friction elements. The foundation rock is not included in the model in order to reduce computational efforts. The tensile cracking at the base of the dam is modeled by introducing gap-friction elements between the dam and the rigid foundation. The gap-friction elements are nonlinear elements that can resist bearing and shear parallel to the bearing plane but not tension. The friction forces follow the Coulomb theory and thus are directly proportional to bearing forces in the element. Figure 12 shows the dam finite-element model with 23 gap-friction elements at the base of the dam. Except for the nonlinear gap-friction elements, the rest of the dam is assumed to behave elastically.

6.3.2 Evaluation Loads and Parameters

The nonlinear dynamic analysis was conducted for the combined action of static and earthquake loads. The static loads included gravity, hydrostatic pressures, and uplift pressures, all of which were applied as initial loads. The uplift pressures were assumed to vary linearly from the headwater to tailwater with no changes during the earthquake ground shaking. A zero tailwater was assumed. The earthquake loads included the same four acceleration time-history records discussed previously.

The gap-friction elements used in this example have friction properties for shear deformation in the horizontal direction and gap behavior in the axial or vertical direction. The gap properties usually include zero tension resistance but the element can be preloaded so that the element could start with a certain amount of tension resistance. In this example, zero cohesion with zero tensile strength and a friction angle of 45 degrees were assumed for sliding along the dam-foundation contact surface.

6.3.3 Evaluation of Nonlinear Response

Stress results in Figure 13 show significant reduction in tensile stresses due to tensile cracking at the base of the dam (compare Figure 13 with Figure 7). Note that magnitudes of stresses within the body of the dam have also dropped significantly. This confirms the assumption that tensile cracking at the base of the dam would preclude tensile cracking elsewhere by relieving high tensile stresses within the body of the dam.

It was also found that the crack propagates through the entire base of the dam followed by sliding of the dam in the downstream direction. Figure 14 shows time histories of sliding displacements for all joints across the base of the dam. The sliding displacement varies from joint to joint due to deformations of the flexible dam. The sliding displacements are slightly higher for nodes closer to the heel and reduce toward the toe. For a rigid dam sliding displacements are the same for all nodes across the base. Figure 15 displays horizontal displacement history for a crest nodal point, where the permanent displacement at the end of the record is evident. The results show an overall permanent displacement of about 0.8 inches. Although this permanent displacement is relatively small and does not appear alarming, the main concern is the post-earthquake stability condition of the dam under static loads. The post-earthquake static sliding factor of safety for the example dam was found to fall below unity. This is because the uplift forces have increased due to formation of crack at the base of the dam. Based on these results the dam should be retrofitted and its shear resistance increased to remedy the situation.

7.0 CONCLUDING REMARKS

The paper presents an overview of an engineering manual for performance-based design and evaluation of concrete hydraulic structures as it relates to concrete dams. The manual introduces procedures that show how to design or evaluate a hydraulic structure including dams to have a predictable performance for specified levels of seismic hazard. Three seismic performance levels including serviceability, damage control, and collapse prevention are introduced. The analysis and evaluation procedures as well as the acceptance criteria for each performance level are described and demonstrated. The performance criteria presented in the manual employs demand-capacity ratios, damage control thresholds, and nonlinear response behavior as well as the post-earthquake stability condition to assess dam safety. This is a departure from the traditional simple stress checks in which the predicted elastic stress is compared with the expected concrete strength.

