Chapter 1
Introduction

1-1. Purpose

This manual describes procedures for the linear-elastic time-history dynamic analysis and development of acceleration time-histories for seismic design and evaluation of concrete hydraulic structures. The manual provides guidance on the formulation and performance of the linear-elastic time-history dynamic analyses and how the earthquake input time-histories are developed and applied. Time-history dynamic analysis is employed as the final design and evaluation procedure to compute the probable seismic behavior of a concrete hydraulic structure in accordance with the progressive method of analysis described in Engineer Regulation (ER) 1110-2-1806 and Engineer Manual (EM) 1110-2-6050.

1-2. Applicability

This manual applies to USACE Commands having responsibility for Civil Works projects.

1-3. Scope

This chapter provides an overview of the earthquake performance evaluation process for concrete hydraulic structures, and summarizes the methodologies for time-history dynamic analysis and development of acceleration time-histories described in following chapters. In Chapter 2, methodology for the time-history dynamic analysis of hydraulic structures is formulated, including a general description of structures, structural modeling, interaction with water and foundation rock, energy absorption at the reservoir boundaries, and the required earthquake input acceleration time-histories for each structure type. Chapter 3 describes computational methods and algorithms for solution of the equations of structural dynamics in the time and frequency domains. In Chapter 4, methodologies for performance evaluation and qualitative estimation of the probable level of damage are discussed. Chapter 5 presents methodologies and procedures for development of earthquake input acceleration time-histories and discusses important factors that should be considered in their selection and development. Chapter 6 provides examples of time-history evaluation for major concrete hydraulic structures including a gravity dam, a concrete arch dam, an inclined intake tower, and a W-frame lock structure. Concrete hydraulic structures are built from plain or lightly reinforced concrete that respond to earthquake in a less ductile manner than normal reinforced concrete building. Such behavior combined with complicated structure-foundation and structure-water interaction effects often requires special time-history analysis for the earthquake performance evaluation of major concrete hydraulic structures.

1-4. References

Required and related publications are listed in Appendix A.

1-5. Explanation of Symbols and Acronyms

Symbols and acronyms used in this manual are explained in the Notation (Appendix F).

1-6. Responsibilities of Project Team

a. Project team concept. Time-history dynamic analysis and selection or development of earthquake input acceleration time-histories for the design and evaluation of concrete hydraulic structures require the close collaboration of a project team that includes the principal design engineer, seismic structural analyst, materials engineer, and geotechnical specialists. The principal design engineer is the leader of the project team and has overall responsibility for the design or evaluation of the structure. The seismic structural analyst plans, executes, and evaluates the results of seismic analyses of the structure for earthquake ground motions for the design earthquakes. The materials engineer characterizes the material properties of the structure based on test data and appropriate assumptions. The geotechnical specialists conduct evaluations to define the design earthquakes and input ground motions and also characterize the properties of the soils or rock foundation for the structure. Any potential for seismically induced failure of the foundation is evaluated by the geotechnical specialists. The geotechnical evaluation team typically involves the participation of geologists, seismologists, and geotechnical engineers who closely work with the principal design engineer and the seismic structural analyst.

b. Consulting technical experts. Time-history evaluation of hydraulic structures is a highly complex field of earthquake engineering, which requires special expertise and substantial judgment to be effective. In many instances, the project team should augment the in-house staff with technical experts to ensure independent review of the methodology and results, to add credibility to the results, and to ensure public acceptance of the conclusions. Such experts should be selected and their responsibilities defined in accordance with ER 1110-2-1806.

1-7. Overview of Seismic Design and Evaluation Procedure

Seismic design and evaluation of hydraulic structures generally consist of the following steps:

- Selection of design/or evaluation earthquakes.
- Selection of method of analysis.
- Development of acceleration time-histories.
- Definition of load combinations.
- Development of structural models.
- Definition of material properties and damping.
- Selection of numerical analysis procedures.
- Determination of performance and probable level of damage, if any.

a. Design earthquake criteria. Design and safety evaluation earthquakes for concrete hydraulic structures are the operating basis earthquake (OBE) and the maximum design earthquake (MDE) as required by ER 1110-2-1806.

