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No. 1110-2-12

30 September 1995

Engineering and Design
SEISMIC DESIGN PROVISIONS FOR ROLLER COMPACTED CONCRETE DAMS

1. Purpose

The purpose of this engineer pamphlet (EP) is to provide preliminary guidance and direction for the earthquake-resistant design of new roller compacted concrete (RCC) dams, and for the evaluation of safety and serviceability of existing RCC dams subjected to earthquake loading.

2. Applicability

This EP applies to all HQUSACE elements and USACE commands having responsibilities for the design of civil works projects.

3. Discussion

a. This EP presents preliminary guidance concerning the design of new RCC dams and the evaluation of existing RCC dams located in zones of high seismic activity. References are included in Appendix A.

b. Appendices B-D present examples of applying this guidance to the design of a new RCC dam.

c. Both the preliminary guidance contained herein and the example problems are based on EM 1110-2-2200 and ER 1110-2-1806. Both of these documents are under revision and the final guidance contained in these documents may vary somewhat from the provisions of this EP. Draft copies of these documents may be obtained from CECW-ED for use in the design of RCC structures.

d. A dynamic stress analysis shall be performed as part of the design procedure for all new RCC dams, or the evaluation of existing RCC dams, located in areas of strong seismicity. Dams shall be shown capable of satisfying general performance requirements for design earthquake seismic events described herein. Linear-elastic analysis methods shall be used in performing dynamic stress analysis.

e. Consultation and approval of CECW-ED are required prior to performing a nonlinear dynamic stress analysis based upon the theory of fracture mechanics to qualify a new design or to evaluate an existing RCC dam with regard to dam safety.

FOR THE COMMANDER:

ROBERT H. GRIFFIN
Colonel, Corps of Engineers
Chief of Staff

CECW-ED

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Chapter 1
Introduction

1-1. General

Roller compacted concrete (RCC) dams are designed in accordance with EM 1110-2-2200. The proportions of the RCC dam are derived by stability analysis in a manner identical to that for a conventional concrete gravity dam and are governed by the static forces to be resisted and not by the dynamic forces generated during seismic activity. After the geometric proportions are determined based on the static loads a dynamic analysis is conducted. Zones requiring superior RCC mixes are established, and vibratory compaction methods and joint preparation methods which affect the RCC tensile strength are also established based on the criteria provided in this engineer pamphlet (EP).

1-2. References

Required and related publications are listed in Appendix A.

1-3. Explanation of Terms

Abbreviations, symbols, and notations used throughout this EP are explained in the glossary.

1-4. Background

Basic criteria and guidance for the design of RCC dams are provided in EM 1110-2-2200. ER 1110-2-1806 provides guidance on analysis methods and procedures for new designs and an investigative program for existing dams. ETL 1110-2-301 gives additional information on specifying earthquake ground motions for a particular site. ETL 1110-2-303 provides guidance on finite element dynamic analysis methods and on evaluating the severity of cracking based on tensile stresses from the linear analysis. EM 1110-2-2006 provides guidance concerning RCC usage and mix design.

1-5. Design Philosophy

a. Response spectrum analysis. The nonlinearities associated with concrete behavior under seismic loading are difficult to assess and beyond practical analyzing capabilities of most design offices. Procedures which permit the use of a linear-elastic type of dynamic analysis adjusted to provide a reasonable but conservative approximation of the nonlinear behavior are adequate in almost all design situations. The philosophy of design followed in this EP will be to establish the procedures applicable to the majority of design situations. This consists of providing in some detail the requirements for performing the linear-elastic response spectrum analysis and the criteria for evaluating the results.

b. Refined analyses. For the few occasions where this approach does not produce a satisfactory design or where an existing dam does not satisfy criteria, the designer is then advised to pursue the more refined analysis methods. Should the even more complex nonlinear analysis become necessary, it should be performed under the guidance of a recognized expert in this specialized field and should only be undertaken with approval of CECW-ED.

1-6. Design Earthquakes

The linear-elastic response spectrum method of analysis is the simplest dynamic analysis method and provides adequate results for most designs. The ground motion is usually defined by design response spectra scaled to peak ground accelerations (PGA) for the two design earthquakes described below.

a. Operating basis earthquake. The operating basis earthquake (OBE) is defined as the earthquake producing the greatest level of ground motion that is likely to occur at the site during the economic life of the dam.

b. Maximum credible earthquake. The maximum credible earthquake (MCE) is defined as the earthquake which produces the greatest level of ground motion at the site as a result of the largest magnitude earthquake that could reasonably occur along the recognized faults or within a particular seismic source.

c. Types of design spectra. Design response spectra for the OBE are usually developed using a probabilistic approach, and design response spectra for the MCE are developed using a deterministic approach. Design response spectra are further classified into two types: (1) site-specific or (2) standard. The seismic zone location of the site, the height of the dam, and the proximity to active faults are the factors used to determine if it is necessary to develop a site-specific design response spectra or if the standard spectra may be used in the dynamic analysis. When standard design response spectra are acceptable, Chapter 5 provides the appropriate spectra along with the PGA values to be used for scaling. These standard design spectra are based on the mean level of the ground motion parameters for the records selected in the development of the standard spectra.

d. Ground motion time histories. The more refined analysis methods require a ground motion time history representation of the design earthquakes. These may be developed using actual past earthquake ground motion records, synthetically, or by modifying an actual record. Ground motion time histories are developed so their response spectrum closely matches the site-specific design response spectrum.

1-7. Acceptance Criteria

a. Cracking of RCC. The ground motion that is produced during a seismic event can cause cracks to occur in an RCC dam. As cracking progresses, serviceability is eventually impaired. If ground shaking is extremely severe, or if strong ground shaking combines with a foundation fault displacement, it is conceivable that continued propagation of the system of cracks could eventually lead to a failure mechanism where the dam is no longer capable of containing the pool. This EP establishes acceptance criteria which maintain serviceability during an OBE, and provide a reasonable safety factor against developing a failure mechanism during a MCE. Because of the complexity and the great number of variables involved in seismic design, the EP criteria should be supplemented with the judgment of structural engineers experienced in seismic design.

b. Direct tensile strength. The direct tensile strength of the RCC is the design parameter used for establishing the acceptance criteria. Unlike conventional concrete, tensile strength of RCC depends on mix consistency and placement and compaction methods as well as mix proportions. Tensile strength of both the lift joint and the parent concrete shall be determined from cores taken from test fill placements for new dam design and from the in-place RCC for existing dams. Although splitting tensile tests may be used, the test results shall be adjusted to reflect direct tensile strength. From the direct tensile strength, the allowable design tensile stresses shall be established for both lift joints and parent concrete by applying adjustment factors to account for high strain rate associated with dynamic loading and certain nonlinear characteristics of the stress/strain curve. Adjustment factors shall be selected to maintain serviceability during an OBE and to produce a reasonable safety factor for a MCE.

1-8. Important Factors

Discussed below are recommendations regarding factors which are important because they have a significant impact on the dynamic response. Recommendations that differ from those contained in ETL 1110-2-303 and ER 1110-2-1806 are identified.

a. Effective damping. The material and radiation damping of the foundation contribute significantly to the damping of the combined dam-foundation system, and must be considered in the analysis. This requires calculating an effective viscous damping ratio to reflect the damping contribution of both the dam and the foundation. This will result in a considerably higher damping ratio for a foundation having a very low modulus than the damping ratio used previously.

b. Hydrodynamic effect. Added mass shall be calculated using standard hydrodynamic pressure function curves which consider compressibility of the water, stiffness characteristics of the dam, and reservoir bottom absorption (Fenves and Chopra 1986). Appendix D provides an example showing the required procedure.

c. Mode combination methods. The complete quadratic combination method (CQC) of combining modes shall be used for final design of dams under critical seismic design conditions and for evaluation of existing dams. Critical conditions are considered to exist when site-specific design response spectra are required by this EP. Either the square root of the sum of the squares method (SRSS) or the CQC method is acceptable for all preliminary designs and for final designs under noncritical seismic conditions. Since the modal frequencies are fairly well separated in gravity dams, the simpler SRSS method produces adequate results which are in balance with the general level of precision required for preliminary or noncritical analyses.

d. Seismic zone map. The seismic zone map, Figure 5-1, shall be used in the dynamic stress analysis phase of the seismic design. The peak ground accelerations for use in scaling standard design response spectra are contained in Table 5-2 and are based on the zone map. The seismic zone maps and the seismic coefficients contained in ER 1110-2-1806 shall be used only in the stability analysis phase of seismic design.

1-9. Analysis Methods and Procedure

In general a dynamic stress analysis shall be performed, and the results shall be evaluated to determine if the response of the RCC dam to the design earthquakes is acceptable. If the response is not acceptable, the design of a new dam may be modified and reanalyzed using the same analysis method, or a more refined analysis method may be employed. For an existing dam, progressively more refined methods of analysis are employed.

a. Method attributes. There are four attributes that characterize a particular dynamic analysis method.

(1) Material behavior. Options are (a) linear-elastic or (b) nonlinear behavior.

(2) Design earthquake definition. Options are (a) design response spectrum or (b) time history ground motion record input.

(3) Dimensional representation. Options are (a) two-dimensional representation or (b) three-dimensional representation.

(4) Model configuration. Options are (a) Chopra's "standardized" model, (b) composite finite element-equivalent mass system model, or (c) finite element-substructure model.

b. Computer programs. Various computer programs are available which are identified with certain analysis methods. Also, Chopra's Simplified Method may be either hand-calculated or done by a computer program. Some computer programs, such as the general purpose finite element programs, allow the attribute options to be changed so that one of several possible methods may be employed for the dynamic analysis. This often allows a transition to a more refined method without necessarily abandoning all the previous computer model input effort. Other computer programs, such as the EAGD-84 program, and Chopra's Simplified Method are single method programs since they have fixed attributes. Chapter 8 discusses dynamic analysis methods in more detail.

c. Preliminary and final design. The two-dimensional, linear-elastic, response spectrum method shall be used for the preliminary design analysis. Either Chopra's Simplified Method or a general-purpose finite element program shall be employed depending on the design conditions. The simplest final design analysis utilizes a composite finite element-equivalent mass system model and general-purpose finite element program.

1-10. Coordination

A fully coordinated team of structural engineers, geotechnical and materials engineers, geologists, and seismologists should ensure that all factors relevant to the dynamic analysis are correct and that the results of the analysis are properly evaluated. Some of the critical analysis and design aspects requiring coordination are discussed below.

a. Design response spectra. Developing site-specific design response spectra when required.

b. Tensile strength of RCC. Obtaining representative cores from test-fill placements for new dams or from the in-place concrete for existing dams for use in determining the direct tensile strength and dynamic tensile strength of both the lift joints and the parent RCC.

c. Foundation properties. Obtaining exploratory corings and evaluating tests to determine the foundation deformation modulus and other foundation properties.

d. Foundation fault displacement. Evaluating geoseismic conditions at the site to determine if foundation fault displacement is possible, and to map the location, strike, and dip of the potential faults.

Chapter 2
Seismic Design Criteria

2-1. Stability

a. Resultant location and sliding. RCC dams shall satisfy the overturning and sliding stability requirements for gravity dams using inertia forces calculated by the seismic coefficient method as set forth in EM 1110-2-2200 and ETL 1110-2-256. The seismic coefficients shall be as shown on the seismic zone maps provided in ER 1110-2-1806.

b. Extreme stability conditions. When intense ground shaking causes serious tensile cracking at the dam-foundation interface, a nonlinear time history analysis shall be performed to evaluate cracking, potential permanent displacements, and the effect these have on sliding stability. Certain stipulations regarding nonlinear analyses are covered in paragraph 2-2*g*.

2-2. Response to Ground Shaking

RCC dams shall be capable of resisting the strong motion ground shaking associated with design earthquakes within the allowable tensile stress design criteria specified in Chapter 4. Dynamic stress analysis methods and procedures are described in Chapter 8. The dynamic analyses shall incorporate the dynamic characteristics of the dam, foundation, reservoir, and backfill or silt deposition when applicable.

a. Defining ground motion. The free field ground motions are used to define the ground motion that would be felt at the site due to two design earthquakes. Free field ground motion associated with each shall be represented by design response spectra and, when required, design acceleration time histories. The design earthquakes are operating basis earthquake (OBE), and maximum credible earthquake (MCE). Both are discussed in detail in Chapter 4.

b. Propagation of cracks in RCC. Most dams with earthquake resistant provisions will probably survive the most severe earthquake shaking possible at the site with little or no damage, although high dams located near major faults have experienced extensive cracking during major earthquakes (Chopra and Chakrabarti 1973). Concrete cracking due to ground shaking combined with cracking due to foundation fault displacement could propagate to an extent where a failure mechanism is formed thus impairing the ability of the dam to contain the pool. Criteria defining an acceptable response of the dam to design earthquakes are based on initiation and propagation of tensile cracking within the RCC.

c. Analyzing response to ground shaking. The process of cracking and the propagation of the cracks result in nonlinear behavior of the dam. There are also nonlinearities associated with dam-foundation interaction and dam-reservoir interaction which are difficult to assess. Approximate linear relationships account for some of the nonlinear dynamic behavior and allow the response of the dam to the design earthquake ground motion to be determined using a linear-elastic analysis method. Tensile stresses can then be evaluated based on tensile strength parameters adjusted to be compatible with linear-elastic analysis methods.

d. Analysis methods. The simplest of the linear-elastic methods uses a response spectrum to define the ground motion as outlined in Chapter 5. Most RCC dams will be found adequate using this method. For the few exceptions, the next level of refinement in determining the dynamic response is the linear-elastic time history method, and in rare cases a nonlinear time history finite element analysis may be required.

e. Allowable tensile stress. The tensile strength of the RCC is the single concrete material property used to evaluate cracking, and to establish acceptable response. Allowable tensile stresses are defined in paragraph 4-2*c* and paragraph 4-3*c* for the OBE and MCE, respectively.

f. Evaluating time-history response. When dynamic response is determined by the linear-elastic time-history method, the allowable tensile stress is the principal criterion for evaluating acceptable response, but additional criteria are also required to qualify other response characteristics such as the number of stress cycles approaching or exceeding the allowable stress, and the magnitude and pattern of these excursions beyond the specified limits.

g. Evaluating nonlinear analyses. When dynamic response is determined by the nonlinear time-history method, criteria for evaluating acceptable response are based on the theory of fracture mechanics. This type of analysis should only be undertaken in consultation with and as approved by CECW-ED.

2-3. Foundation Fault Displacement

a. General. Most RCC dam sites are not subject to any significant differential displacement of the ground surface at the dam-foundation interface during a seismic event. Dam sites should always be avoided when located near a major active fault system with the potential to trigger sympathetic foundation displacements at the site. Occasionally it is not possible to avoid these sites, and it becomes necessary to evaluate the response of the dam should such a foundation fault displacement occur.

(1) Considerable judgment is required in the evaluation process. At best, analysis methods for foundation fault displacement are approximate and are generally unsupported by past observations of the response of existing dams to fault displacements occurring at the dam foundation. Furthermore, considerable judgment is required in the prediction of future fault movement and in the magnitude of the fault displacement. For example, the estimate of the magnitude of potential fault displacement provided by different experts for a specific site could vary from a few inches to several feet. This necessitates consulting several geotechnical firms to provide site-specific fault displacement estimates, and then carefully scrutinizing these estimates before finally establishing the design fault displacement.

(2) Experts in plate tectonics, geology, seismology, and finite element analysis techniques should be consulted to provide guidance for any dam located on a site subject to foundation fault displacement. Because of the many uncertainties and the risk involved, approval by CECW-ED is required for any RCC dam which is located on a site subject to foundation fault displacement.

b. Types of faults. Fault slip is the relative displacement of two adjacent tectonic plates with respect to each other. This refers to large active fault systems such as the San Andreas or Hayward faults in California. On a smaller scale, the foundation rock mass beneath a dam contains various discontinuities, joint sets, and shear and fault zones. Normally this is a system of historically inactive discontinuities; however, there is a potential for fault slippage particularly when triggered by a great earthquake on a nearby large active fault. The three general types of fault slips are strike-slip, normal-slip (dip-slip), and reverse-slip (thrust-slip). Refer to Figure 2-1 for illustrations of the various types of faults and how the magnitude of slip is measured. The strike of the fault is the trace the fault makes with respect to the ground surface, and it may be at any orientation with respect to the dam axis.

c. Design fault displacement. The design fault displacement (DFD) is defined as the maximum possible free field fault slip movement that could reasonably occur in the dam foundation as measured at the ground surface. The return period that would be associated with the DFD is similar to that of the MCE. Therefore, the DFD and the free field ground motion together specify the site-specific seismic activity associated with the MCE. To fully describe the DFD, three factors must be specified: magnitude, type of slip, and strike of the fault.