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Table 1. DCR Allowable Values for Response-Spectrum Analysis of Concrete Dams

Action In terms of stresses	Performance Objectives	
	Damage Control (MDE)	Serviceability (OBE)
Tension due to flexure	1.5	1.0
Diagonal tension due to shear.	0.9	0.8
Shear due to sliding.	1.0	0.8

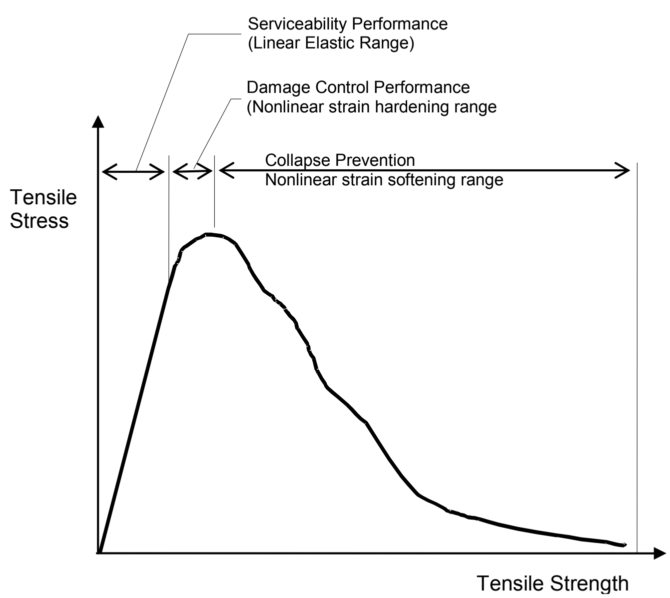

Figure 1. Stress – Strain relationship for plain concrete structures
illustrating three performance levels