(1) Operating Basis Earthquake (OBE). The OBE is defined in ER 1110-2-1806 as an earthquake that can reasonably be 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. The associated performance requirement is that the project function with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service; therefore, alternative choices of return period for the OBE may be based on economic considerations. The OBE is determined by probabilistic seismic hazard analysis (PSHA). The response spectrum method of analysis described in 1-7b(2) is usually adequate for the OBE excitation, except for the severe OBE ground motions capable of inducing damage. In these situations, the time-history analysis described in this manual may be required.

(2) Maximum Design Earthquake (MDE). The MDE is defined in ER 1110-2-1806 as the maximum level of ground motion for which a structure is designed or evaluated. The associated performance requirement is that the project performs without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may be tolerated.

(a) For critical structures ER 1110-2-1806 requires the MDE to be set equal to the maximum credible earthquake (MCE). Critical structures are defined as structures whose failure during or immediately following an earthquake could result in loss of life. The MCE is defined as the greatest earthquake that can reasonably be expected to be generated by a specific source on the basis of seismological and geological evidence (ER 1110-2-1806).

(b) For other than critical structures the MDE is selected as a less severe earthquake than the MCE, which provides for an economical design meeting specified safety standards. This less severe earthquake is chosen based upon an appropriate probability of exceedance of ground motions during the design life of the structure (ER 1110-2-1806). In these cases, the MDE is defined as that level of ground motion having as a minimum a 10 percent probability of exceedance in 100 years. This corresponds to a return period of 950 years for a project with a service life of 100 years.

(c) Design and evaluation of hydraulic structures for the MDE ground motion may require time-history analysis as described in 1-7b(3).

b. Method of analysis. Seismic analysis of concrete hydraulic structures, whenever possible, should start with simplified methods and progress to a more refined analysis as needed. A simplified analysis establishes a baseline for comparison with the refined analyses, as well as providing a practical method to determine if seismic loading controls the design, and thereby offers useful information for making decisions about how to allocate resources. In some cases, it may also provide a preliminary indication of the parameters significant to the structural response. The simplified methods for computation of stresses and section forces consist of the pseudo-static or single-mode response-spectrum analysis. The simplified method for sliding and rotational stability during earthquake excitation is usually based on the seismic coefficient method. The permanent sliding displacements may be computed using Newmark's rigid block model or its numerous variants. The response-spectrum mode superposition described in EM 1110-2-6050 is the next level in the progressive method of dynamic analysis. The response-spectrum mode superposition fully accounts for the multimode dynamic behavior of the structure, but it is limited to the linear-elastic range of behavior and provides only the maximum values of the response quantities. Finally, the time-history method of analysis is used to compute deformations, stresses, and section forces more accurately by considering the time-dependent nature of the dynamic response to earthquake ground motion. This method also better represents the foundation-structure and fluid-structure interaction effects.

(1) Simplified procedures.

(a) Simplified procedures are used for preliminary estimates of stresses and section forces and sliding and rotational stability due to earthquake loading. The traditional seismic coefficient is one such procedure employed primarily for the analysis of rigid or nearly rigid hydraulic structures. In this procedure the inertia forces of the structures and the added mass of water due to the earthquake shaking are represented by the equivalent static forces applied at the structure center of gravity and at the resultant location of the hydrodynamic pressures. The inertia forces are simply computed from the product of the structural mass or the added mass of water times an appropriate seismic coefficient in accordance with ER 1110-2-1806. The static equilibrium analysis of the resulting inertia forces together with the customary static forces will then provide an estimate of the stresses and section forces.

(b) The sliding stability is determined on the basis of the limit equilibrium analysis. The sliding factor of safety is computed from the ratio of the resisting to driving forces along a potential failure surface. The resisting forces are obtained from the cohesion and frictional forces and driving forces from the resultant of static and seismic forces in the tangential direction of the sliding surface. When the factor of safety against sliding is not attainable, the sliding may occur as the ground acceleration exceeds a critical acceleration a_c and diminish as the acceleration falls below a_c. If a hydraulic structure is treated as a rigid block, the critical acceleration a_c is estimated from the seismic inertia forces necessary to initiate sliding. The upper bound estimate of permanent sliding displacement may be obtained using Newmark's charts (Figure 2-11 of EM 1110-2-6050).