(1) The geology of the dam foundation is complex, and the foundation may be crossed by a number of discontinuities with fault displacement potential. Experts in the fields of geology and seismology should be consulted to study the foundation fault system, determine which faults are capable of surface displacement, and finally recommend which faults are critical and specify the DFD for each critical fault.

(2) Normally, foundation fault displacements are not considered to occur concurrently with strong motion shaking associated with the OBE. The active fault near the dam site that produces a seismic event of OBE magnitude is not likely to trigger sympathetic slippage in the fault system in the dam foundation. The probability of sympathetic foundation fault displacement is normally several orders of magnitude less than the recurrence rate for the strong motion shaking associated with the OBE; therefore, the probability of the OBE being accompanied by significant foundation displacement is usually considered negligible.

(3) On rare occasions, the probability logic discussed above may not apply when considering if it is appropriate to combine foundation fault displacement with ground shaking in specifying the OBE. For example, unusual geology of the foundation could make it susceptible to a reservoir-induced foundation fault displacement or to other

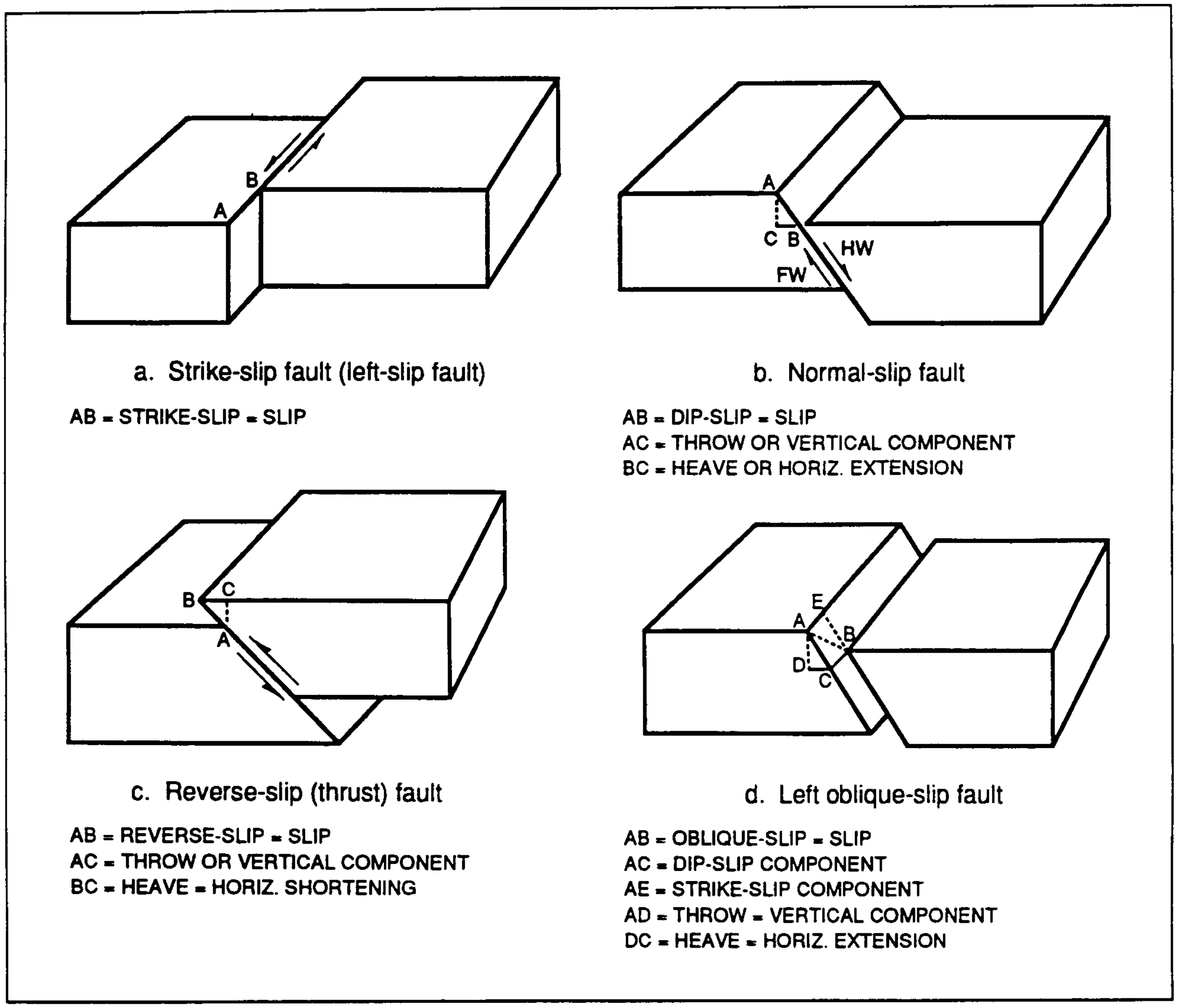


Figure 2-1. Types of fault slips

unusual causes of foundation fault displacement discussed later in this chapter. In these situations the strong motion shaking accompanying the local fault slip may be nearly as intense or even more intense than the gound motion shaking associated with an OBE produced by a major active fault slip occurring some distance from the site. When this is the case, a reduced value of the DFD would be included with free field ground motion to describe the OBE.

d. Combined DFD and ground shaking. Stresses associated with the DFD result from highly complex nonlinear behavior; however, simplified fault displacement analysis procedures, such as the one described below, are normally used to investigate concrete stresses that may occur due to fault displacement. Stresses due to ground shaking are determined by methods discussed earlier in this chapter. Thus, stresses due to fault displacement and stresses due to ground shaking are obtained from two separate, independent, and approximate analyses. The response to the design earthquake is then obtained by direct addition of the two sets of stresses without accounting for any interaction. Actually, the fault displacement may cause inelastic behavior at the dam-foundation interface, cracking within the RCC, or other inelastic response which changes the dynamic characteristics of the dam, which in turn interacts with and effects the ground shaking response. Because these simplified and approximate procedures have not been supported by nonlinear finite element analyses that properly combine the effects of fault displacement and ground shaking, they should be used with caution.

e. Simplified DFD analysis procedure. The simplified procedure described below was used to investigate concrete stresses due to fault displacement in the Auburn Dam in California (U.S. Department of the Interior, Bureau of Reclamation 1980). The dam and foundation are modeled with finite elements with the mesh geometry adjusted to allow the fault to be properly oriented. Refer to Figure 2-2. The foundation model consists of a fixed block with conventional boundary supports, and a movable block with special boundary conditions that allow forces to be applied at the boundary parallel to the fault to produce the DFD. The fixed and movable block are separated by elastic orthotropic elements which allow the sharp displacement discontinuity to take place as the movable block displaces upward.

(1) The finite element model is first loaded with the gravity loads followed by the hydrostatic loads, and finally the movable block is forced to undergo the DFD. Each loading is applied incrementally. After each loading increment, tensile stresses are evaluated and elements are softened in areas where the tensile strength is exceeded. Elements are softened by reducing their elastic modulus until the tensile stress is eliminated. Most elements requiring softening are located in the foundation because jointing and discontinuities in the rock prevent it from sustaining high tensile stress. When the DFD is reached, the extent of the tensile failure areas is evaluated. The dam tends to bridge over the fracture zone in the foundation. Resulting stresses induced in the RCC are obtained from the finite element analysis for the final increment of loading which produced the DFD.

(2) The method of incremental loading and softening of element properties allows the use of a simplified static, linear-elastic finite element analysis approach. Disadvantages of the procedure are that it gives only an approximation of the complex nonlinear behavior associated with fault displacement, it is time consuming, and it requires considerable judgment.

(3) The example shown in Figure 2-2 is typical for a normal or reverse fault where the fault strike is approximately parallel to the dam axis so a two-dimensional analysis is adequate. If the fault strike is not close to parallel to the dam axis, or for a strike-slip fault, a three-dimensional analysis is required. The three-dimensional analysis is even more time consuming and complex, but the principles and general procedure are similar to the two-dimensional analysis described.

f. Acceptable response to DFD. When the seismic activity associated with the design earthquake consists of both fault displacement and ground shaking, stresses for the combined response described in paragraph 2-3*d* must satisfy the allowable tensile stress criteria of paragraph 2-2*e*. Beyond these tensile stress requirements, additional consideration is required regarding general performance requirements of Chapter 4 related to dam safety and operations in the event of foundation fault displacement. The potential fault displacement and the effect it has on the dam must be evaluated on a case-by-case basis. The analysis procedures described above for evaluating the effect of fault displacement are rough approximations, but they do provide an indication of the extent of the fracture zones that could occur in the foundation or lower portions of the RCC dam. The analysis results must be coupled with considerable judgment to determine if this damage could lead to the erosion of the foundation or RCC materials to the extent that finally causes an uncontrolled release of the reservoir.

g. Dam failures caused by fault displacements. To help identify some of the judgment factors involved in evaluating sites with fault displacement potential, the following is a brief review of historical information on dams that failed directly or indirectly as a result of fault displacement. Differential displacements across a fault have been recorded due to: triggering of the fault by a seismic event; a difference in consolidation of materials on either side of the fault; a reduction in resistance to fault movement created by the lubricating effects of water, or the erosion of fault materials by flowing water; and increase in hydrostatic pressures along the fault.

(1) Earth-fill dams, concrete gravity dams, and concrete arch dams have failed due to fault movements. Failures of the Baldwin Hills earth-fill dam, the Malpasset concrete arch dam, and the St. Francis concrete gravity dam (James et al. 1988) can all be attributed in part to forces and movements occurring along fault surfaces. Although these forces and movements were not triggered by seismic activity, it can be surmised that if a seismic event had occurred, it would have likely triggered similar failures. These examples show that fault movement can cause a

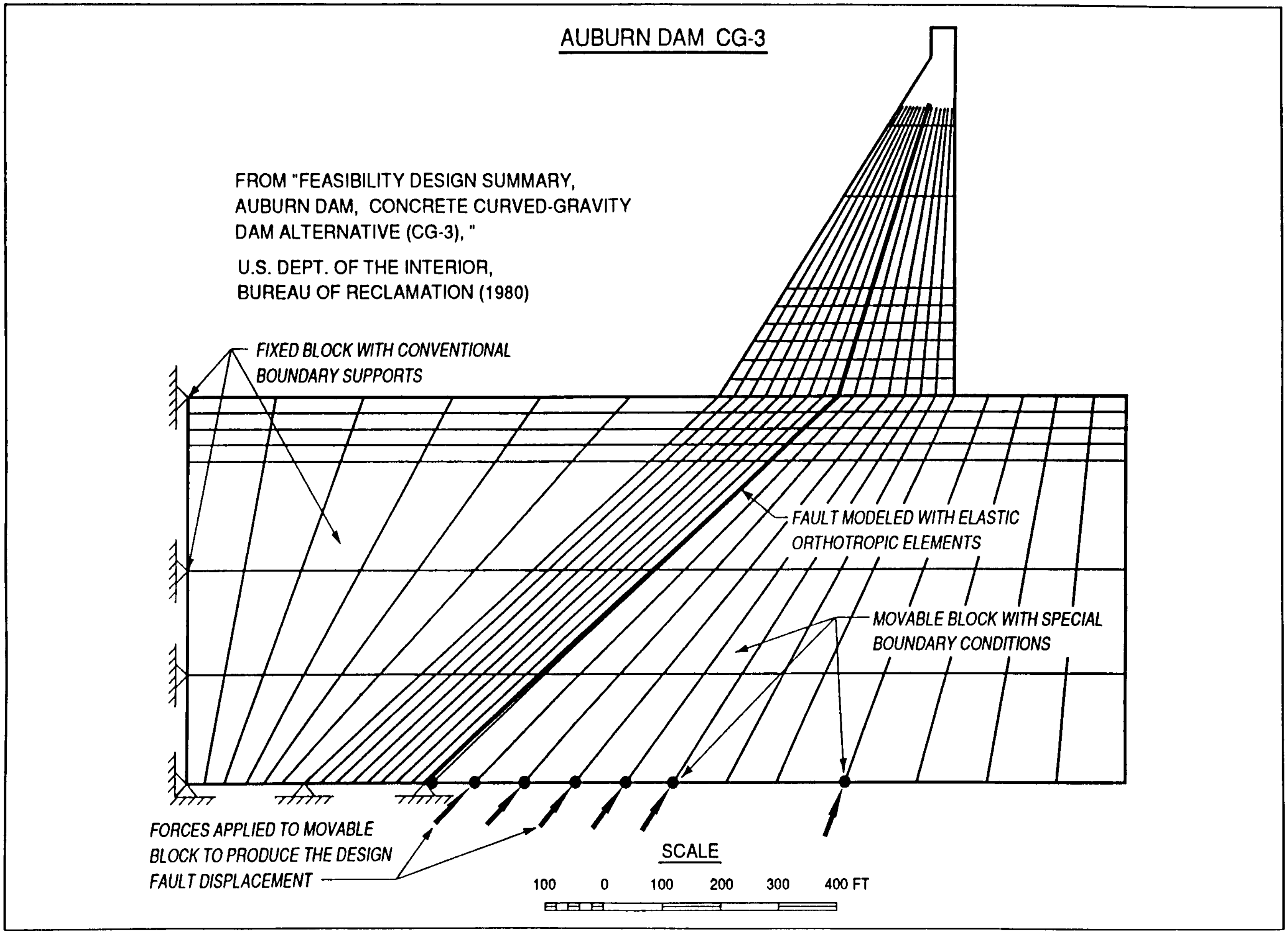


Figure 2-2. Finite element model for fault displacement analysis

failure mechanism to form in the dam structure which results in dam failure; however, it is more likely that the fault movement would create flow paths that could lead to a release of the impounded reservoir. Seepage can erode dam or foundation materials which eventually results in failure because capability for controlled release of the pool is lost.

(2) An earth-fill dam with a flexible core is normally considered less susceptible to failure due to foundation fault displacement because it would tend to conform to the displaced shape of the foundation. Although this flexibility of the dam material will reduce voids and flow paths in the dam and foundation it will not completely eliminate them. Thus, an earth-fill dam is susceptible to erosion of core or foundation material from water flowing through faults or through voids in the dam or foundation created by fault movements. For this reason, an earth-fill dam is not necessarily superior to a concrete gravity dam in resisting the effects of fault movement.

h. Defensive design features. Defensive design features which can be employed in the design of an RCC dam susceptible to foundation displacement are discussed below.

(1) The arching action provided by laying out the dam axis on a curve may better distribute the forces on a gravity dam due to foundation fault displacement, and reduce the tensile stresses and cracking of the RCC. This defensive feature is only effective if the heave of the foundation block is generally in a downstream direction, and providing the fault movement does not occur at either abutment.

(2) Special sliding joints may also be used to reduce cracking of the RCC due to fault displacement. For example, vertical joints may be located in the RCC to accommodate potential strike-slip fault displacements where the strike is generally in the upstream-downstream direction.

(3) A design feature for controlling the reservoir release is to provide a buttress fill against the upstream face of the dam. This requires the reservoir water to pass through a succession of filters and crack stoppers in a manner analogous to the behavior of the transitions and filters in a zoned embankment dam. This defensive measure would be effective for flood-control projects where the reservoir pool elevation is low enough that the required height of the buttress fill is economically feasible, and does not impair the stability of the dam.

2-4. Refined Dynamic Analyses Methods

a. Need for refinement. When the simplified linear-elastic analysis methods described above for an existing RCC dam produce tensile stresses in excess of the allowables discussed in paragraph 2-2*e*, more refined analyses methods shall be pursued before the dam is judged unsafe. Also, if all practical and economical adjustments to the design of a new dam have been exhausted in the attempt to satisfy the allowables based on simplified linear-elastic methods, the more refined analyses methods may be pursued to better evaluate nonlinear structural behavior. Refined analyses consist of linear or nonlinear time history analyses as discussed in paragraph 2-2*d*, with some additional details of the nonlinear analysis provided below. The response produced by refined analyses shall be evaluated in accordance with the stipulations of paragraphs 2-2*f* and 2-2*g*.

b. Fracture mechanics. Nonlinear dynamic analysis is based on fracture mechanics theory which is presently in the research phase. It is also difficult to determine just what level of structural damage can be sustained safely by the dam and still consider it to satisfy the performance requirements. The nonlinear attribute requires this type of dynamic analysis be performed in a time domain (time history analysis) rather than a frequency domain (response spectrum analysis), and use a direct integration solution. The analysis accounts for: energy dissipation by cracking, strength of cracked concrete, changes in vibration characteristics caused by cracking, changes in damping, and changes in strength due to strain rate and loading history.

c. Nonlinear analysis requirements. Because it is very complex, costly, and requires a considerable amount of judgment to interpret the results, an expert in fracture mechanics and nonlinear analysis techniques should be consulted to provide guidance when pursuing a nonlinear analysis.