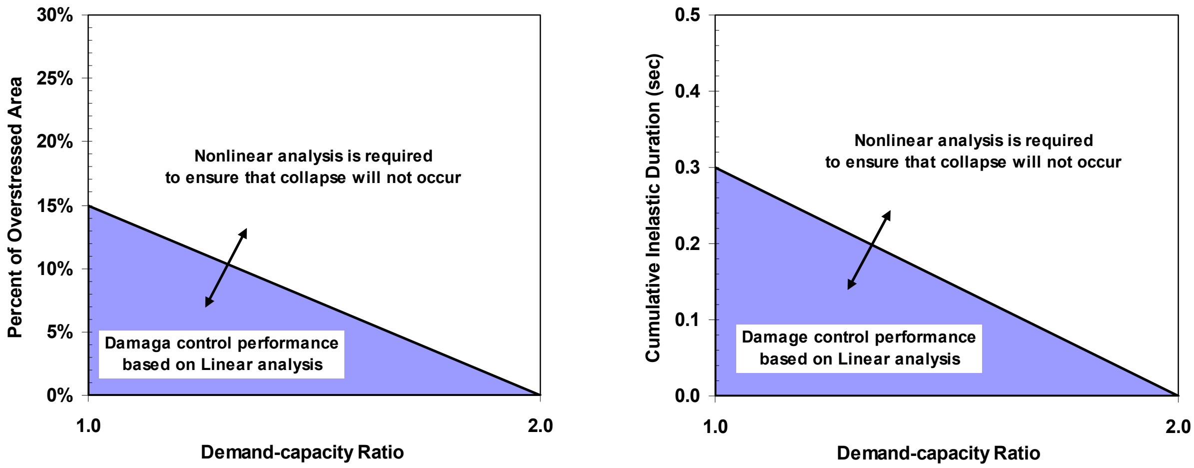

Figure 2. Performance threshold curves for concrete gravity dams

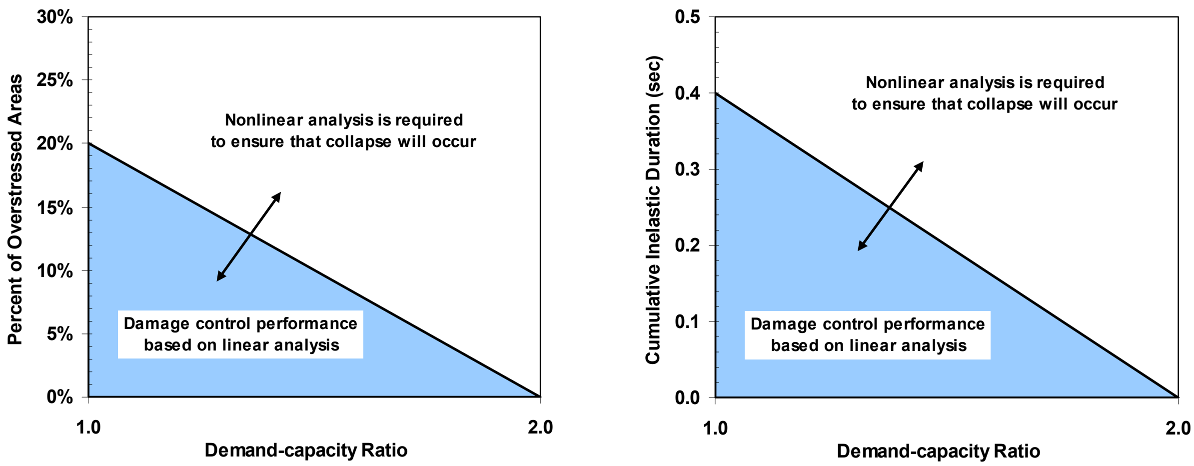

Figure 3. Performance threshold curves for concrete arch dams

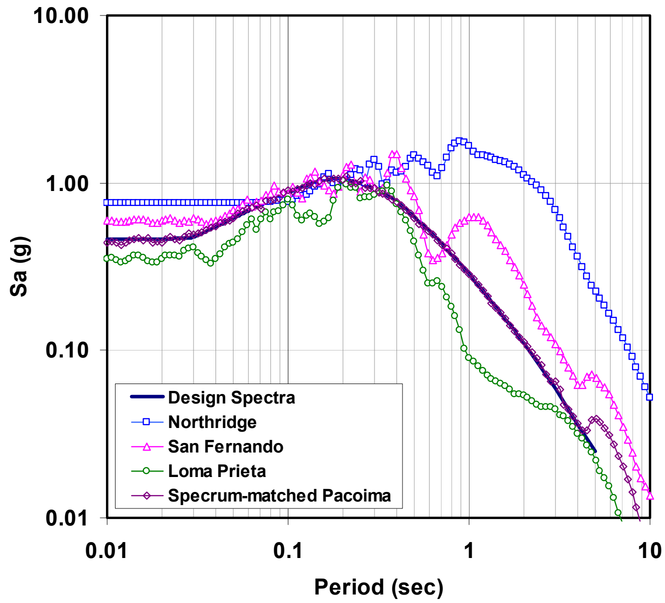

Figure 4. Comparison of design response spectra with spectra of scaled records