(2) Response-spectrum modal analysis. The maximum linear elastic response of concrete hydraulic structures can be estimated using the response-spectrum mode superposition method described in EM 1110-2-6050. The procedure is suitable for the design, but it can also be used for the evaluation of hydraulic structures subjected to low or moderate ground motions that are expected to produce linear elastic response. In response-spectrum analysis, the maximum values of displacements, stresses, and section forces are first computed separately for each individual mode and then combined for all significant modes and multicomponent earthquake input. The modal responses due to each component of ground motion are combined using either the square root of the sum of the squares (SRSS) or the complete quadratic combination (CQC) method. The SRSS combination method is adequate if the vibration modes are well separated. Otherwise the CQC method may be required to account for the correlation of the closely spaced modes. Finally the maximum response values for each component of ground motion are combined using the SRSS or percentage methods in order to obtain the maximum response values due to multicomponent earthquake excitation. The response-spectrum method of analysis, however, has certain limitations that should be considered in the evaluation of results. All computed maximum response values including displacements, stresses, forces, and moments are positive and generally nonconcurrent. Therefore, a plot of deformed shapes and static equilibrium checks cannot be performed to validate the results. For computation of section forces from element stresses, appropriate signs should be assigned to the stresses by careful examination of deflected shapes of the predominant response modes. Alternatively, section forces may be computed first for each individual mode and then combined for the selected modes and multicomponent earthquake input, a capability that may not exist in most finite-element computer programs. Other limitations of the response-spectrum method are that the structure-foundation and structure-water interaction effects can be represented only approximately and that the time-dependent characteristics of the ground motion and structural response are ignored.

(3) Time-history analysis. Time-history earthquake analysis is conducted to avoid many limitations of the response-spectrum method and to account for the time-dependent response of the structure and better

representation of the foundation-structure and fluid-structure interaction effects. The earthquake input for time-history analysis is usually in the form of acceleration time-histories that more accurately characterize many aspects of earthquake ground motion such as the duration, number of cycles, presence of high-energy pulse, and pulse sequencing. Time-history analysis is also the only appropriate method for estimation of the level of damage as described in 1-7*h* and Chapter 4. Response history is computed in the time domain using a step-by-step numerical integration or in the frequency domain by applying Fourier transformation described in 1-7*g*.

c. Development of acceleration time-history input motions. Chapter 5 describes the procedures for developing site-specific acceleration time-histories of ground motion for dynamic analysis of hydraulic structures. 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 specific structures. The following steps are included in this process:

(1) 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). If sufficient recorded motions are not available, simulated recorded time-histories can be developed using ground motion modeling methods.

(2) Modifying time-histories selected in (1) 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.

(3) Further modifying the time-histories for site response effects, if the site is a soil site and the time-histories have been developed for outcropping rock conditions.

(4) Further modifying the time-histories for spatial variations of ground motion, if it is desired to incorporate effects of wave passage and incoherence in the ground motions that would arrive beneath a very large or long structure.

d. Load combinations. Concrete hydraulic structures should be designed and evaluated for three basic *usual*, *unusual*, and *extreme* loading combinations. In general, the usual loading combinations are formulated based on the effects of all applicable static loads that may exist during the normal operation of the structure such as the usual concrete temperatures and the most probable water level, with dead loads, tailwater, ice, uplift, and silt. The unusual static loading combinations refer to all applicable static loads at the floodwater pool elevation with the effects of mean concrete temperatures, dead loads, and silt. The unusual dynamic loading combination includes the OBE loading plus any of the usual loading combinations. Extreme loading combinations consist of the effects of the MDE loading plus any of the usual loading combinations.

(1) Combination with usual static loads. Time-history dynamic analysis is conducted mainly for the MDE loading conditions but also for the OBE if seismic demand is severe. At each time-step, results of such analyses should be combined with results of any of the usual loading combinations in order to obtain total displacements, stresses, and section forces needed for design or evaluation of structures.

(2) Combination for multicomponent earthquake input. Modeled as two- or three-dimensional (2-D or 3-D) structural systems, time-history analysis of concrete hydraulic structures should consider two or three orthogonal components of acceleration time-histories of earthquake ground motions. At each time-step,

response quantities of interest are first computed for each component of the earthquake input and then combined algebraically to obtain the total responses due to two or all three components. Only scalar and similarly oriented response quantities are combined algebraically. After the initial algebraic combination, the resulting displacements, shear forces, and moments in orthogonal directions need to be combined vectorially if the absolute maximum values of such response quantities are required.