Chapter 3
Material Properties of RCC

3-1. Similarities of RCC and Conventional Concrete

The strength and elastic properties of RCC vary depending on the mix components and mix proportions in much the same manner as that for conventional mass concrete. Aggregate quality and water-cement ratio are the principal factors affecting strength and elastic properties. Properties important to the seismic analysis of RCC dams include compressive strength, tensile strength, shear strength, modulus of elasticity, Poisson's ratio, and unit weight. Except for unit weight, all these properties are strain rate sensitive, and the strain rates that occur during major earthquakes are in the order of 1,000 times greater than those used in standard laboratory testing. Guidance concerning the determination of RCC material properties is given in EM 1110-2-2006 and ETL 1110-2-343.

3-2. Compressive Strength

The relationship between water-cement ratio and compressive strength is the same for RCC as for conventional mass concrete. Normally, for durability reasons, the RCC mix will be designed to provide a minimum strength of 2,000 psi; however, for seismic reasons higher compressive strengths are often required to achieve the desired tensile and shear strength. The compressive strength at seismic strain rates will be 15 to 20 percent greater than that at the quasi-static rates used during laboratory testing (ACI Committee-439 1969); however, compressive strength is never the governing factor in seismic design.

3-3. Tensile Strength

The tensile strength of RCC shall be based on the direct tensile strength tests of core samples. For the final design of new dams, cores shall be taken from test-fill placements made with the proposed design mixes, and placed with the proposed consolidation and joint treatment methods. When an existing dam is evaluated for compliance with the requirements of this EP, cores shall be taken directly from the structure. Cores should be taken vertically so that tests can be made which reflect weaknesses inherent at lift joint surfaces in addition to the tests to determine the tensile strength of the parent concrete.

a. Location of critical tensile stress. Critical tensile stresses are located at the upstream and downstream faces of the dam. The tensile stress distribution within the dam mass is of interest to help establish zone boundaries for superior, higher strength RCC mixes that may be required to control cracking near the faces.

(1) Usually the tensile stress in the lift joints in the direction normal to the joint surface is critical near the upstream face of the dam. This is because the direction of the principal tensile stress near the upstream face is very nearly normal to the joint surface, thus there is little difference between the joint stress and the maximum principal stress in the parent concrete. Since tensile strength of the lift joint is notably less than the parent RCC, it will control the design near the upstream face.

(2) Near the downstream face, the direction of the principal tensile stress is nearly parallel to the face which results in significantly higher principal tensile stresses in the parent concrete compared to the tensile stresses in the lift joints normal to the joint surface. The ratio of the tensile strength of parent concrete to the tensile strength of the lift joints varies according to several parameters including workability of the mix, joint preparation, and maximum size aggregate. Thus, it usually becomes necessary to investigate both the principal tensile stress and the component tensile stress normal to the lift joints to determine which is critical near the downstream face.

b. Preliminary design. For preliminary design, the tensile strength of the RCC may be obtained from Figures 3-1 through 3-6 for the proposed concrete compressive strength (f'_c). These figures show both the tensile strength of the parent material and the tensile strength of the lift joint based on the proposed consolidation and joint treatment method. These figures were developed from Tables E2 and E3, Appendix E.

c. Tensile strength tests. Splitting tensile tests are easier to perform and provide more consistent results than direct tensile tests. However, splitting tensile test results tends to overpredict actual tensile strengths, and should be adjusted by a strength reduction factor to reflect results that would be obtained from direct tensile tests. When splitting tensile tests

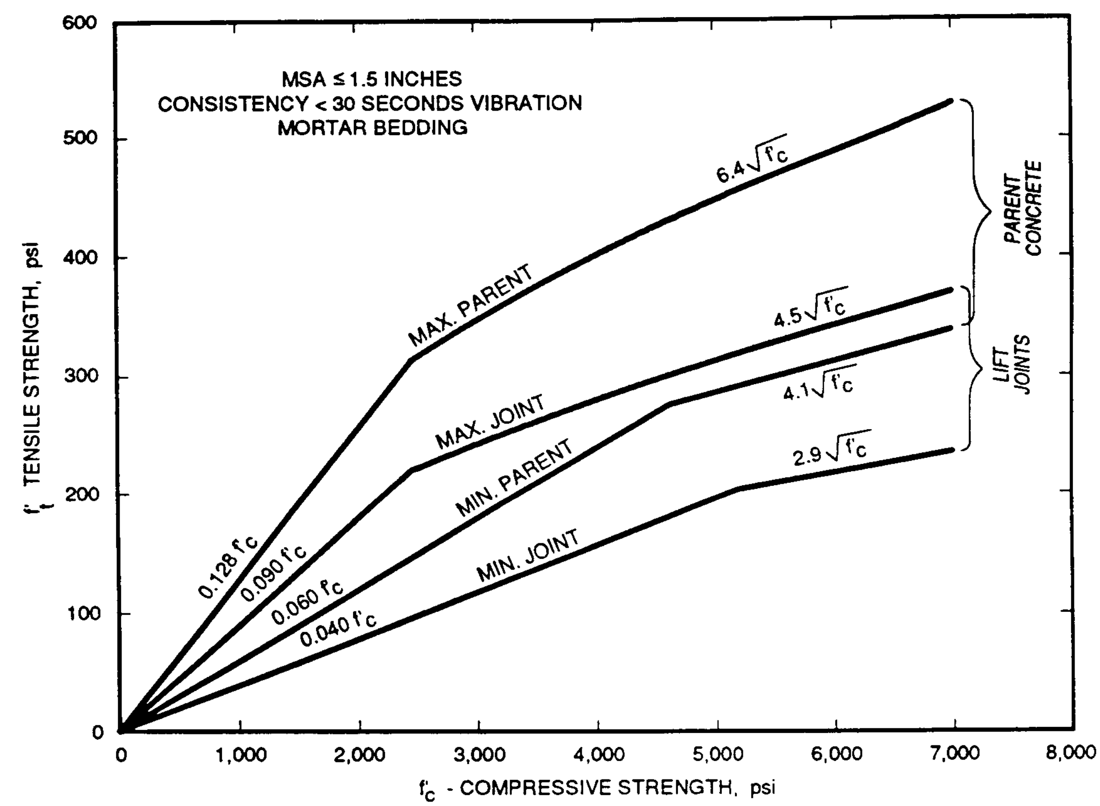


Figure 3-1. Tensile strength range, RCC, MSA ≤ 1.5 inches, consistency < 30 seconds vibration, mortar bedding

are used as the basis for determining the tensile strength of RCC, the test results shall be reduced by a strength reduction factor of 75 percent as recommended in Appendix E.

d. Factors affecting tensile strength. The tensile strength of RCC, as well as of conventionally placed mass concrete, is dependent on many variables including paste and aggregate strength, aggregate size, loading history, and load deformation rates. See paragraph 3-9 concerning strain rate sensitivity and dynamic tensile strength.

(1) RCC differs from conventionally placed mass concrete due to the many horizontal planes of weakness (construction joints) created during placement. RCC is placed and compacted in layers ranging from 6 to 24 inches with each layer creating a joint with tensile strength less than that of the parent concrete. The joint strength can be improved by placing a layer of high slump bedding mortar on each lift; however, the resulting joint strength is always somewhat less than the parent concrete. The consistency of RCC can also affect tensile strength with lower strength values for harsh mixes with low paste contents. Refer to Chapter 2 for additional discussion of these factors.

(2) Inherent in some RCC mixes are certain anisotropic material properties. In the RCC compaction process, the flatter coarse aggregate particles in these mixes have a tendency to align themselves in the horizontal direction. When this occurs, the strength of vertical cores will be less, and the strength of horizontal cores greater than the average tensile strength. The variance from average could be as high as 20 percent, although in general these effects will

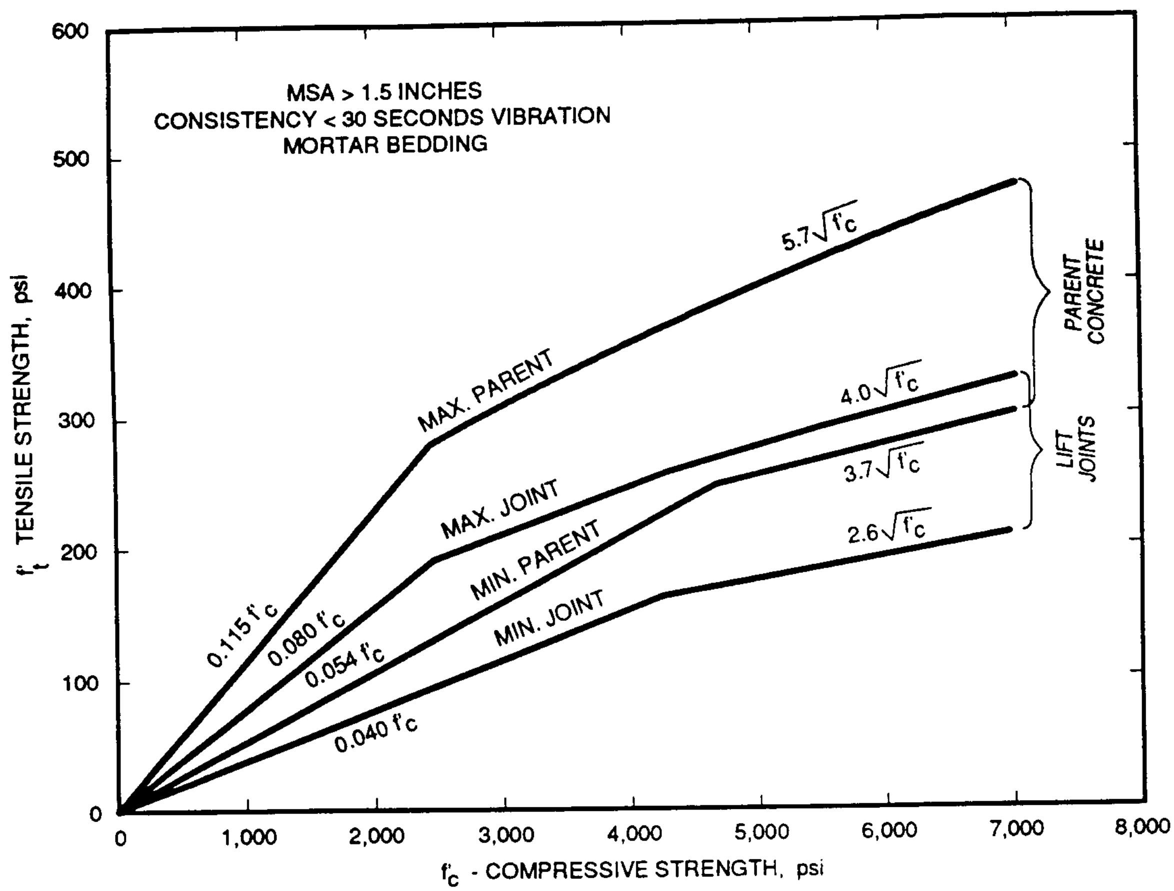


Figure 3-2. Tensile strength range, RCC, MSA > 1.5 inches, consistency < 30 seconds vibration, mortar bedding

be small. If the coarse aggregate particle shape indicates the possibility of significant anisotropy, both vertical and horizontal cores obtained from the laboratory test placement should be tested.

3-4. Shear Strength

The shear strength along lift joint surfaces is always less than the parent concrete; therefore, final shear strength determination should be based on tests of representative samples from the dam or test fill. Both the bond strength and the tangent of the angle of internal friction can be increased by 10 percent to account for the apparent higher strengths associated with seismic strain rates.

3-5. Modulus of Elasticity

RCC will usually provide a modulus of elasticity equal to, or greater than, that of conventional mass concrete of equal compressive strength. The modulus of RCC in tension is equal to that in compression. The static modulus of elasticity, in the absence of testing, can be assumed equal to (ACI Committee-207 1973):

$$E = 57{,}000\sqrt{f_c'}$$

where E = static modulus of elasticity

f_c' = static compressive strength of RCC

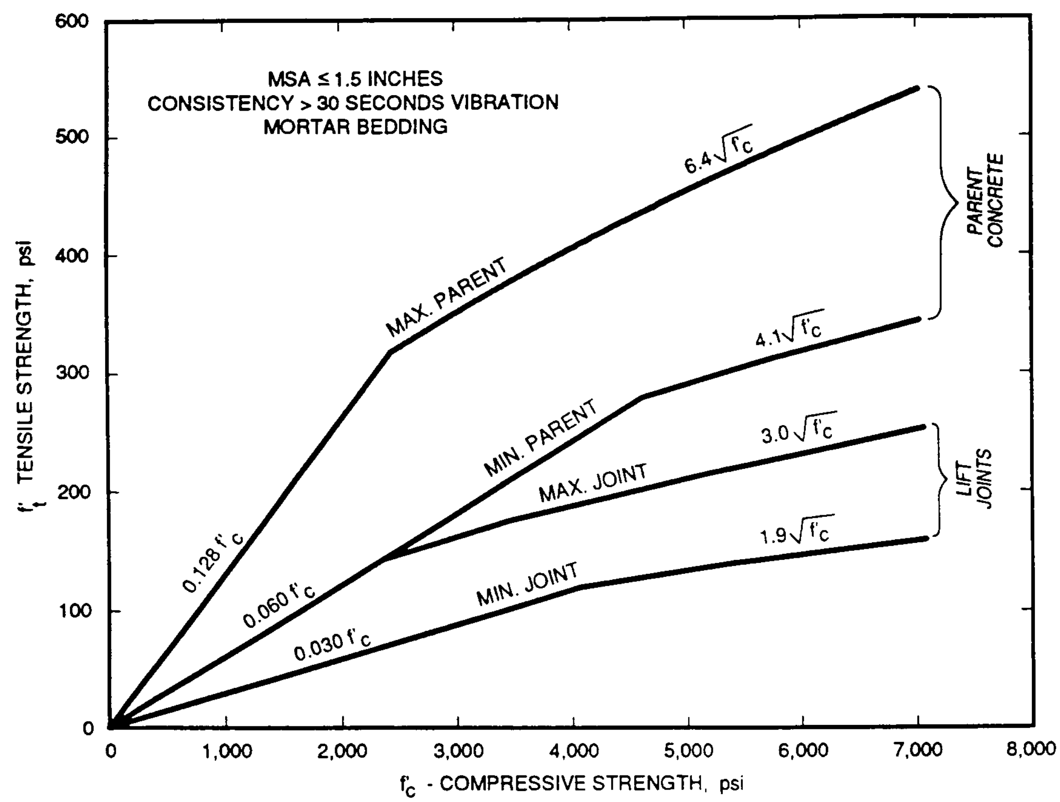


Figure 3-3. Tensile strength range, RCC, MSA ≤ 1.5 inches, consistency > 30 seconds vibration, mortar bedding

The relationship between strain rate and modulus of elasticity is as follows (Bruhwieler 1990):

$$E' = E(E_r)^{0.020}$$

where E = static modulus of elasticity

E' = seismic modulus of elasticity at the quasi-static rate

$$E_r = \frac{\text{high seismic strain rate}}{\text{quasi-static rate}}$$

For a seismic strain rate equal to 1,000 times the quasi-static rate the seismic modulus of elasticity is 1.15 times the static modulus. For long-term loadings where creep effects are important, the effective modulus of elasticity may be only 2/3 the static modulus of elasticity calculated by the above formula (Dunstan 1978). The modulus of elasticity may exhibit some anisotropic behavior due to the coarse aggregate particle alignment as discussed in paragraph 3-3*d*(2); however, the effects on the modulus will be small and can be disregarded when performing a dynamic stress analysis.

3-6. Poisson's Ratio

Poisson's ratio for RCC is the same as for conventional mass concrete. For static loads, values range between 0.17 and 0.22, with 0.20 recommended when testing has not been performed. Poisson's ratio is also strain rate sensitive, and the static value should be reduced by 30 percent when evaluating stresses due to seismic loads (Bruhwieler 1990).