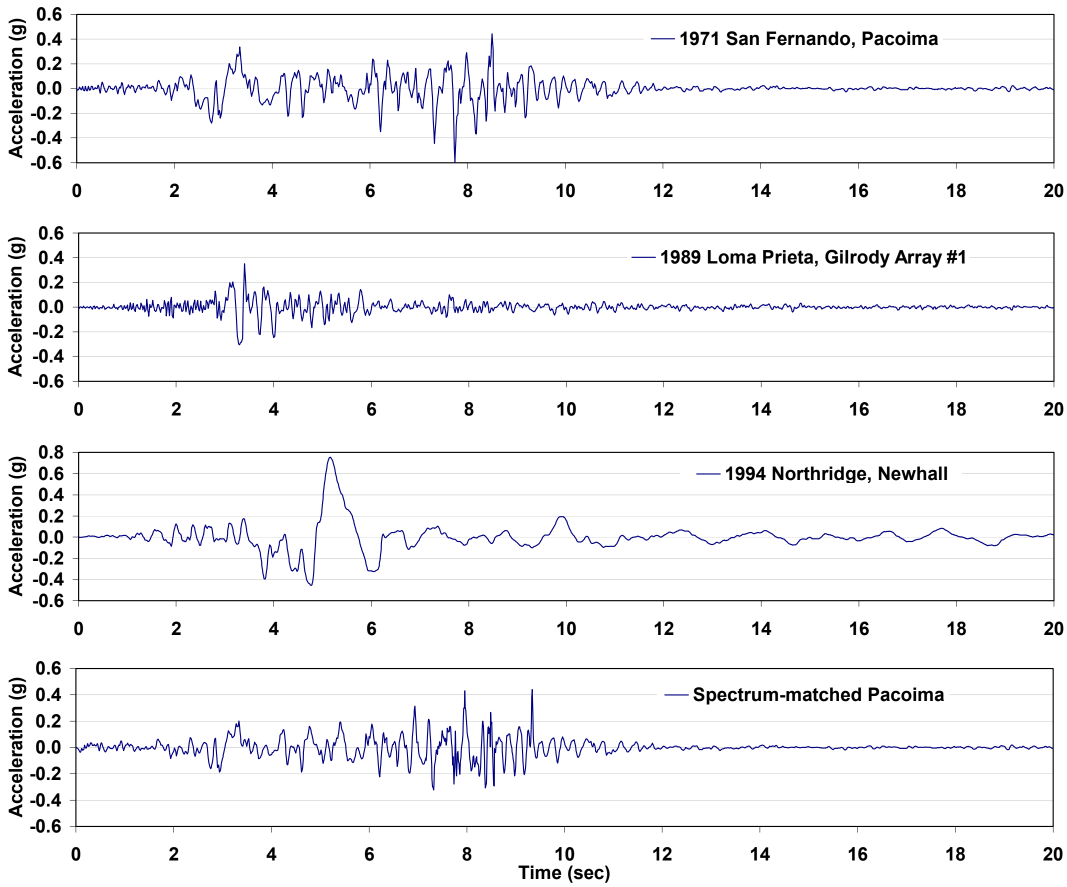

Figure 5. Horizontal component of earthquake input acceleration time histories

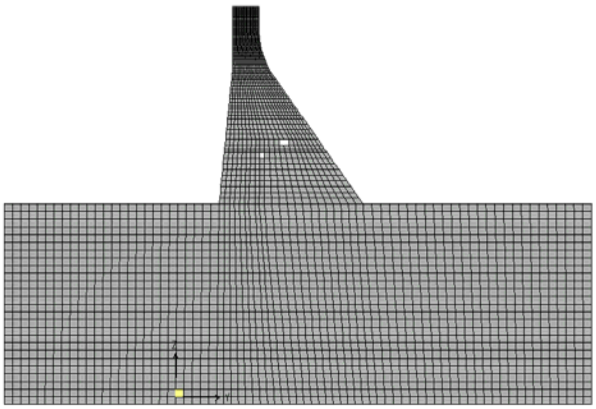

Figure 6. Dam-foundation finite-element model

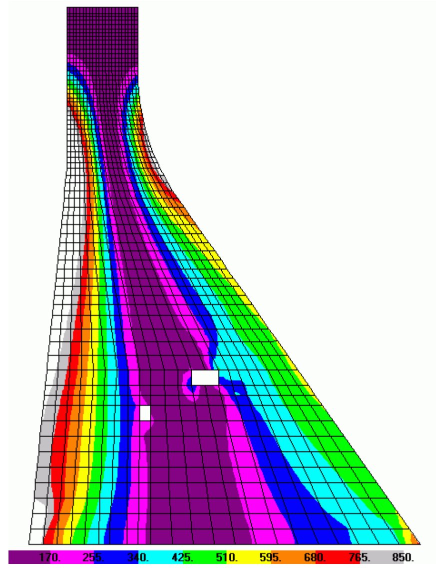

Figure 7. Envelopes of maximum vertical stresses for San Fernando record (linear model) (1 MPa = 145 psi)