(3) Combination for earthquake input direction (phase relation). Seismic waves of identical amplitudes, but traveling in two opposite directions, could lead to different structural response. The opposite of acceleration time-histories (i.e., all values multiplied by minus one) should also be considered as a simple way to account for some directional effects. In general, a complete permutation of all three components with positive and negative signs may be required to obtain the most critical directions that would cause the largest structural response.

e. Development of structural models. Meaningful time-history analysis of probable seismic behavior of a concrete hydraulic structure for design and evaluation requires thorough understanding of the system components, their interaction, and their material properties. Modeling of the structural system and its interaction with the foundation and water are summarized in this section. The required material properties for the analysis are specified in *f* below. In general, structural models for the time-history analysis should be developed to capture the main dynamic characteristics of the structure and represent the effects of fluid-structure interaction and foundation-structure interaction accurately. Depending on the geometry and mass and stiffness distributions, a particular hydraulic structure may be idealized using a simple beam, a 2-D finite element, or a 3-D finite element model. The structural model should provide an accurate representation of the mass and stiffness distributions, and in the case of existing structures it should account for the effects of any existing cracks, deteriorated concrete, or any deficiency that might affect the stiffness. The fluid-structure interaction effects may be adequately represented by simple added hydrodynamic mass coefficients, or may require a finite element (or boundary element) solution with or without the effects of water compressibility and boundary absorption. Modeling of the foundation-structure effects may range from a simplified massless finite element mesh to more elaborate formulations involving soil-structure or soil-pile-structure interaction analyses. For embedded structures, the effects of dynamic backfill pressures on the structure can also be significant and should be considered.

(1) Concrete gravity dams.

(a) Relatively long and straight concrete gravity dams built as independent monoliths separated by transverse joints may be idealized using a 2-D finite element model including the foundation rock and the impounded water. The 2-D dam-water-foundation model, usually of the tallest cross section, may be analyzed as three separate systems in the frequency domain using the substructure method (2-12*a*(1)) or as a single complete system in the time domain using the standard finite element procedures (2-12*a*(2)). The substructure method may be employed if the assumption of homogeneous material properties for the foundation region can be judged reasonable and a more rigorous formulation of the dam-water interaction including water compressibility and reservoir bottom absorption is desirable. Otherwise the standard finite element method with much simpler added-mass representation of the dam-water interaction should be used in order to account for variation of the foundation rock properties.

(b) Curved concrete gravity dams and those built in narrow canyons should be analyzed using 3-D finite element models similar to those described for arch dams in (2) below.

(2) Concrete arch dams. The complicated 3-D geometry of an arch dam requires a rather refined 3-D model of the dam, its foundation, and the impounded water for evaluation of its response to all three components of seismic input (2-13). The arch dam-water-foundation system may be formulated in the time

domain using the standard finite element procedures or in the frequency domain using the substructure method. The standard method employs a massless foundation rock included as part of the dam finite element model in conjunction with an incompressible liquid mesh representing the impounded water. Treating each system separately, the substructure method considers the same dam model as the standard method, but employs the flexibility as well as the damping and inertial effects of the foundation rock, with a reservoir water that accounts for the effects of water compressibility and the reservoir boundary absorption. In both methods the seismic input consists of three components of the free-field acceleration time-histories applied uniformly along the dam-foundation interface in the substructure method and at the fixed boundary of the massless foundation in the standard method. The standard method provides reasonable results for small dams and those built on a competent foundation rock having a deformation modulus at least equal that of the concrete and with impounded water whose fundamental resonance frequency is at least twice that of the arch dam. Otherwise, the more rigorous formulation of the dam-water and dam-foundation interaction effects offered by the substructure method might be required.

(3) Intake-outlet towers. Freestanding towers may be idealized using the substructure or standard finite element method of analysis. The available substructure method is restricted to towers having two axes of plan symmetry and supported on the horizontal ground surface of a foundation with homogeneous material properties (2-14*a*(2)). The substructures consist of the tower, surrounding water, contained water, and the foundation rock or soil system. The tower is modeled as an assemblage of beam elements. The hydrodynamic forces and moments due to pressures on the outside and inside surfaces of the tower are determined by the finite element and boundary integral procedures; and the foundation region is represented by the frequency-dependent stiffness or impedance functions. The seismic input for the tower-water-foundation system is defined by two horizontal components of the free-field acceleration time-histories applied at the base of the tower. The effects of vertical component of the ground motion are considered negligible and thus are ignored.