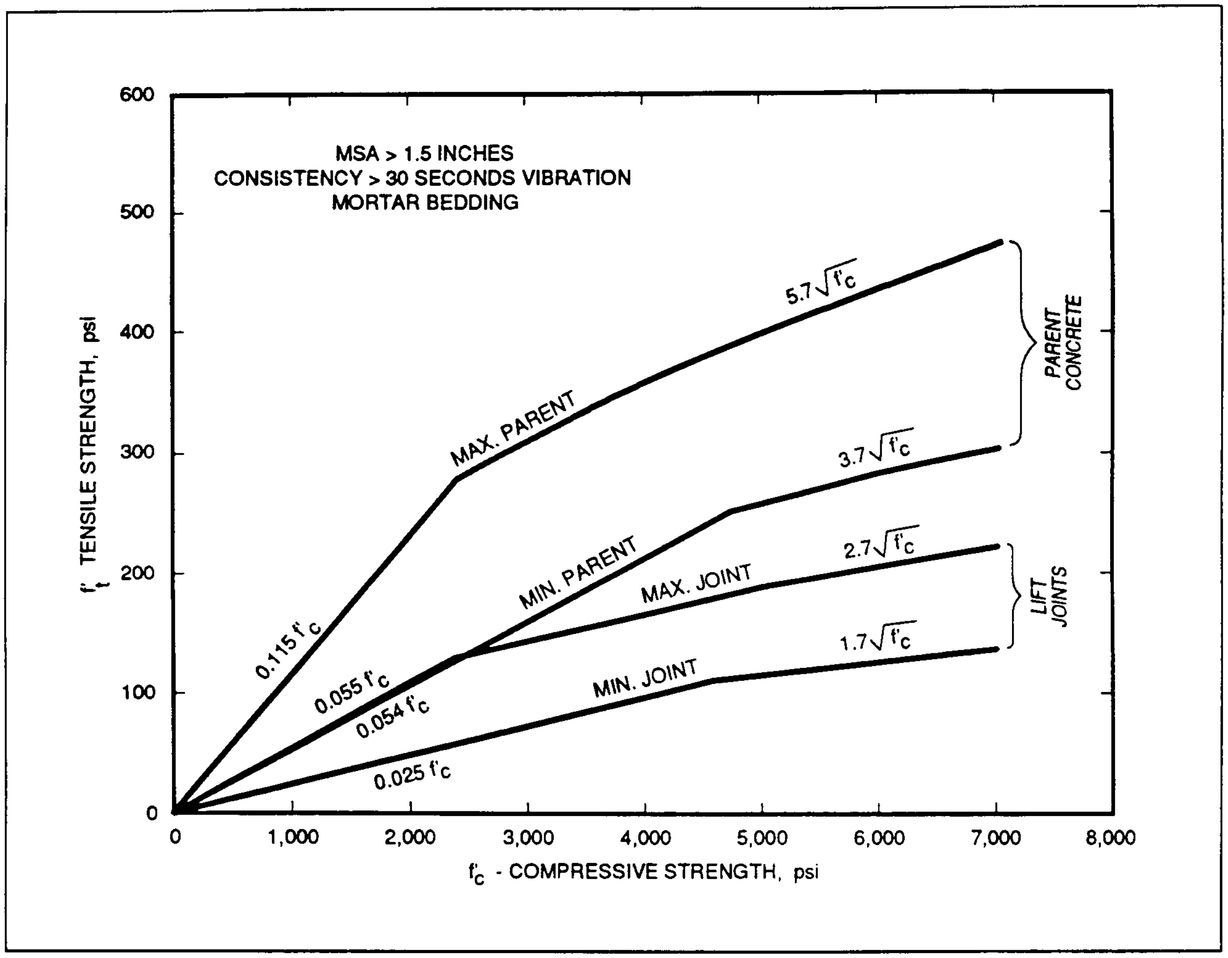


Figure 3-4. Tensile strength range, RCC, MSA > 1.5 inches, consistency > 30 seconds vibration, mortar bedding

3-7. Tensile Stress/Strain Relationship

As mentioned in paragraph 2-2*b*, concrete cracking, crack propagation, and the energy dissipated in the process are complex and nonlinear in nature. For a simplified linear-elastic analysis, a constant modulus of elasticity is required. Thus, a linear stress/strain relationship is used for the analysis with a tensile modulus equal to the modulus of elasticity for concrete in compression.

a. Compression and tension differences. Although a linear relationship is assumed for the analysis, in actuality the stress/strain relationship becomes nonlinear after concrete stresses reach approximately 60 percent of the peak stress (Raphael 1984). In compression this does not cause a problem because, in general, concrete compressive stresses even during a major earthquake are quite low with respect to the peak stress or ultimate capacity. In tension, it is a different matter since tensile stress can approach and exceed the peak tensile stress capacity of the concrete and in some cases cracking will occur.

b. Tensile stress/strain curve. The actual nonlinear stress/strain relationship for RCC concrete is shown in Figure 3-7. The assumed linear relationship used for finite element analysis was developed from the work done by Raphael (1984). The actual nonlinear performance of concrete in tension consists of a linear region from zero stress up to 60 percent of the peak stress, a nonlinear ascending region from 60 percent of peak stress to peak stress (this point on the curve corresponds to the direct tensile strength test value described in paragraph 3-3*c*), and a nonlinear descending region from peak stress back to zero

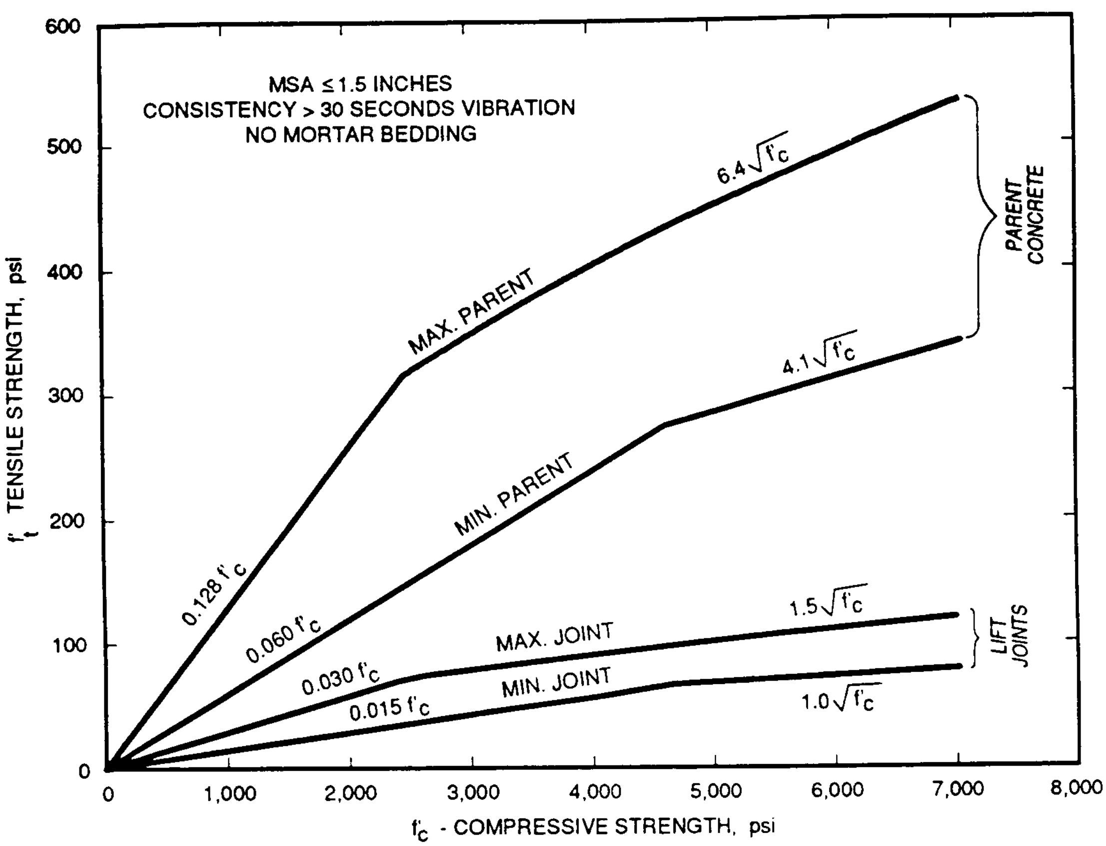


Figure 3-5. Tensile strength range, RCC, MSA ≤ 1.5 inches, consistency > 30 seconds vibration, no mortar bedding

stress. The last region is termed the "tensile softening zone." In this region, where deformation increases with decreasing stress, deformation controlled stable test procedures are required to capture the stress/strain behavior (Bruhwieler 1990), where conventional test procedures will cause the strain to fall off abruptly to zero strain at a point on the curve just beyond the peak stress point. The area under the tensile softening region of the stress/strain curve represents additional energy absorbed by the RCC structure during the crack formation process. As such, this region is quite instrumental in dissipating the energy imparted to the dam through seismic ground motion. The transition from linear to nonlinear in the ascending region of the stress/strain curve represents the development of microcracking within the concrete. These microcracks eventually coalesce into macrocracks as the tensile softening zone is reached.

3-8. Dynamic Tensile Strength (DTS)

The tensile strength of concrete is strain rate sensitive. During seismic events strain rates are related to the fundamental period of vibration of the dam with the peak stress reached during a quarter cycle of vibration. The high strain rates associated with dam response to ground motion produce tensile strengths 50 to 80 percent higher than those produced during direct tensile strength testing where the strain rate is very slow. For this reason, the dynamic tensile strength (DTS) of RCC shall be equivalent to the direct tensile strength multiplied by a factor of 1.50 (Cannon 1991, Raphael 1984). This adjustment factor applies to both the tensile strength of the parent material and to the tensile strength at the lift joints.

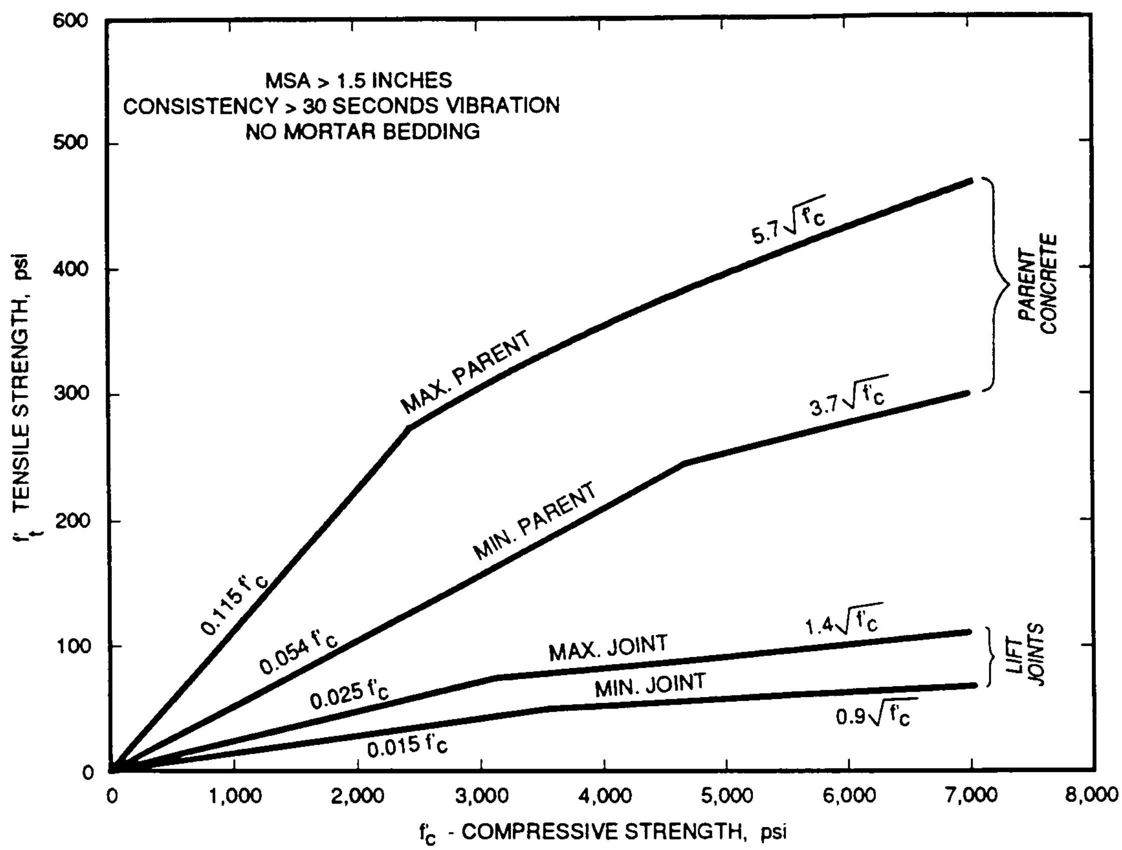


Figure 3-6. Tensile strength range, RCC, MSA > 1.5 inches, consistency > 30 seconds vibration, no mortar bedding

3-9. Allowable Tensile Stresses

When the response to ground motion increases beyond the elastic limit, energy is dissipated through crack development and crack propagation in accordance with the stress/strain relationship shown in Figure 3-7. To account for all nonlinear response including that in the tensile softening zone of the stress/strain curve requires a complex nonlinear analysis. The simpler linear-elastic analysis may be utilized in a manner which accounts for response in the linear region, and the nonlinear pre-peak region.

a. Comparing linear and nonlinear curves. Since a linear-elastic analysis converts strains to stress using a constant modulus of elasticity, the stresses from the analysis will be higher than actual stresses when in the nonlinear pre-peak and post-peak strain regions. This may be compensated for by establishing an allowable tensile stress which is greater than the actual peak tensile stress as shown in Figure 3-7. In this figure, the dashed line represents the tensile stress/strain relationship assuming linear-elastic behavior as opposed to the actual nonlinear stress/strain relationship which is shown as a heavy solid line. The amount the peak tensile stress is increased in establishing the allowable stress depends on the extent of tensile cracking that can be tolerated, which in turn is based on the performance requirements for the design earthquake under consideration. The economics of the design also becomes a factor in the higher seismic zones. In these zones, a somewhat greater amount of cracking can be justified economically because there is a point where the cost of producing RCC mixes with high tensile strengths to resist cracking will exceed the cost of repairing the cracks as long as the cracking is not too extensive.

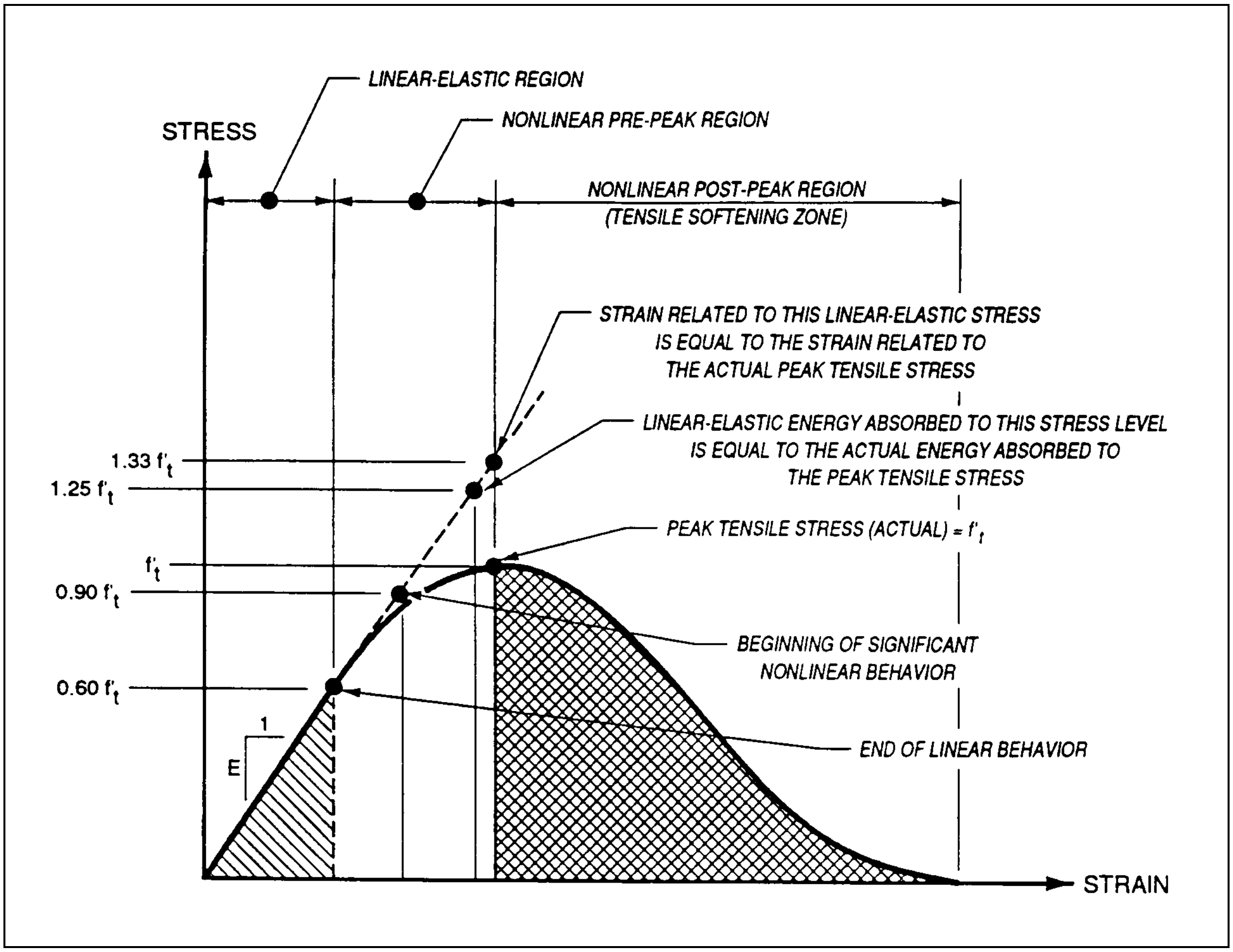


Figure 3-7. Tensile stress/strain diagram for RCC

b. Key points on stress/strain curve. Several points on the stress/strain curve are of interest when establishing the allowable tensile stresses that are used in linear-elastic analyses (refer to paragraphs 4-2*c* and 4-3*c*). Based on f'_t = actual peak tensile stress (tensile stress that corresponds to that which would be attained by a direct tensile strength test), and f_t = the stress level based on linear-elastic behavior (refer to the dashed line in Figure 3-7), the following key values of f_t are of interest:

(1) $f_t = 0.60\, f'_t$ -- the end of the elastic range and the beginning of microcracking.