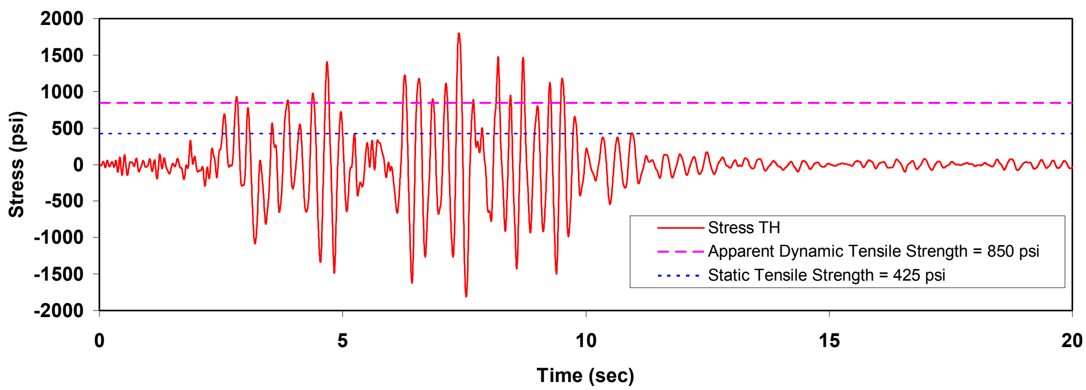

Figure 8. Time history of maximum vertical stresses at the heel of the dam for San Fernando record.

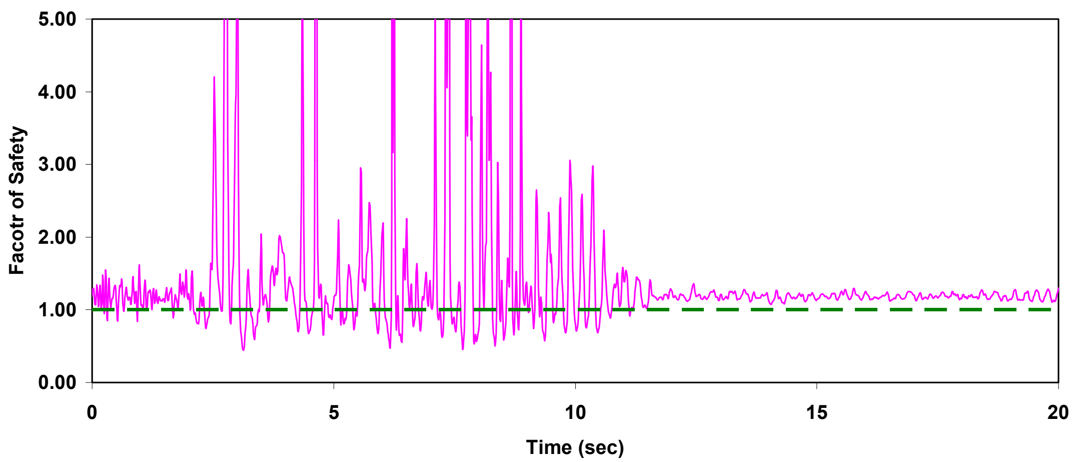

Figure 9. Instantaneous factors of safety for San Fernando record.

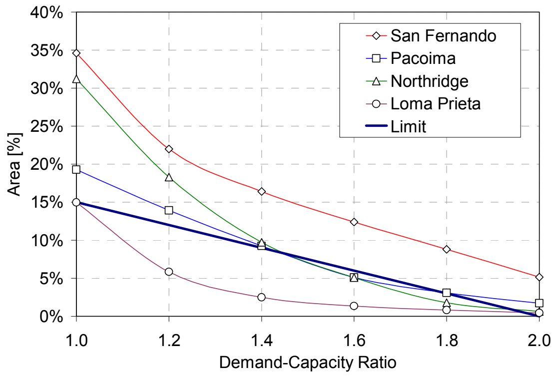

Figure 10. Comparison of percentage of overstressed areas with acceptance limits

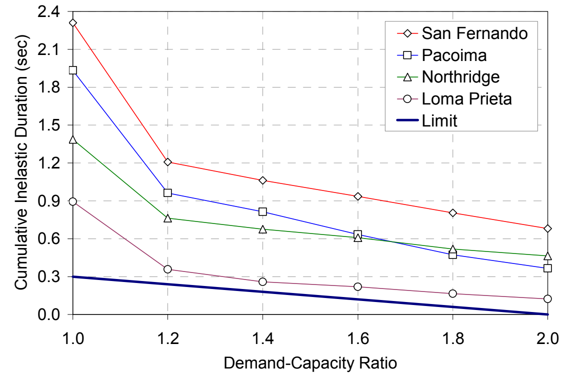

Figure 11. Comparison of cumulative duration of stress cycles with acceptance limits for stresses at the heel of the dam