(a) The standard method is restricted neither to symmetric towers nor to homogeneous foundation material properties, but employs foundation models that only approximately account for the structure-foundation interaction (2-14*a*(1)(a)). Towers with regular cross-section geometry may adequately be represented by beam elements in conjunction with a foundation model idealized by equivalent linear springs attached to the base or to the embedded portion of the tower model. The water-structure interaction for such a model is approximated by the added hydrodynamic mass described in 2-19*d*. The earthquake input for the tower beam model is defined by two horizontal components of acceleration time-histories. The irregular freestanding towers may require finite-element idealization, in which case a finite element model should also be developed for an appropriate portion of the foundation region. The seismic input for such a model should include two horizontal components and possibly the vertical component of the free-field ground acceleration histories.

(b) The 3-D geometry of supported towers combined with seismic input exciting the tower not only at the base but also at the abutment supports requires a 3-D finite element treatment of the structure. Three-dimensional solid elements or a combination of 3-D solid and shell elements may model the tower. An appropriate portion of the foundation and the abutment support regions should be modeled as part of the finite element model of the tower. The effects of hydrodynamic pressures are represented by the equivalent added mass concept but should be computed using the finite element or boundary element procedures to accurately account for the tower geometry as well as topography of the surrounding foundation-abutment region. The seismic input for the analysis of a supported tower includes all three components of the ground acceleration time-histories applied at fixed exterior nodes of the foundation model.

(4) U-frame and W-frame navigation locks. Navigation locks constructed of separate monoliths are generally idealized using 2-D finite element procedures. The lock itself is modeled using the standard finite element method, but dynamic interactions with the soil-pile foundation, backfill soils, and the contained and

surrounding water require special considerations and must be represented accurately. The soil-pile-structure interaction (SPSI) effects can be incorporated in the analysis by two different approaches: direct method (2-15*a*) and substructure method (2-15*b*). In the direct method a complete model of the soil-pile-structure system is developed and subjected to the vertical and one horizontal component of the seismic input usually prescribed at the rock outcrop. In the substructure method, the lock structure and the foundation soil including the piles are treated separately. The lock structure is modeled by the standard finite element method, and the soil-pile foundation is represented either by impedance functions in the form of frequency-dependent springs and dashpots or by simple frequency-independent springs attached to the base of the structure. To obtain conservative values of forces and moments at pile heads and to model imperfect contact and possible separation along the structure-foundation interface, the finite element model should also incorporate a thin soft-soil layer beneath the base of the lock structure. The model is then subjected to a foundation-input motion developed on the basis of kinematic interaction analysis. In both methods, the water-structure interaction effects are adequately represented by the added hydrodynamic mass described in 2-15*a*(3).

(5) Massive concrete lock walls. Time-history analysis of lock walls founded on rock with no backfill soils is evaluated using the standard finite element method described for concrete gravity dams in 2-12*a*(2). Analysis of lock walls with backfill soils involves consideration of dynamic soil pressures induced by the ground shaking in addition to the interactions with the foundation rock and water. Depending on the expected movement of the backfill soil, it may be modeled as yielding backfill, nonyielding backfill, or an intermediate case in accordance with 2-16*b*. The yielding backfill, which may induce a limit or failure state, is analyzed based on the well-known Mononobe-Okabe method. The nonyielding backfill responding within the linear elastic range of deformations may be modeled using a constant-parameter single-degree-of-freedom (SDOF) model or a more elaborate frequency-independent multi-degree-of-freedom (MDOF) lumped-parameter system. The intermediate case is evaluated as an equivalent linear system using an SPSI model.

(6) Massive concrete guide walls. Fixed guide walls supported on cellular piles or drilled shafts are best represented by 3-D soil-structure-interaction models (2-17). The 3-D model may include only one cellular pile or drilled shaft, but whenever possible may take advantage of the structural symmetry to reduce the size of the problem. The overall model usually consists of various components including the cellular sheet pile, soil layers and soil within the sheet pile, concrete pier or block above the ground surface, precast concrete beams, and the hydrodynamic forces acting on the pile and the concrete block. The seismic input includes three components of acceleration time-histories usually prescribed at the rock outcrop.

f. Material properties. Concrete hydraulic structures are built using both plain and lightly reinforced forms of concrete construction and may be supported by rock, soil, or pile foundations. Concrete condition, function, age, and properties for existing structures and concrete mix and properties for new designs usually vary widely from structure to structure. These factors and geotechnical information of the subsurface conditions have potentially significant influence on the seismic performance of concrete hydraulic structures. It is essential that the time-history seismic evaluation effort conform to guidelines for determination of material properties and assessment of physical condition described in other references. The primary material properties relevant to time-history dynamic analysis are summarized in the following paragraphs.