(2) $f_t = 0.90\, f'_t$ -- this point was selected because the stress/strain dashed line for linear-elastic behavior is just beginning to significantly separate from the actual stress/strain curve. If the tensile stresses for a linear-elastic analysis stay within the stress level for this point, the response can still be judged as primarily linear.

(3) $f_t = 1.25\, f'_t$ -- the area under the dashed line for linear-elastic behavior up to this stress level is approximately equal to the area under the solid line for the actual stress/strain curve up to the peak tensile stress point (this point is the end of microcracking and the beginning of macrocracking). Thus, the energy absorbed in a linear-elastic analysis to this point of stress is equal to the actual energy absorbed through the microcracking pre-peak region.

(4) $f_t = 1.33\, f'_t$ -- the strain corresponding to this point of stress based on linear-elastic behavior is equal to the strain corresponding to the actual peak tensile stress. This strain point signifies the end of microcracking and the beginning of macrocracking. This point also represents a practical limit for the

linear-elastic response spectrum analysis described in paragraph 2-2*c*. Beyond this point in the tensile softening zone, the stress/strain relationship based on linear-elastic behavior diverges so rapidly from the actual stress/strain curve that a linear-elastic analysis will no longer provide an acceptable approximation of either the energy absorbed by the dam-foundation system, or the strain deformation of the system. Cracking could be extensive enough to change the dynamic properties of the dam structure.

Chapter 4
Design Earthquakes

4-1. Definition

The term “design earthquake” refers to the specification of the free field ground motion that would be felt at the dam site due to a particular seismic event that is used as the basis for earthquake resistant design of new RCC dams, or to evaluate the response of existing RCC dams.

4-2. Operating Basis Earthquake (OBE)

The OBE is defined as the earthquake producing the greatest level of ground motion that is likely to occur at the site during the service life of the dam. The service life shall be taken as 100 years for both new dams and existing dams. The seismic risk or adverse consequences of failure of an existing dam is not reduced as long as the dam is in operation; therefore, the “remaining service life” of an existing dam shall not be substituted for the 100-year service life specified above. The OBE is determined using probabilistic methods and, as such, is defined as the earthquake with a 50 percent chance of exceedance in the service life of the dam.

a. General performance requirements. All structural, mechanical, and control equipment used to regulate the reservoir shall be capable of remaining fully operational during and after an OBE. New RCC dams located in low seismic regions shall be designed to prevent the initiation of cracking in the concrete structure. Tensile cracking in new RCC dams located in high seismic regions and in existing dams in all seismic regions is allowed; however, it shall be limited to only “minor cracking” that requires little or no repair.

b. Structural criteria. The following general structural criteria shall be the basis for satisfying the concrete cracking performance requirements stated above.

(1) Initiation of cracking is prevented when the tensile stresses are less than 0.60 f'_t as shown in Figure 3-7.

(2) The level of cracking is considered to be “minor cracking” when the tensile stresses are less than 1.25 f'_t as shown in Figure 3-7.

c. Allowable tensile stress. The allowable tensile stresses $f_{t\ (allowable)}$ for the OBE are established below. The formulae apply to the calculation of both allowable tensile stress of the parent material and allowable tensile stress of the lift joints. DTS = Dynamic Tensile Strength, and f'_t = direct tensile strength.

(1) Existing dams:

$$f_{t\ (allowable)} = 1.25 \times \text{DTS} = 1.875 \times f'_t$$

(2) New dams in seismic zones 0, 1, 2A, and 2B:

$$f_{t\ (allowable)} = 0.60 \times \text{DTS} = 0.90 \times f'_t$$

(3) New dams in seismic zones 3 and 4:

$$f_{t\ (allowable)} = 0.90 \times \text{DTS} = 1.35 \times f'_t$$

d. Damping. Studies on dams under severe ground motion which cause stresses in the upper reaches of the elastic range indicate a dampened response which corresponds to a damping factor of about 5 percent of critical. On this basis the OBE shall be analyzed using a damping ratio equal to 5.0 percent of critical damping for the concrete dam structure only. This factor must be modified as outlined in paragraph 7-3 to account for foundation damping.

4-3. Maximum Credible Earthquake (MCE)

The MCE is defined as the largest possible earthquake that could reasonably occur along the recognized faults or within a particular seismic source. Often several fault sources must be investigated to determine which will produce the critical site ground motion. By definition the MCE has a very low probability of occurence. Ground motion associated with the MCE is established using the deterministic approach.

a. General performance requirements. Both new RCC dams and existing dams shall be capable of surviving the MCE without a failure of a type that would result in the loss of life or significant damage to downstream property caused by an uncontrolled release of the reservoir pool. Nonlinear behavior with associated damage is permissible, but the post earthquake damaged condition of the dam shall allow for controlled lowering of the pool to facilitate repair.

b. Structural criteria. The upper limit of linear elastic analysis is considered to be that point on the straight stress/strain line corresponding to a linear stress level of 1.33 f'_t (see Figure 4-7). When tensile strains exceed the strain associated with this linear stress limit, macrocracking occurs and the RCC will be subject to some degree of structural damage. As the strain level increases well into the tensile softening zone, response becomes markedly nonlinear and it is clear that a linear-elastic analysis no longer approximates the response. Although crack damage increases in this zone, performance requirements may still be satisfied. Thus, the structural criteria for the MCE, when using linear-elastic analysis, are set by limitations of the method of analysis rather than on criteria that relate to an acceptable level of structural concrete damage.

c. Allowable tensile stresses. The allowable tensile stress $f_{t\,(allowable)}$ for the MCE is established below. DTS = Dynamic Tensile Strength, and f'_t = the direct tensile strength.

$$f_{t\,(allowable)} = 1.33 \times DTS = 2.000 \times f'_t$$

d. Damping. The linear-elastic analysis for the MCE shall utilize a damping ratio equal to 7.0 percent of critical damping for the concrete dam structure only. The increase in the damping ratio from 5 percent for the OBE to 7 percent for the MCE helps account for some additional nonlinear behavior while using a linear-elastic approach.

Chapter 5
Design Response Spectra and Acceleration Time Histories

5-1. Defining the Design Earthquake

In a linear-elastic response spectrum analysis, response spectra define the free field ground motion for the design earthquake. A response spectrum gives the maximum damped response (expressed as displacement, velocity, or acceleration) of all possible linear single degree-of-freedom systems using the natural frequency (or period) to describe the system. Viscous damping expressed as a percentage of critical damping is used to develop a response spectra. A design earthquake is often defined by a set of response spectra for various damping ratios. The response spectra produced by recorded earthquake events are characterized by a jagged shape made up of peaks and valleys of varying magnitude; however, design response spectra are smoothed so that they are not frequency sensitive.

5-2. Developing Design Response Spectra

a. Deterministic and probabilistic approaches. Design response spectra are developed by using either a “deterministic approach” or a “probabilistic approach.” The probabilistic approach is based on probabilistic seismic hazard analysis methodology which in essence uses the same elements as the deterministic approach, but adds an assessment of the likelihood that ground motion will occur during a specified time period.

b. Procedures. There are two basic procedures for developing design response spectra using either the deterministic or probabilistic approach. They are: (1) anchoring the spectral shape to the peak ground acceleration; and (2) estimating the spectrum directly. Although procedure (1) is more often used, the use of procedure (2) is increasing, and for some situations is preferred because it incorporates factors besides just the local site conditions.

c. Obtaining design response spectra. It is beyond the scope of this EP to present the detailed procedures for developing design response spectra, or for forecasting PGA’s for design earthquakes. Refer to ETL 1110-2-301, ETL 1110-2-303, and “Tentative Provisions for the Development of Seismic Regulations for Buildings” (Applied Technology Council 1984) for further information on developing design response spectra to define the design earthquakes.

5-3. Developing Acceleration Time Histories

a. Matching design response spectrum. The more refined methods of analysis discussed in paragraph 2-2*d* are of the time-history type. Time histories usually express the ground motion as a record of acceleration with respect to time. Acceleration time histories should be developed so their response spectrum is consistent with the previously established site-specific design response spectrum described in paragraph 5-5*c*. The time histories should also have a strong motion duration appropriate to the particular design earthquake.

b. Procedures. There are two basic procedures for developing acceleration time histories: (1) selecting a suite of past recorded earthquake ground motions, and (2) synthetically developing or modifying one or more ground motions.

(1) When selecting a suite of time-history records for the first procedure, the intent is to cover the valleys of the spectrum produced by one record, which fall significantly below the site-specific design response spectrum, with better matching spectral values at these frequencies as produced by the other records in the suite. It is also necessary that the spectra produced by the suite of records not significantly exceed the site-specific design response spectrum. Primary advantage of this procedure is that the structure is analyzed by real, natural ground motions that are representative of what the structure could experience.

(2) When using the second procedure, it is possible to either completely synthesize an accelerogram, or modify an actual recorded earthquake accelerogram so that the response spectrum of the resultant accelerogram closely fits or matches the site-specific design response spectrum. The primary advantage of this procedure is that a good fit to the design response spectrum can be achieved with a single accelerogram, thus only a single dynamic analysis is required.

5-4. Dynamic Analysis by Modal Superposition

a. Frequencies and mode shapes. The linear-elastic response spectrum method utilizes modal superposition dynamic analysis to determine the structural response.

b. Time-history analysis. Once the modes are derived, the response of the complex multiple degree-of-freedom system is reduced to the solution of the simple, single basic equation of motion for a single degree-of-freedom (SDOF) system. For time-history analysis, the response is easily obtained using step-by-step integration of the equation of motion for the SDOF system for each significant mode based on the frequency (eigenvalue) of the mode. In essence the response contribution of each mode is determined for a series of time steps using a prescribed time-step interval, and the response at each time step is simply the superposition, or addition, of characteristic mode shapes adjusted by coefficients obtained from the integration procedure. Normally, only a few mode shapes are found to contribute significantly to the response, so that the modal superposition method produces a precise response with minimum computational effort.

c. Response spectrum analysis. In a response spectrum analysis, the step-by-step integration part of the dynamic analysis, described above for time-history analysis, is performed in the process of developing the response spectrum. The response spectrum may be envisioned as a display of the results of this part of the modal analysis, and it is presented in the form of "maximum" response versus frequency (or period). In the response spectrum modal analysis, eigenvalues, eigenvectors, and modal participation factors are computed and used in the analysis procedure just as they are in a time-history modal analysis. Precise "maximum" modal responses are easily calculated from a simple equation that relates these parameters and the appropriate spectral value that corresponds to the modal frequency.

d. Combining modal responses. The final step in a response spectrum analysis consists of correct superpositioning of the "maximum" modal responses; however, there is not a unique solution to this final step in the response spectrum method. This is because the exact mode contributions at the critical point in time when the response peaks are not available from a response spectrum representation of a particular ground motion. One advantage of a smooth design response spectrum is that it is a statistical representation, or an envelope, of the many possible ground motions that could occur at the site rather than only a single ground motion. The superposition of the maximum modal responses is accomplished by use of one of several statistical methods described in Chapter 7.

5-5. Types of Design Response Spectra

a. Probability level. Design response spectra are usually based statistically either on the mean, median (50th percentile probability level), or the median plus one standard deviation (84th percentile probability level), of the ground motion parameters for the records chosen. Design response spectra used for design of new RCC dams or for evaluation of the safety and serviceability of existing dams shall be based on the mean level of the ground motion parameters.

b. Type of spectrum required. Either a "site-specific" or a "standard" design response spectra shall be used to describe the design earthquakes. The type required shall be based on the seismic zone, the proximity of the seismic source, and the maximum height of the dam.

c. Site-specific design response spectra. The site-specific design response spectra should be developed based on earthquake source conditions, propagation path properties, and local foundation characteristics associated with the specific site. This type of design spectra may be established by anchoring a selected response spectral shape for the site to the estimated peak ground acceleration, or by estimating the design spectra directly using response spectral attenuation relationships, performing statistical analysis of strong-motion records, or applying theoretical (numerical) ground motion modeling. In the requirements that follow, a site is classified as a "high seismic risk site" when it is located within 20 kilometers of an active fault or area source in the western United States (WUS), or within a tectonic province in the eastern United States (EUS) where the source or province has a maximum local magnitude of 6.0 or greater. The boundary between the WUS and the EUS is defined as the eastern boundary of the Rocky Mountains. Site-specific design response spectra are required for:

(1) Dams greater than 100 feet in height located at a site classified as a “high seismic risk site.”

(2) Dams greater than 100 feet in height located in Seismic Zone 2B, 3, or 4 even though the site is not classified as a “high seismic risk site.”

(3) Dams not greater than 100 feet in height located in Seismic Zone 2B, 3, or 4 when the site is classified as a “high seismic risk site.”

d. Standard design response spectra. Standard design response spectra are based on fixed spectral shapes established for very general site classifications such as rock or soil site. They ignore the effects of earthquake magnitude and distance, and the specific foundation characteristics at the site. The standard design spectra are usually “anchored” to the estimated peak ground acceleration (PGA) established for the design earthquake. The fixed spectral shape is usually presented such that it is normalized to a 1.0 g value of maximum ground acceleration. This normalized value can be easily checked by observing the spectral acceleration value from the spectrum plot for frequencies above about 50 cps where the response and the maximum ground acceleration coincide. Standard design response spectra are adapted to the severity of ground motion associated with the OBE or MCE by using the PGA as a scaling factor. The standard design response spectra can be used for:

(1) Dams greater than 100 feet in height located in Seismic Zone 0, 1, or 2A when the site is not classified as a “high seismic risk site.”

(2) Dams not greater than 100 feet in height located in Seismic Zone 0, 1, or 2A.

(3) Dams not greater than 100 feet in height located in Seismic Zone 2B, 3, or 4 when the site is not classified as a “high seismic risk site.”

e. Required design spectrum. When it is acceptable to use a standard design response spectrum to define the design earthquakes, the standard design spectrum shown in Figure 5-2 shall be used (Applied Technology Council 1984). This spectrum is considered conservative but reasonable for essential structures such as dams. It is fully described by only five control points on a tripartite plot. Table 5-1 presents the spectrum in equation format so it is easily developed for any damping value. The standard design spectrum shown in Figure 5-2 and defined in equation format in Table 5-1 is normalized to 1.0 g PGA. The standard spectrum shall be anchored to the PGA for the OBE and the MCE by using the appropriate scaling factors provided in Table 5-2. The correct scaling factors are selected based on the seismic zone location of the site using the seismic zone map shown in Figure 5-1.

5-6. Horizontal and Vertical Design Response Spectra

a. Site-specific design response spectra. When site-specific design response spectra are required in accordance with paragraph 5-5*c*, two independent design response spectra shall be developed, one to define the horizontal component of ground motion, and the second to define the vertical component. The vertical component of ground motion usually contains much higher frequency content than the horizontal component, therefore the spectral shape is quite different than that of the horizontal component. The PGA associated with the vertical component will also be different than the PGA of the horizontal component. Both values of PGA are dependent on the distance from the source, but for short distances, the PGA of the vertical component may actually exceed the PGA of the horizontal component.

b. Standard design response spectra. When it is acceptable to use standard design response spectra to define the design earthquakes, the horizontal component of ground motion shall be defined by anchoring the standard design response spectra for the appropriate damping factor developed from Table 5-1 with the scaling factor provided in Table 5-2. The vertical component of ground motion shall utilize the same standard design response spectrum used for the horizontal component, but it shall be scaled using the appropriate ratio of the PGA for the vertical component to the PGA for the horizontal component as provided in Figure 5-3. This ratio is based on the site to source distance (R) and the fundamental natural period of vibration of the structure.