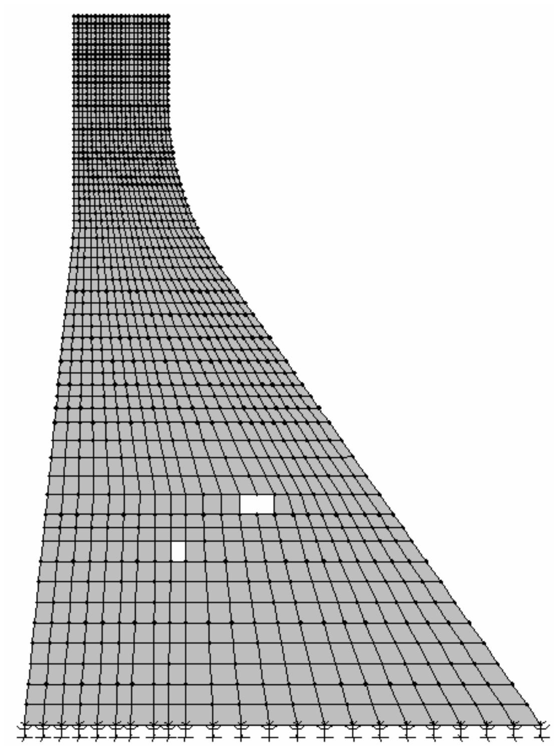

Figure 12. Dam finite-element model with gap-friction elements

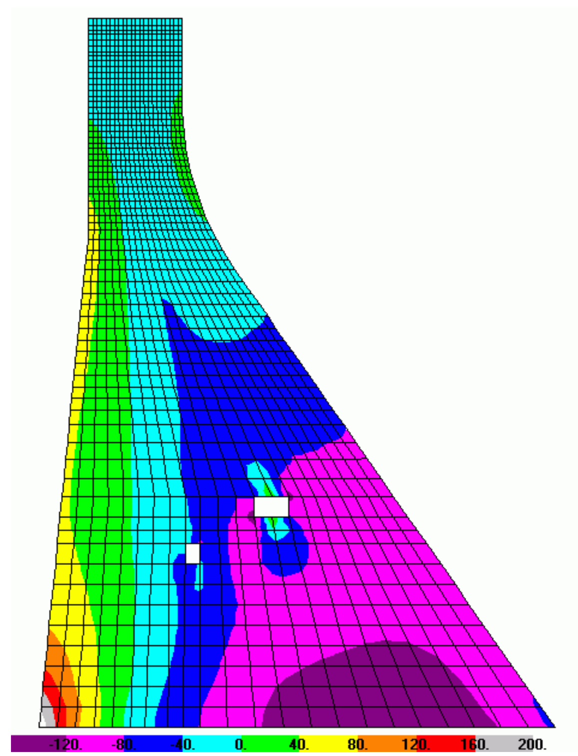

Figure 13. Envelopes of maximum vertical stresses for San Fernando record from nonlinear analysis (1 MPa = 145 psi)