(1) Concrete properties. The primary material properties of interest in a concrete structure are those that affect prediction of the structural response and those that are required for evaluation of the structural performance. The structural response is predicted on the basis of unit weight and elastic properties of the concrete including modulus of elasticity and Poisson's ratio. Many laboratory and field measurements have shown that modulus of elasticity is affected by the rate of loading and generally is higher for the dynamic than it is for the static loading. Under the sustained static loading conditions, the effects of creep on the mass concrete may be important and generally can be considered by determining a sustained modulus of elasticity taken as 60 to 70 percent of the laboratory value of the instantaneous modulus of elasticity. For seismic

analyses the measured or estimated dynamic modulus is more appropriate and should be used. In the absence of measured data, dynamic modulus of elasticity should be obtained by increasing the laboratory value of the instantaneous modulus by 20 to 30 percent. Compressive and tensile strengths of concrete are properties used to evaluate acceptability of new designs or seismic performance of the existing structures. Like modulus of elasticity, concrete strength parameters are also affected by the rate of loading. Seismic design and performance evaluation of concrete hydraulic structures should therefore be based on the measured or estimated dynamic strength of concrete. Other material properties such as shear strength of concrete, tensile and shear strengths of construction joints, yield strength and modulus of elasticity of reinforcing steel, and reinforcing steel bond strength and ductility may also be required. In general tensile strength across the deteriorated or poorly constructed joints could significantly be lower than that of the parent concrete. Determination of tensile and shear strengths across such joints may be warranted under severe earthquake loading.

(2) Foundation rock properties. Foundation rock properties for use in structural analyses include shear strength and rock mass modulus of deformation. Procedures for estimating shear strength and modulus of deformation are described in Chapter 10 of EM 1110-2-2201. Shear strength parameters provide a measure of shearing resistance to sliding at the structure-rock interface or within the foundation and abutments, when potential sliding wedges or planes of rock that could cause instability have been identified. The modulus of deformation is a measure of foundation deformations for the rock mass as a whole including the effects of its discontinuities. In contrast, modulus of elasticity is determined for an intact specimen of the rock.

(3) Foundation soil properties. Foundation soil properties for use in soil-structure or SPSI studies include low-strain shear wave velocity or shear modulus of soil layers, mass density, Poisson's ratio, material damping, and variation of shear modulus and damping with strain. The low-strain material parameters are estimated from the geotechnical data such as the blow counts, measured shear wave velocities, and soil borings. Many studies have concluded that shear modulus and damping characteristics of soils vary with level of strain and that the strain dependency is a function of soil type, stress history, density state, and other factors. In practice, a set of modulus reduction and damping curves suggested by Seed and Idriss (1970) is commonly used to account for variation of these parameters with strain.

(4) Reservoir bottom absorption. Studies of the dam-water interaction indicate that the earthquake response of concrete dams is sensitive to the water energy loss at the reservoir boundaries. If the reservoir boundary materials are relatively soft, an important fraction of the reservoir water energy can be absorbed, leading to a major reduction in the dynamic response of the dam. An earthquake-generated hydrodynamic pressure wave impinging on the reservoir boundary is partly reflected in the water, and partly refracted (absorbed) into the boundary materials. The energy loss or partial absorption at the reservoir boundary is approximately represented by a reflection coefficient α, which is the ratio of reflected to incident wave amplitudes (Hall and Chopra, 1980; Fenves and Chopra 1984b). The reflection coefficient α varies between 1 and -1, where $\alpha = 1$ corresponds to a total reflection (nonabsorptive or rigid boundary), $\alpha = 0$ represents a complete absorption or transmission into the boundary materials, and $\alpha = -1$ characterizes 100 percent reflection from a boundary with an attendant phase reversal. The in situ values of α for the seismic safety evaluation of concrete dams can be measured using three independent approaches developed and employed at several dams in the United States and abroad. These include the seismic reflection and refraction techniques (Ghanaat and Redpath 1995) and a technique based on the acoustic reverberation (Ghanaat et al. 1999).