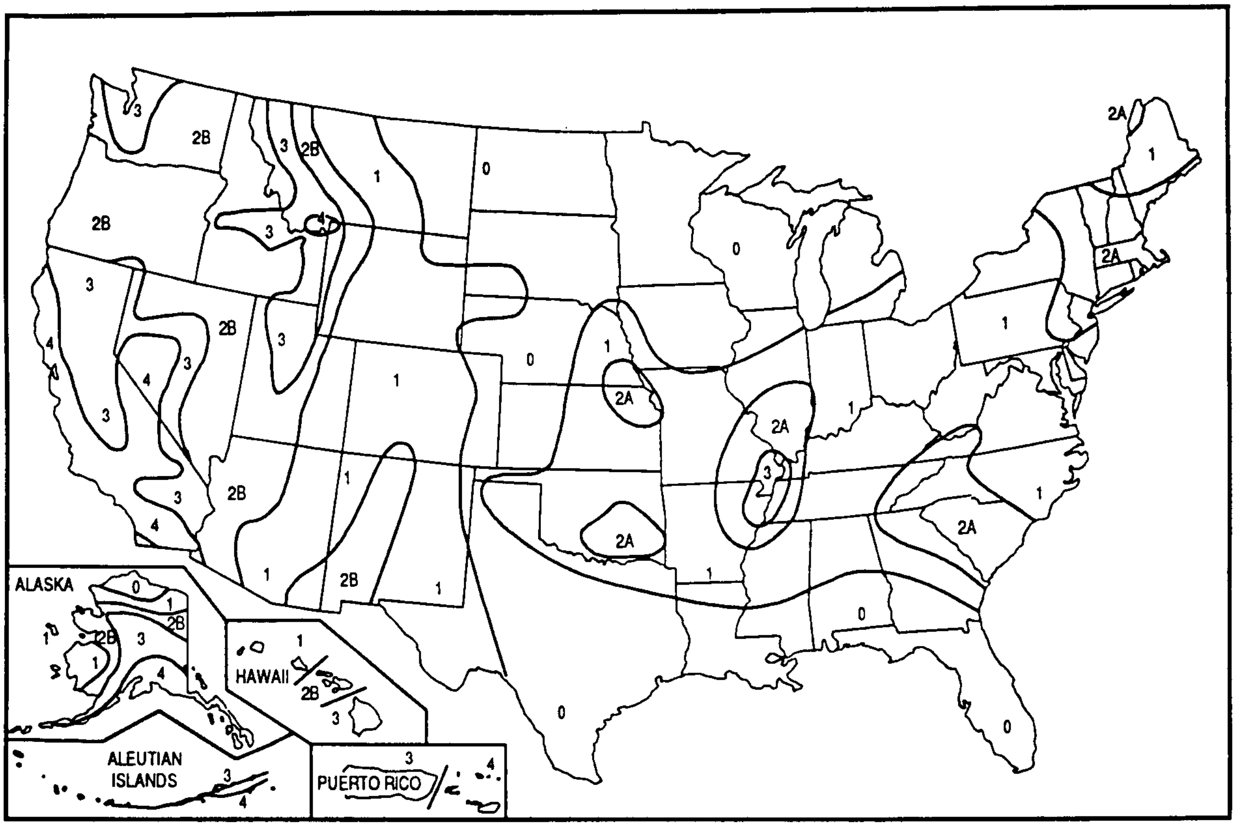
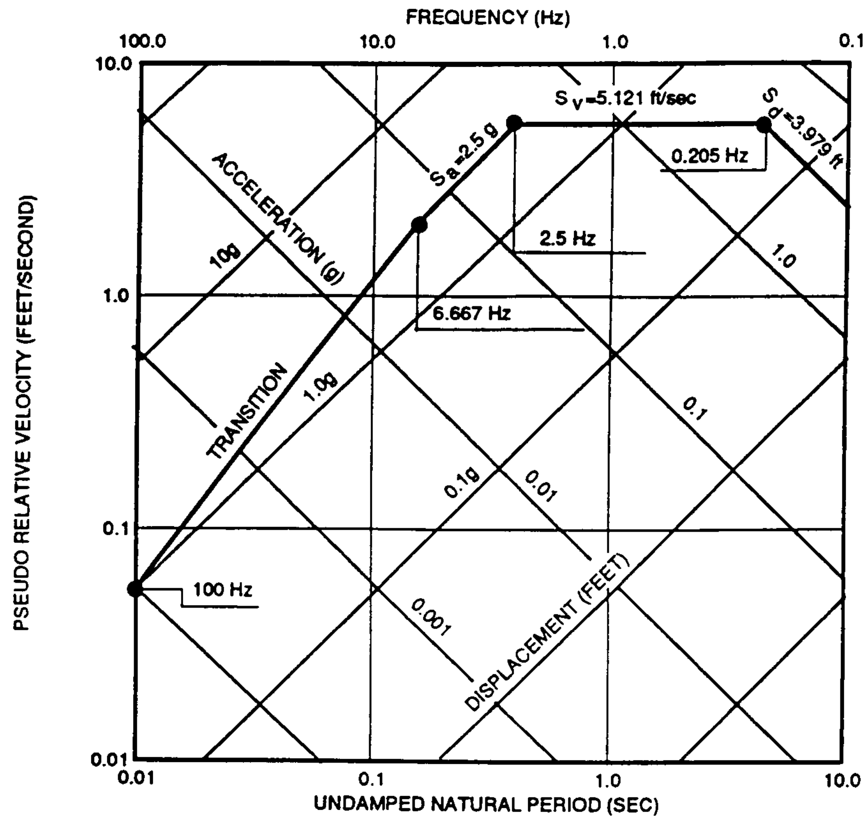
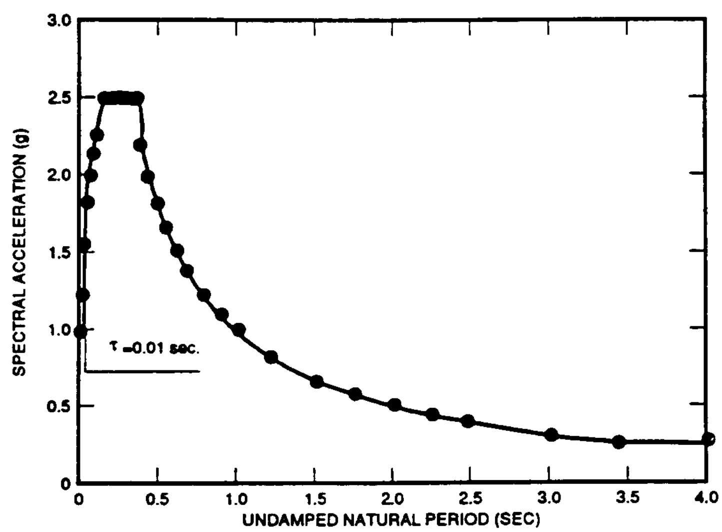


Figure 5-1. Seismic zone map of the United States. (Uniform Building Code, 1988 Edition)



Tripartite Logarithmic Representation (5% damped)



Arithmetric Plot of Spectral Acceleration versus Period (5% damped)

Figure 5-2. Standard design response spectra for horizontal component of ground motion - normalized to PGA = 1.0 g. (Applied Technology Council ATC-3-06 Tentative Provisions, 1984)

Table 5-1
Determining the Standard Design Response Spectrum for Horizontal Component of Ground Motion - Normalized to PGA = 1.0 g, for Any Value of β (Percent of Critical Damping)

	T (sec)	f (Hz)	$S_{a(5\%)}$(g's)	K_1	
*	0.002	500.000	1.0000	0.00000	EQUATION 1
	0.005	200.000	1.0000	0.00000	EQUATION 1
	0.008	125.000	1.0000	0.00000	EQUATION 1
*	0.010	100.000	1.0000	0.00000	EQUATION 1
	0.020	50.000	1.2643	0.25596	EQUATION 1
	0.040	25.000	1.5985	0.51192	EQUATION 1
	0.060	16.667	1.8335	0.66164	EQUATION 1
	0.080	12.500	2.0210	0.76787	EQUATION 1
	0.100	10.000	2.1795	0.85028	EQUATION 1
	0.120	8.333	2.3182	0.97160	EQUATION 1
*	0.150	6.667	2.5000	1.00000	EQUATION 1
	0.200	5.000	2.5000	1.00000	EQUATION 1
	0.250	4.000	2.5000	1.00000	EQUATION 1
	0.300	3.333	2.5000	1.00000	EQUATION 1
	0.350	2.857	2.5000	1.00000	EQUATION 1
*	0.400	2.500	2.5000	1.00000	EQUATION 1
	0.450	2.222	2.2222		EQUATION 2
	0.500	2.000	2.0000		EQUATION 2
	0.550	1.818	1.8182		EQUATION 2
	0.600	1.667	1.6667		EQUATION 2
	0.650	1.538	1.5385		EQUATION 2
	0.700	1.429	1.4286		EQUATION 2
	0.800	1.250	1.2500		EQUATION 2
	0.900	1.111	1.1111		EQUATION 2
	1.000	1.000	1.0000		EQUATION 2
	1.250	0.800	0.8000		EQUATION 2
	1.500	0.667	0.6667		EQUATION 2
	1.750	0.571	0.5714		EQUATION 2
	2.000	0.500	0.5000		EQUATION 2
	2.250	0.444	0.4444		EQUATION 2
	2.500	0.400	0.4000		EQUATION 2
	3.000	0.333	0.3333		EQUATION 2
	3.500	0.286	0.2857		EQUATION 2
	4.000	0.250	0.2500		EQUATION 2
*	4.882	0.205	0.2048		EQUATION 2
	5.000	0.200	0.1953		EQUATION 2
	6.000	0.167	0.1356		EQUATION 2
	7.000	0.143	0.0996		EQUATION 2
	8.000	0.125	0.0763		EQUATION 2
	9.000	0.111	0.0603		EQUATION 2
*	10.000	0.100	0.0488		EQUATION 2

* INDICATES THE ONLY POINTS NEEDED TO SPECIFY THE RESPONSE SPECTRUM FOR COMPUTER PROGRAMS WITH LOGARITHMIC INTERPOLATION CAPABILITY.

DEFINITION OF TERMS

S_a = SPECTRAL ACCELERATION IN g's FOR β PERCENT OF CRITICAL DAMPING

$S_{a(5\%)}$ = SPECTRAL ACCELERATION IN g's FOR 5 PERCENT OF CRITICAL DAMPING

T = UNDAMPED NATURAL PERIOD, SECONDS

f = FREQUENCY, Hz

β = PERCENT OF CRITICAL DAMPING

K_1, K_2, K_3 = CORRECTION FACTORS USED TO DEVELOP A DESIGN RESPONSE SPECTRUM FOR β PERCENT OF CRITICAL DAMPING

CORRECTION FACTORS

K_1 = FACTOR SHOWN IN TABLE FOR VALUES OF THE NATURAL PERIOD T BETWEEN 0.000 AND 0.400

$K_2 = 1.466 - 0.2895\ \mathrm{LN}(\beta)$

$K_3 = \mathrm{LOG}(2.5 \times K_2)$

EQUATIONS FOR S_a

WHEN $T \leq 0.400$ $S_a = 10.0^{K_1 K_3}$ (EQUATION 1)

$T > 0.400$ $S_a = K_2 S_{a(5\%)}$ (EQUATION 2)

NOTE: THE VALUES OF SPECTRAL VELOCITY S_v AND SPECTRAL DISPLACEMENT S_d CAN BE CALCULATED ONCE S_a IS KNOWN:

$S_v = 5.1207\ (S_a T)$

$S_d = 0.81498\ (S_a T^2)$

Table 5-2
Peak Ground Accelerations (PGA's) for Use in Scaling the Standard Design Response Spectra

	PGA	
Seismic Zone	Operating Basis Earthquake (OBE)	Maximum Credible Earthquake (MCE)
0	0.030	0.130
1	0.050	0.210
2A	0.095	0.360
2B	0.115	0.430
3	0.210	0.550
4	0.270	0.610

NOTES:

1. Refer to Figure 5-1 for the seismic zone maps.
2. PGA's are expressed as the decimal ratio of the acceleration due to gravity (g).
3. PGA's are obtained from curves of "Annual Risk of Exceedance vs. PGA" in Figure C1-7 of ATC-3 Tentative Provisions, April 1984.
4. The PGA for the OBE is based on a 50 percent chance of exceedance in 100 years.
5. The MCE is considered to be the event with a 5,000-year return period (annual risk of exceedance = 0.0002 chance/year).

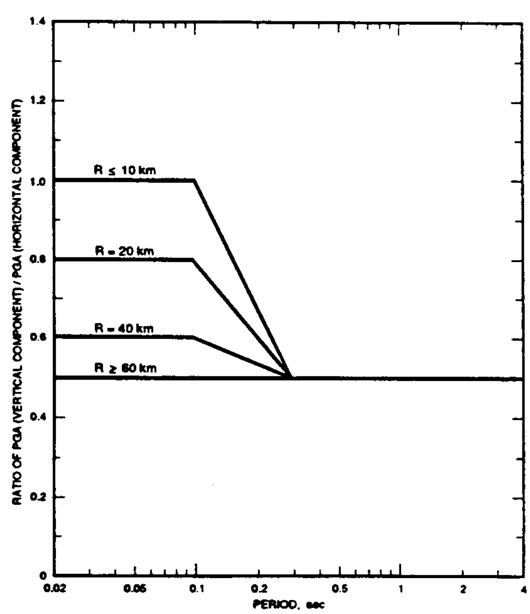


Figure 5-3. Ratio of PGA for the horizontal component to the PGA for the vertical component as a function of source to site distance (R) and the fundamental period of vibration of the structure

Chapter 6
Earthquake Load Cases

6-1. Load Combinations

The cyclic and oscillatory nature of vibratory response can cause critical tensile stresses to occur in either the upstream or the downstream face of the dam. Therefore, the earthquake load cases must consider combinations of the design earthquake loading with other loads which lead to critical tension in both the upstream and downstream faces. Usually two or more OBE load cases and two or more MCE load cases must be evaluated. The discussion of earthquake load cases that follows refers to seismic criteria regarding ground shaking and foundation fault displacement as discussed in paragraphs 2-2 and 2-3, respectively, and not stability criteria described in paragraph 2-1. Load case requirements for stability are covered in EM 1110-2-2200.

6-2. Dynamic Loads To Be Considered

The design earthquake imposes several types of dynamic loads on the dam. The greatest dynamic load is the inertia load caused by the response of the concrete mass to ground motion accelerations. Next is the hydrodynamic load created by a high reservoir and tailwater condition. Hydrodynamic forces are imposed on the dam due to motions of the dam reacting with the surrounding water, and motions of the reservoir bottom. Finally, backfill or silt deposits against the faces of the dam will interact with the structural mass of the dam in a manner similar to the hydrodynamic load.

6-3. Static Loads To Be Considered

The effects on the dam structure due to static loads, as discussed below, are determined by conventional static analysis methods. The results of the dynamic and static analyses are combined by superposition to determine the total stresses for the earthquake load case.

a. Reservoir and tailwater loads. Load cases shall be included to cover both the highest and the lowest reservoir pool elevations that can be judged on a statistical basis to have a reasonable chance of occurrence at the time of the design earthquake.

(1) Flood frequency data from project flood flow and flood routing studies provide a basis for establishing reasonable high pool elevations. Each dam must be evaluated based on its own set of unique conditions.

(2) The conservation pool elevation for the project shall be used for earthquake load cases involving low pool conditions. If there is no established conservation pool, use the lowest average pool elevation that can best be judged to exist for a 30-day period in a normal yearly flow cycle.

(3) Where tailwater is applicable for an earthquake load case, the elevation shall be selected which increases the response while being consistent with the reservoir conditions.

b. Backfill load. Earth or rock fill placed against either face of the dam has both a static and dynamic load effect during an earthquake. These loads shall be included in all earthquake load cases. Static loading shall be based on at-rest pressures. Dynamic loading may be approximated by the Mononobe and Okabe method utilizing the inertia force acting on the Coulomb sliding wedge in the appropriate direction as discussed in EM 1110-2-2502. For finite element analyses the dynamic effect may be approximated by added mass based on the Coulomb sliding wedge.

c. Siltation load. During the life of the dam, silt may build up against the upstream face to a depth which may cause a moderate increase in the tensile stresses in load cases where tension in the upstream face is critical. For these load cases, siltation loading shall be considered based on the full depth expected during the life of the dam. In load cases where tension in the downstream face is critical, the siltation load will decrease the tensile stresses. For these load cases a zero depth of silt shall be assumed. When silt is included, both static and dynamic loading effects should be incorporated using the same methods as discussed for backfill loads.

d. Gravity loads. Gravity loads shall include the weight of the RCC, weight of backfill or silt on battered faces of the dam, and weight of equipment if significant.