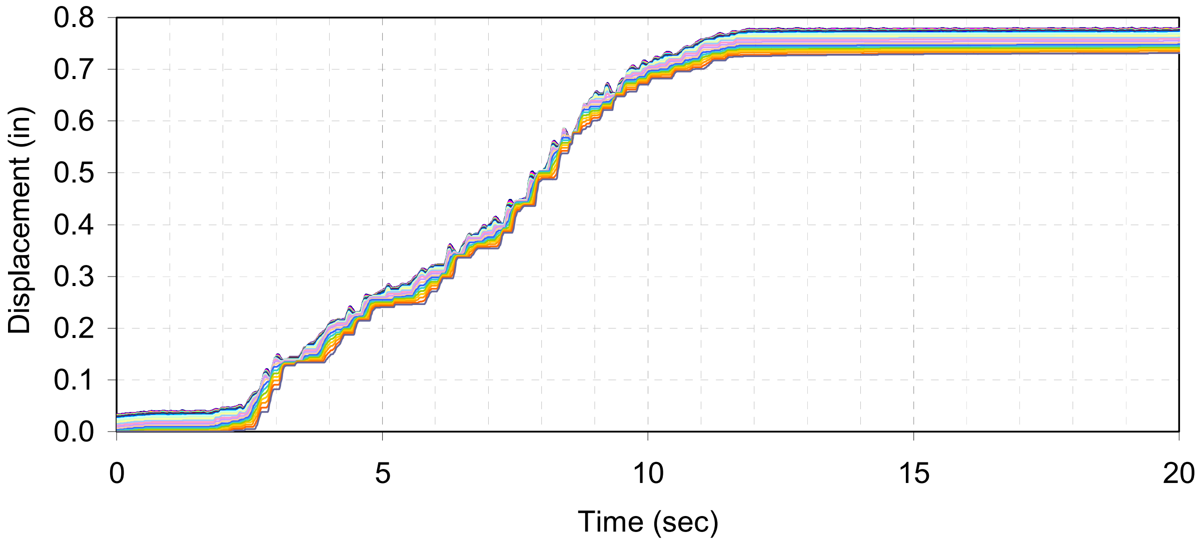

Figure 14. Time history of static plus seismic sliding displacements for San Fernando record (1 in. = 25.4 mm)

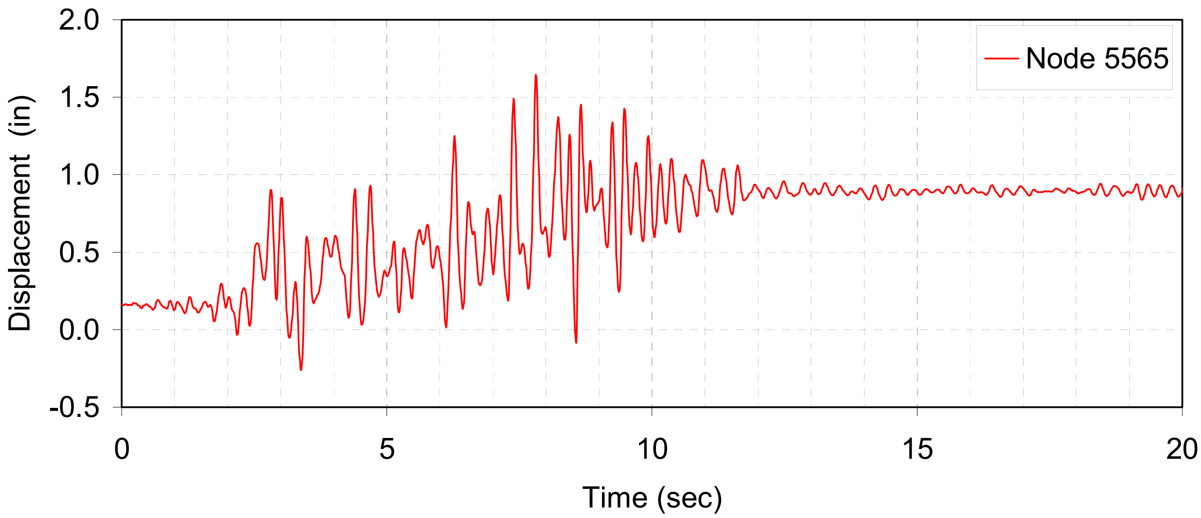

Figure 15. Time history of total horizontal displacement of dam crest for San Fernando record (1 in. = 25.4 mm)