(5) Damping.

(a) In practice, damping characteristics of typical structures are generally expressed in terms of equivalent viscous damping ratios. The velocity-proportional viscous damping is commonly used because it leads to convenient forms of equations of motion. The energy-loss mechanism for the viscous damping, however, depends on the frequency of excitation, a phenomenon that has not been observed experimentally. As a result it is desirable to remove this frequency dependency by using the so-called *hysteretic* form of damping. The hysteretic damping is defined as a damping force proportional to the strain or deflection amplitudes but in phase with the velocity. The structural response provided by hysteretic damping can be made identical to that with viscous damping if the hysteretic damping factor is selected as

$$\varsigma = 2\xi\beta \tag{1-1}$$

where

ζ = hysteretic damping factor

ξ = viscous damping ratio

β = ratio of the excitation frequency to the natural free-vibration frequency

To remove the frequency dependency term from Equation 1-1, the value of hysteretic damping ζ is computed at resonance by setting $\beta = 1$. The hysteretic damping computed in this manner provides identical response to that of the viscous damping at the resonance and nearly identical response at all other frequencies for $\xi < 0.2$.

(b) Viscous damping is commonly used in the time-domain solution, whereas the hysteretic damping factor taken as twice the viscous damping ratio is usually employed in the frequency domain solution. Linear time-history analysis of concrete hydraulic structures should employ a damping equivalent to a 5 percent viscous damping ratio. However, in situations where a moderate level of nonlinear behavior such as joint opening and cracking is predicted by a linear analysis, a higher damping ratio in the range of 7 to 10 percent could be used to account somewhat for the energy loss due to nonlinear behavior.

g. Numerical analysis procedures. Computation of earthquake response history for typical concrete hydraulic structures involves solution of coupled sets of equations of motion that include large numbers of equations or degrees of freedom. In linear response analyses the system equations of motion can be formulated either in the time domain or in the frequency domain. Only time domain formulation is suited to analysis of nonlinear response. These formulations and the corresponding response analysis procedures are described in Chapters 2 and 3, respectively. Following is a brief summary to provide a general idea of how these techniques are applied in the solution of the earthquake response behavior of concrete hydraulic structures.

(1) Analysis in the time domain. In practice, time-domain response analyses are generally based on some forms of step-by-step methods using numerical integration procedures to satisfy the equations of motion. In all the step-by-step methods the loading and the response history are divided into a sequence of time intervals or “steps.” The response during each step is computed from the initial conditions (displacement and velocity) at the beginning of the step and from history of loading during the step. The structural properties within each step are assumed to remain constant, but could vary from one step to another (nonlinear behavior) or remain the same during all time-steps (linear behavior). The step-by-step methods may be classified as explicit or implicit. In an explicit method, the new response values calculated in each step depend only on the response quantities available at the beginning of the step. The analysis therefore proceeds directly from one step to the next. In an implicit method, on the other hand, the new response values for a given step include one or more values pertaining to the same step, so that trial values and successive iterations are necessary. The iteration within a step makes implicit formulations inconvenient and in some cases even prohibitive. Only explicit methods such as those described in Chapter 3 may be considered. The primary factors to be considered in

selecting a step-by-step method include efficiency, round-off and truncation errors, instability, phase shift or apparent change of frequency, and artificial damping in accordance with Chapter 3.

(a) Mode superposition method. In linear response analysis, the mode superposition techniques can be used to uncouple the system equations of motion, so that the dynamic response can be obtained separately for each mode of vibration and then superimposed for all significant modes to obtain the total response. This way the step-by-step integration discussed in (1) above is applied separately to a number of independent SDOF equations and then the resulting modal response histories are superimposed to compute the total response of the structure. The main effort in this method includes computation of eigenvalue problems followed by modal coordinate transformation to uncouple the MDOF dynamic analysis to the solution of a series of SDOF systems. It is important to note that the equations of motion will be uncoupled only if the damping can be represented by a mass proportional and stiffness proportional damping matrix known as Rayleigh damping. The Rayleigh damping is suitable when the damping mechanism is distributed rather uniformly throughout the structure.

(b) Direct step-by-step method. In this method, the step-by-step integration is applied directly to the original equations of motion with no need for modal coordinate transformation to uncouple them. Thus there is no need to obtain natural mode shapes and frequencies or to limit damping to the proportional type. The method can be used for both the linear and nonlinear response analyses.