6-4. Static Loads Not To Be Considered

There are several types of loads where the magnitude of the load and the load pattern that would exist at the time of the design earthquake event cannot be defined on a logical basis or to any degree of accuracy. However, based on the general nature and range of magnitude normally associated with loads of this type, and in comparing these loads with the dynamic and static loads already discussed, these loads normally do not contribute significantly to the results of the analyses for earthquake load cases. However, the designer should at least make a cursory evaluation of these loads to be sure that no unusual site conditions exist that would warrant including one or more of them in the earthquake load cases. For this reason, a brief discussion of these loads is included.

a. Pore pressure. When evaluating dam stability using the seismic coefficient method described in paragraph 2-1, uplift is considered to act over the entire interface area. Under the MCE, any cracking in the concrete would only extend just beyond the microcracking level. These fine cracks are open and subject to buildup of internal water pressure for a short period of time due to the oscillatory nature of the dynamic response. Therefore, uplift or internal water pressure within concrete cracks would be quite small and may be ignored in the dynamic analysis phase of design.

b. Temperature stresses. Except under extreme climatic conditions, temperature stresses need not be included as part of the earthquake load cases.

c. Wind load. Wind load on an RCC dam is so small it can be considered insignificant.

d. Ice load. Ice loading need not be included as part of an earthquake load case except for unusual climatic conditions which would cause a great depth of ice to exist over an extended period of time.

Chapter 7
Factors Significantly Affecting Dynamic Response

7-1. Evaluation Procedure and Objectives

There are many important factors in a dynamic stress analysis that can greatly affect the response of a dam. The influence which the various material and strength parameters and loads have on the final results must be evaluated. This can be done by executing the model using a typical dam cross-section and typical material properties, then modifying the loads and parameters one-by-one to give an indication of the influence each factor has on the dynamic response. Once the important factors have been identified, the design effort should concentrate on the more critical factors that form the input to the dynamic analysis. Following is a discussion of the impact some of the parameters have on the response of a dam.

7-2. Design Response Spectra

a. Spectral shape. Both the shape of the spectrum and the PGA used to anchor the spectrum affect the dam response and should be established carefully. The dynamic response in a linear-elastic analysis is directly proportional to the PGA, but minor changes in the shape of the spectra may not result in proportional changes in the response.

b. Comparison of standard spectra. For comparison purposes, three widely accepted standard design response spectra will be considered, each representing the same site conditions. The design spectra are: (1) Applied Technology Council spectrum for rock of any characteristic whether shalelike or crystalline in nature (ATC 1984), (2) H. B. Seed spectrum for rock based on 28 records (Seed 1974), and (3) Newmark-Hall spectrum using recommended values for maximum ground velocity and displacement for competent crystalline rock (Newmark and Hall 1987). Figure 7-1 shows all three spectra normalized to 1.0 g PGA for the same rock foundation site conditions. The Newmark-Hall spectrum is based on the median or 50th percentile cumulative probability, where the other two spectra are based on the mean of the records used in their development. This difference in probability level is reflected in the spectral shape. The primary cause for the difference in shape of these three spectra can be attributed to the assumptions and techniques used in smoothing the jagged spectra produced from the statistical combination of real earthquake records.

c. Spectral accelerations. Referring to Figure 7-1, the range of interest of natural period would be for periods of less than 1.0 second. This range would cover the mode shapes that produce significant response. In this range the spectral acceleration values for a given period vary between spectra up to as much as 65 percent. The ATC spectrum envelopes the other two design spectra, and is recommended for use as the standard design response spectrum. In linear-elastic response spectrum analyses, dynamic response of a particular system evaluated by two different response spectra is directly proportional to the spectral ordinates taken from the two spectra at the natural period of the system. Thus the shape of the design response spectrum greatly influences the results of the dynamic analysis.

7-3. Dam-Foundation Interaction, Damping Effect

a. Properties of the foundation. The two properties of the foundation rock that have a significant influence on the dynamic response are the damping ratio and the deformation modulus. The damping characteristics of the foundation contribute significantly to the damping of the combined dam-foundation system and must be considered in the analysis. When the foundation deformation modulus is low, the damping ratio of the combined system is considerably higher than the damping ratio of the RCC dam structure alone.

b. Effective damping ratio. There are two sources of damping for the foundation rock: (1) material (hysteretic) and (2) radiation. In contrast to this type of damping is the viscous type of damping (directly proportional to velocity) used in producing design response spectra. Therefore, it is necessary to develop an effective viscous damping ratio to represent the combined dam-foundation system in a response spectrum analysis. This is accomplished by using the curves provided in Figure D-6 of Appendix D, and the following equation is for an empty reservoir condition which allows the effects of foundation damping to be isolated. This method, developed by A. K. Chopra, is based on the

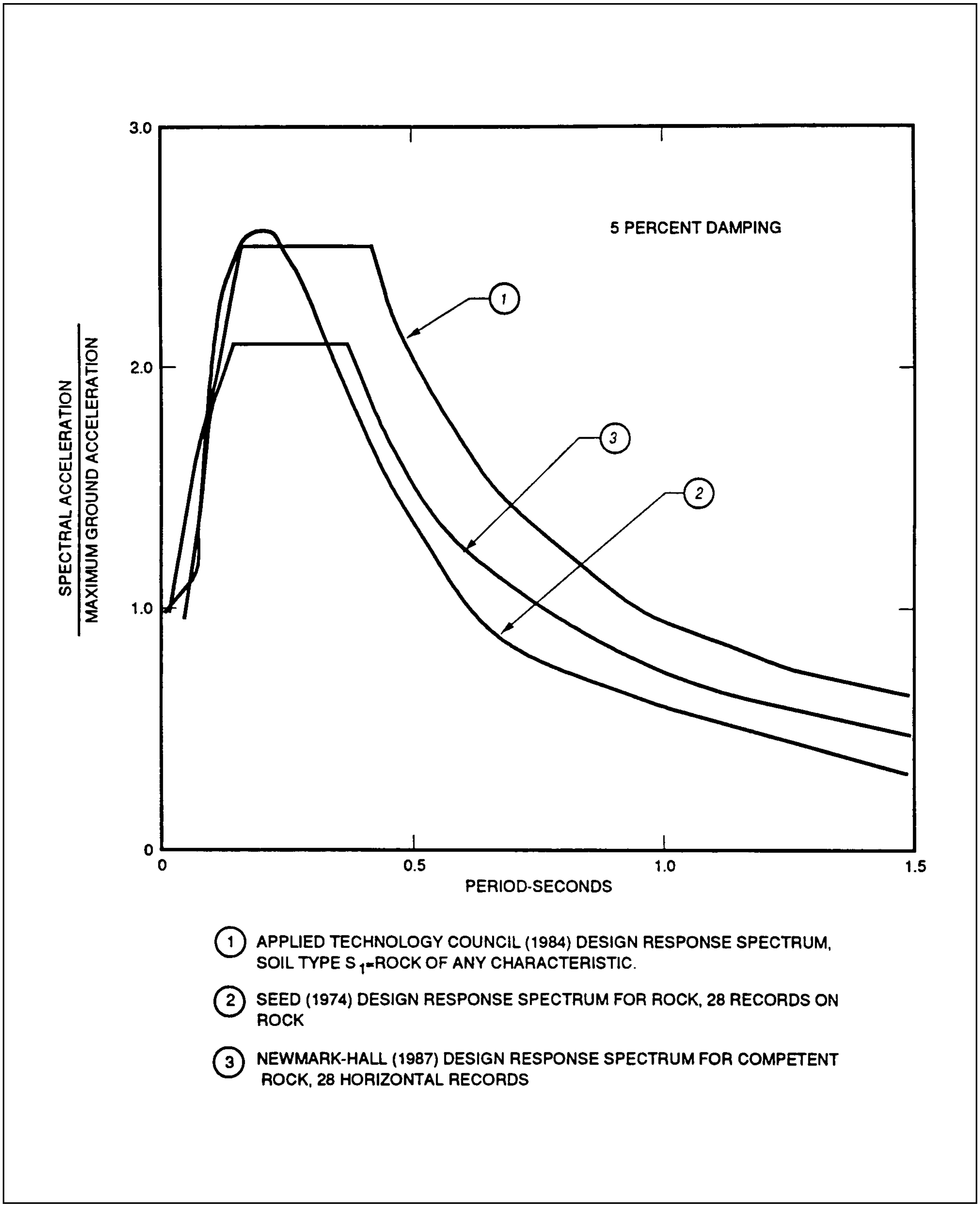


Figure 7-1. Comparison of design response spectra for rock foundations

fundamental mode of vibration, and has been shown to be reasonably close for the significant higher vibration modes (Fenves and Chopra 1986). In Figure D-6, damping for the foundation rock is expressed by the constant hysteretic damping factor.

$$\bar{\xi}_1 = \frac{1}{(R_f)^3}\xi_1 + \xi_f$$

where

$\bar{\xi}_1$ = the effective viscous damping ratio for the empty reservoir condition

ξ_1 = the viscous damping ratio for the RCC dam structure only

ξ_1 = 5.0 percent for the OBE
ξ_1 = 7.0 percent for the MCE

R_f = ratio of the fundamental period of the dam on a rigid foundation to the fundamental period of the dam on a foundation with a deformation modulus = E_f

ξ_f = added damping ratio due to dam-foundation rock interaction taken from Figure D-6

c. Effect of damping on response. To determine the effect that the damping ratio has on the response of a dam, the fundamental frequency of the composite finite element dam-foundation model must be determined. It is noted that for the response spectrum method, the effects of damping are contained only in the response spectrum itself. Thus, the ratio of the response of a dam/foundation system responding at one damping factor to the same system responding at a second damping factor is equal to the ratio of the spectral ordinates taken from the two spectra evaluated at the fundamental frequency of the system.

d. Conclusion. The damping characteristics of the foundation can have a great influence on the dynamic response. This indicates the need to carefully determine the value of the constant hysteretic damping factor for the foundation rock. This can be determined from experimental tests of appropriate rock samples subject to harmonically varying stress and strain. From such tests, the inelastic energy lost and the strain energy stored per cycle are determined and the hysteretic damping factor is calculated.

7-4. Dam-Foundation Interaction, Foundation Modulus Effect

a. Modulus of deformation. The flexibility of the jointed rock foundation is characterized by the modulus of deformation which represents the relationship between applied load and the resulting elastic plus inelastic deformation. It is best determined by in-situ testing, but may be estimated from the elastic modulus of the rock by applying an appropriate reduction factor. In a linear-elastic analysis, the modulus of deformation is synonymous with Young's modulus of elasticity (E_f).

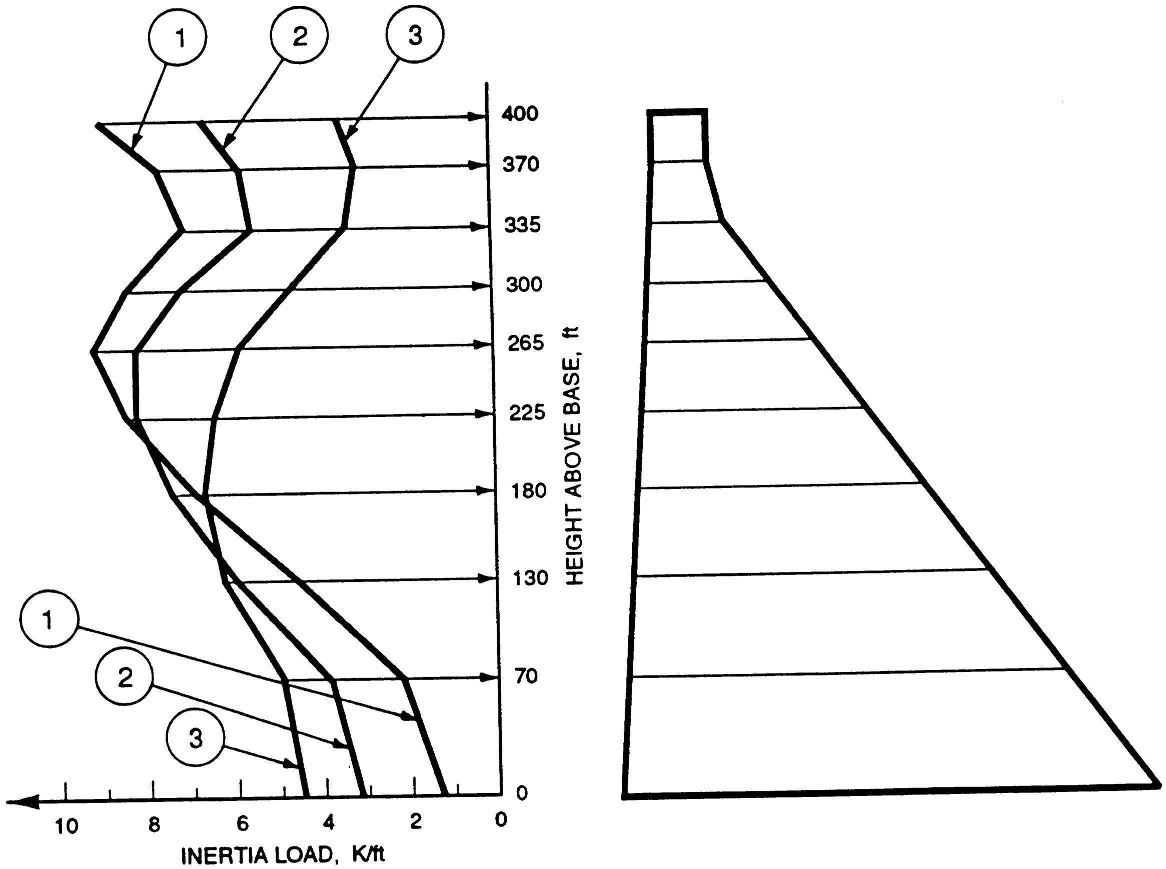
b. Dynamic characteristics affected. The elastic modulus of the foundation influences the response because it directly affects the following dynamic characteristics of the dam-foundation system:

(1) Modal frequencies. As the modulus of deformation decreases, the modal frequencies of the composite dam/foundation system also decrease.

(2) Mode shapes. As the modulus of deformation decreases, the mode shapes are affected by increased rigid body translations and rotation of the dam on the elastic foundation.

(3) Effective damping ratio. As the modulus of deformation decreases, the effective damping ratio of the dam/foundation system increases.

c. Effect of foundation modulus on response. To determine the effect of the foundation modulus on dynamic response, a typical dam model was analyzed on foundations that bracket a wide range of foundation stiffness from infinitely stiff (E_s/E_f = 0.0), to relatively flexible (E_s/E_f = 2.5). The response was expressed as the distributed lateral inertia loading acting over the full height of the dam. Figure 7-2 shows the response graphically for three different values of E_s/E_f. It is noted that the total inertia load, or base shear, only varied by 15 percent, but a considerable variation occurred in the load pattern. As the foundation becomes more flexible, the greatest inertia load shifts from the upper portion of the dam to the lower portion. This would be accompanied by a considerable change in the concrete stresses.



KEY PARAMETERS

	E_s / E_f	$\tilde{T}_1$	R_f	$\bar{\varepsilon}_t$	$\tilde{S}_a$	BASE SHEAR
1	0	0.295	1.000	5.0	25.12	$2{,}450^K$
2	0.5	0.333	1.127	7.08	18.23	$2{,}489^K$
3	2.5	0.462	1.565	15.6	11.16	$2{,}171^K$

Figure 7-2. Fundamental mode response expressed as the distributed lateral inertia load for various foundation stiffnesses

7-5. Hydrodynamic Effect

a. Dynamic characteristics affected. Hydrodynamic load results from the interaction of the reservoir and the structural mass of the dam in response to ground motion. The dam-reservoir interaction changes the water pressure acting on the face of the dam, and directly affects the following dynamic characteristics of the system:

(1) Modal frequencies. As the depth of the reservoir increases beyond a depth equal to about one half of the height of the dam, there begins to be a noted decrease in the modal frequencies.

(2) Mode shapes. The equivalent added mass to account for reservoir effects, as discussed in paragraph 7-5*c*, changes the relative distribution of mass in the system. Thus, the normalized mode shapes will be affected to some degree.