(2) Analysis in the frequency domain. An alternative approach to solving the modal equations of motion for linear systems is to perform the analysis in the frequency domain. In particular, when the equation of motion contains frequency-dependent parameters such as foundation stiffness and damping, the frequency domain approach is much superior to the time domain approach. In simple terms the frequency domain solution involves expressing the ground motion in terms of its harmonic components; evaluating the response of the structure to each harmonic component; and superposing the harmonic responses to obtain total structural response. In this process, the harmonic amplitudes of the ground motion in the first step and superposition of harmonic responses in the third step are obtained using the Fast Fourier Transform (FFT) algorithm.

h. Structural performance and damage criteria. Chapter 4 describes methodologies and procedures for evaluation of earthquake performance and qualitative estimation of the probable level of damage using the results from linear time-history analyses. The overall process involves describing the results in terms of the demand-capacity ratios, cumulative inelastic duration of excessive stresses or forces, and spatial extent and distribution of high-stress or high-force regions, and then comparing them with a set of acceptance criteria set forth for each type of structure. Another important factor in the evaluation process is consideration of probable nonlinear mechanisms and modes of failure that might develop in a concrete hydraulic structure. The damage in a particular structure is considered to be minor and the linear time-history analysis will suffice if estimated level of damage meets the acceptance requirements established for that structure. Otherwise the damage is considered to be severe, in which case a nonlinear time-history analysis would be required to estimate damage more accurately.

(1) Gravity dams. The dam response to the MDE is considered to be within the linear elastic range of behavior if the computed stress demand-capacity ratios are less than or equal to 1.0. The level of nonlinear response or cracking is considered acceptable if demand-capacity ratios are less than 2, overstressed regions are limited to 15 percent of the dam surface area, and the cumulative duration of stress excursions beyond the tensile strength of the concrete falls below the performance curve shown in Figure 4-2.

(2) Arch dams. The dam response to the MDE is considered to be nearly within the linear elastic range if the computed stress demand-capacity ratios are less than or equal to 1.0. The dam is considered to exhibit

nonlinear response in the form of opening and closing of contraction joints and cracking of lift lines if the estimate demand-capacity ratios exceed 1.0. The amount of joint opening and cracking is considered acceptable if demand-capacity ratios are less than 2, overstressed regions are limited to 20 percent of the dam surface area, and the cumulative inelastic duration falls below the performance curve given in Figure 4-18.

(3) Navigation locks. If all computed demand-capacity ratios for piles and concrete sections are less than or equal to 1, then the lock structure and piles are expected to respond elastically with no damage. Demand-capacity ratios of greater than 1 show the structure will experience nonlinear behavior in the form of yielding of steel members and cracking or crushing and spalling of the concrete. On the basis of linear time-history analysis the level of damage or nonlinear response will be acceptable if the following performance conditions are met:

- Pile demand-capacity ratios remain below 1.5 and the percentage of piles yielding with the associated cumulative yield duration fall below the performance curves given in paragraph 4-5a (Figures 4-25 and 4-26). For example, for the "*Expected Yield*" case, the yielding should be limited to less than 10 percent of piles and the cumulative yield duration should not exceed one-tenth of a second.

- Demand-capacity ratios of concrete sections should not exceed 1.5 and those exceeding one be limited to less than 10 percent surface area of the lock.

(4) Free-standing intake towers. If the computed demand-capacity ratios are less than or equal to 1.0, the tower is considered to respond essentially within the linear elastic range with negligible or no damage. Demand-capacity ratios of greater than 1 indicate the tower would suffer some damage in the form of yielding of steel members and cracking or crushing and spalling of the concrete. In this situation, the damping ratio of 5 percent for the linear time-history analysis should be increased to 7 percent if demand-capacity ratios are approaching 2 and to 10 percent if they exceed 2. After adjustment for the damping, the damage is considered moderate and acceptable if the following conditions are met:

- Bending moment demand-capacity ratios computed on the basis of linear time-history analysis remain less than two.

- Cumulative duration of bending-moment excursions above demand-capacity ratios of 1 to 2 fall below the acceptance curve given in Figure 4-53.

- The extent of yielding along the height of tower (i.e., plastic hinge length for demand-capacity ratios of 1 to 2) is limited and falls below the acceptance curve.

Otherwise, the damage is considered to be severe and should be assessed using nonlinear analysis procedures.