(3) Effective damping ratio. As the depth of the reservoir increases, dam-reservoir interaction tends to increase the effective damping ratio.

b. Added mass based on Westergaard's formula. Accounting for hydrodynamic effects when using a composite finite element model (refer to paragraph 8-1*d*(3)(a)) requires developing an equivalent mass system which strategically adds mass to the dam-foundation model. The amount and location of the added lumped masses must be such that they correctly alter the dynamic properties described above in a manner which will also produce the desired pressure changes. Often the added mass is calculated based on Westergaard's pressure diagram divided by the acceleration due to gravity to convert it from a distributed load to a distributed mass.

c. Added mass based on Chopra's method. A. K. Chopra's Simplified Analysis Procedure (Chopra 1978) uses an equivalent mass system to consider compressibility of water and the dynamic properties of the dam and reservoir bottom. Chopra suggests that the key parameter that determines the significance of water compressibility is

$$\Omega_r = \frac{\omega_1^{\,r}}{\omega_1}$$

where

Ω_r = water compressibility significance parameter

$\omega_1^{\,r}$ = fundamental frequency of the impounded water idealized by a fluid domain of constant depth and infinite length

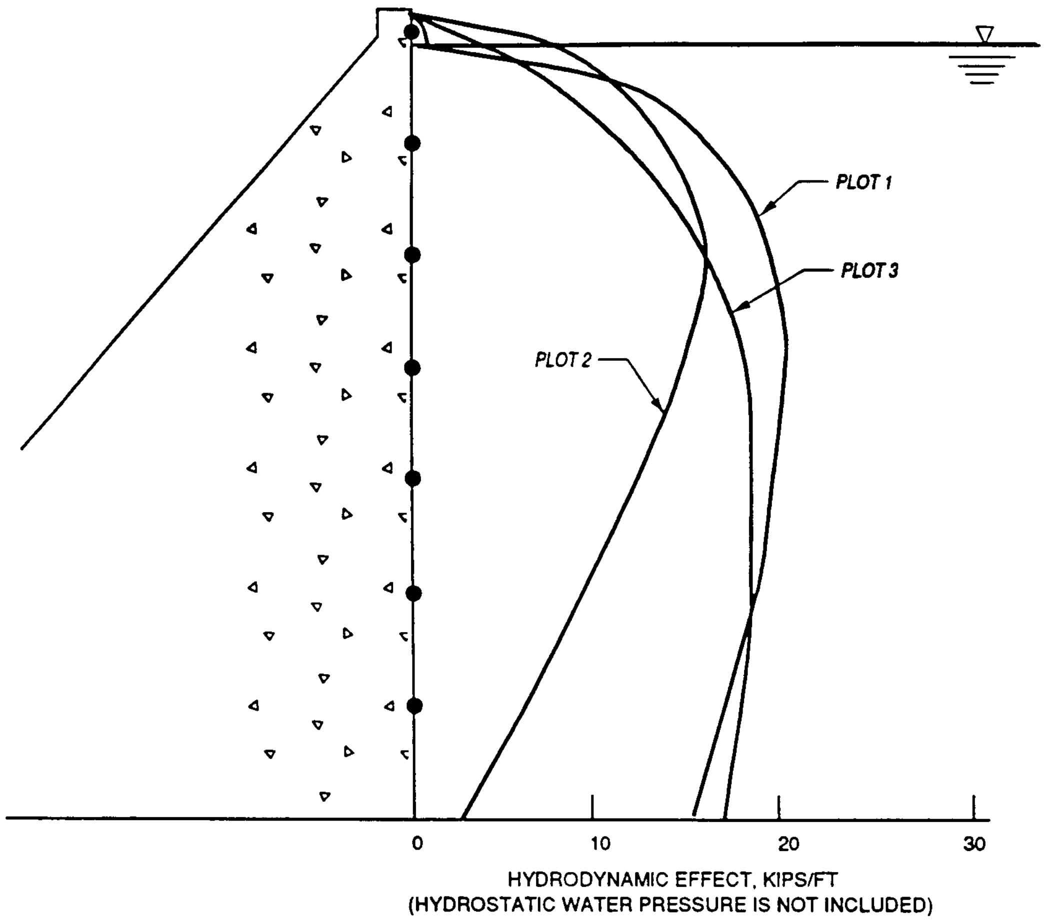
ω_1 = fundamental frequency of the dam alone

and when

$\Omega_r \le 0.5$, compressibility of water is significant and should be accounted for in determining the hydrodynamic effect

d. Standard pressure function curves. In Chopra's system, the hydrodynamic pressure distribution and equivalent mass system are derived using a set of standard hydrodynamic pressure function curves. The equivalent mass system for the composite finite element method may be developed using the same principles as those for the Simplified Procedure. The added mass is determined by using the appropriate pressure function curve, certain equations from Chopra's Simplified Procedure, and the fundamental mode shape and frequency obtained from the finite element analysis of the dam-foundation model. Some additional requirements applying to added mass are discussed in paragraph 7-8*c*, and complete details for deriving the equivalent mass system for the composite finite element method are provided in Appendix D of this EP.

e. Hydrodynamic pressure distribution. Figure 7-3 shows the hydrodynamic pressure distribution associated with the fundamental mode for a typical dam with a high reservoir condition. Plot 1 shows the distribution calculated by Chopra's Simplified Procedure, where Plot 2 and Plot 3 were obtained using the composite finite element method with equivalent mass systems as discussed above. The added mass for Plot 2 was based on Westergaard's formula, and the added mass for Plot 3 was based on the standard pressure function curves and the method described in Appendix D. To extract the hydrodynamic pressure distribution using the composite finite element method, the dynamic analysis was



LEGEND

PLOT 1 CALCULATED USING CHOPRA'S SIMPLIFIED ANALYSIS PROCEDURE FOR THE FUNDAMENTAL MODE RESPONSE WHICH INCORPORATES THE ELASTIC STIFFNESS CHARACTERISTICS OF THE DAM AND THE FOUNDATION, AND COMPRESSIBILITY OF WATER.

PLOT 2 RESULTS FROM FINITE ELEMENT ANALYSIS WHICH MODELS THE HYDRODYNAMIC EFFECT AS ADDED MASS CALCULATED BY WESTERGAARD FORMULA.

PLOT 3 RESULTS FROM FINITE ELEMENT ANALYSIS WHICH MODELS THE HYDRODYNAMIC EFFECT USING THE "EQUIVALENT MASS SYSTEM" WITH THE ADDED MASS CALCULATED USING CHOPRA'S STANDARD HYDRODYNAMIC PRESSURE FUNCTION CURVES AND EQUATIONS FROM THE SIMPLIFIED ANALYSIS PROCEDURE.

Figure 7-3. The hydrodynamic effect expressed as an "equivalent applied static pressure"

first performed on the dam-foundation model without added mass (which represents the empty reservoir condition), and then a second dynamic analysis was performed on the model with added mass. The difference in the unbalanced nodal forces between these two analyses represented the hydrodynamic forces exerted on the nodes. From these nodal forces the pressure distribution was readily determined.

f. Comparison of hydrodynamic methods. Although Chopra's Simplified Procedure is only an approximate procedure based on the standard fundamental mode shape and simplified methods for determining the required periods of vibration, it is assumed that the procedure provides hydrodynamic loading that is at least within the general order of accuracy expected in dynamic analyses. On this basis, the equivalent mass system based on Westergaard's formula (Plot 2), underestimated the hydrodynamic loading on the typical dam section by about 40 percent. The equivalent mass system developed by the method described in Appendix D produced hydrodynamic loading (Plot 3) which correlated reasonably well with Chopra's Simplified Procedure. On this basis, the method described in Appendix D, which uses the standard pressure function curves, is recommended for developing the equivalent mass system.

g. Hydrodynamic contribution to response. For high pool conditions, a large portion of the dynamic response is attributable to the hydrodynamic effect. In the example that produced Plot 3 in Figure 7-3, 47 percent of the total equivalent mass system consisted of the added mass representing the hydrodynamic effects. Therefore, the equivalent mass system will significantly affect the response for pool depths greater than about half of the height of the dam.

7-6. Reservoir Bottom Absorption

a. Wave reflection coefficient. The nonrigid reservoir bottom partially absorbs incident hydrodynamic pressure waves. This moderates the increase in response of the dam due to the dam's interaction with the impounded water. This is readily apparent by comparing the standard hydrodynamic pressure function curves for two different reservoir bottom absorption conditions. Reservoir bottom absorption is expressed by a wave reflection coefficient which varies from zero for a fully absorptive condition to 1.0 for a fully reflective condition. Figure D-4 in Appendix D shows the pressure function curves for reservoir bottom conditions with wave reflection coefficients of 0.50 and 0.75. As apparent from these curves, the hydrodynamic pressure increases with an increase in the reflection coefficient.

b. Effects of R_w. When the fundamental vibration period of impounded water and the fundamental period of the dam are approximately equal, R_w approaches 1.0. This condition indicates the approach of a state of resonance, and the pressure function then becomes quite large for a nonabsorptive reservoir bottom. In contrast, the pressure function for an absorptive bottom is much less affected by the approach of resonance, because the effect of reservoir bottom absorption is to reduce the large resonant displacement peaks.

c. Estimating reservoir bottom absorption. Assuming a nonabsorptive reservoir bottom may lead to an overly conservative hydrodynamic response for dams when the earthquake load condition includes a high forebay pool. The degree of adsorptiveness characterized by the wave reflection coefficient is usually difficult to determine reliably. The value of the wave reflection coefficient will likely increase during the life of the dam as sediments are continuously deposited. Therefore, it is recommended that the effects of reservoir bottom absorption be included in the dynamic analysis by using a wave reflection coefficient based on the properties of the impounded water and the foundation rock, and neglect the additional adsorptiveness due to sediments that will eventually be deposited (Fenves and Chopra 1984). The wave reflection coefficient is determined by the following equation:

$$\alpha = \frac{1 - k}{1 + k}$$

where

α = wave reflection coefficient

$k = \rho C/\rho_r C_r$

ρ = mass density of water = 1.938 (lb-sec^2) /ft^4

C = velocity of pressure waves in water = 4,720 ft/sec

ρ_r = mass density of the foundation rock in (lb-sec^2) /ft^4

C_r = velocity of pressure waves in the foundation rock = $12\sqrt{E_f/\rho_r}$

E_f = deformation modulus of the foundation rock in lb/in.^{2}

7-7. Method of Combining Modes

a. Maximum modal responses. The maximum modal response in a response spectrum analysis is the maximum possible contribution that a particular mode can make to the dynamic response. However, all the modes do not arrive at their maximums at the same point of time during the period of ground motion. Thus, for a single ground motion record there is one point in time when the maximum dynamic response is reached, and this maximum response is made up of various fractional parts of the individual maximum modal responses. The "fractional parts" are unique for each ground motion record. If a response spectrum analysis is made for a single ground motion record, the maximum dynamic response can only be approximated because the exact makeup of the "fractional parts" of the maximum modal responses cannot be computed. A time-history analysis is required to determine the exact solution for a given ground motion record.

b. Statistical combination methods. A smooth design response spectrum may be considered as a convenient representation of many possible ground motion records that could make up the design earthquake. As discussed in paragraph 5-5*a*, design response spectra are often referred to as statistical representations of the ground motion records used in their development (such as mean, median, 84th percentile). On a similar basis, the maximum modal responses of a response spectrum analysis are combined by statistical methods to produce a reasonable dynamic response to the many possible ground motions that could make up the design earthquake.

c. Coupling coefficients. Tables 7-1 and 7-2 present four commonly used mode combination methods. The difference in the methods is in the calculation of the coupling coefficient between modes. The coupling coefficients may be simple discrete functions as is the case with the square root of the sum of the squares method (SRSS) which treats the modal responses as random variables. The functions may be more complex involving modal frequencies or both modal frequencies and damping factors as is the case with the complete quadratic combination method (CQC) and the double sum method (DSM). The more complex methods give additional accounting in the coefficient calculation when the frequencies of the two modes under consideration are close. Two closely spaced modes are coupled, and when one of the modes is excited, it tends to excite the other mode. However, the modal frequencies associated with gravity dams are normally fairly well separated.

d. Comparing methods. The base shear was the response parameter used for comparing the four combination methods. By using several load cases, foundation conditions, and damping ratios, eight sets of maximum modal base shear values were made available to test the combination methods. The more complex methods, CQC and DSM, increased the coupling coefficients for closely spaced mode which produced greater combined responses than the SRSS method. The spacing of the modal frequencies for the TPM was such that no two modes qualified for "additional accounting," so the combined response for the TPM is the same as SRSS. The two most often used methods are SRSS and CQC.

e. Conclusion. The mode combination method does not greatly affect the order of accuracy of the dynamic analysis. The factors discussed previously have far greater influence on the dynamic response. The preliminary design of new dams, and the final design of dams not considered to be under critical seismic conditions, may use either the SRSS or the CQC method. Final design of dams under critical seismic conditions and evaluation of existing dams shall use the more refined CQC method.

7-8. Vertical Component of Ground Motion

a. Factors that contribute to the response. It is very difficult to make a general assessment of the influence of the vertical component of ground motion on the total dynamic response because of the number of factors involved. The vertical component of ground motion can be significant under certain conditions. The most important factors that affect the contribution of the vertical component to the response are:

Table 7-1
Combining Modal Responses: Square Root of the Sum of the Squares Method (SRSS) and Ten Percent Method (TPM)

STATISTICAL METHODS consider the phasing of the modes by utilizing a “coupling coefficient” between the various modes as expressed by the basic equation:

$$R = \left[\sum_{i=1}^{N} \sum_{j=1}^{N} R_i \, P_{ij} \, R_j \right]^{1/2}$$

where: N = number of modes to be considered
R = total modal response
R_i = maximum modal response in the ith mode
R_j = maximum modal response in the jth mode
P_{ij} = coupling coefficient between modes *i* and *j*

There are several methods for determining the P_{ij} values. They are given below in the order of complexity:

Method 1: Square Root of the Sum of the Squares (SRSS)

$$P_{ij} = \begin{cases} 1.0 \ \ if \ \ i = j \\ 0.0 \ \ if \ \ i \neq j \end{cases}$$

The basic equation then reduces to

$$R = \sum_{i=1}^{N} \left[R_i^2 \right]^{1/2}$$

Method 2: Ten Percent Method (TPM)

$$P_{ij} = \begin{cases} 1.0 \ \ if \ \ \dfrac{\omega_j - \omega_i}{\omega_i} \leq 0.1 \\[2ex] 0.0 \ \ if \ \ \dfrac{\omega_j - \omega_i}{\omega_j} > 0.1 \end{cases}$$

where

ω_i = the natural frequency for the ith mode

ω_j = the natural frequency for the jth mode

This method gives additional accounting for modes with nearly the same frequency. If none exist, TPM reduces to SRSS.

Table 7-2
Combining Modal Responses: Complete Quadratic Combination Method (CQC) and Double Sum Method (DSM)

Method 3: Complete Quadratic Combination (CQC)

$$P_{ij} = \left[\frac{8\sqrt{\varepsilon_i \ \varepsilon_j} \ (\varepsilon_i + r\varepsilon_j) r^{3/2}}{\left(1 - r^2\right)^2 + 4\varepsilon_i \ \varepsilon_j \ r \left(1 + r^2\right) + 4\left(\varepsilon_i^2 + \varepsilon_j^2\right) r^2} \right]$$

where

ε_i = modal damping ratio for the ith mode

ε_j = modal damping ratio for the jth mode

$$r = \frac{\omega_j}{\omega_i}$$

This method is based on both modal frequency and modal damping. However, for design of gravity dams, there is no procedure available to establish reasonable damping ratios for the higher modes. The effective viscous damping factor calculated according to the recommended procedure in this EP is used for all modes.

Method 4: Double Sum Method (DSM)

$$P_{ij} = \left[1 + \left\{ \frac{(\omega_i^{/} - \omega_j^{/})}{(\varepsilon_i^{/} \ \omega_i + \varepsilon_j^{/} \ \omega_j)} \right\}^2 \right]^{-1}$$

where

$$\begin{aligned}
\omega_i^{/} &= \omega_i \ (1 - \varepsilon_i^2)^{1/2} \\
\omega_j^{/} &= \omega_j \ (1 - \varepsilon_j^2)^{1/2} \\
\varepsilon_i^{/} &= \varepsilon_i + \frac{2}{t_d \omega_i} \\
\varepsilon_j^{/} &= \varepsilon_j + \frac{2}{t_d \omega_j} \\
t_d &= 10 \ \mathit{seconds} \ (\mathit{earthquake} \ \mathit{duration})
\end{aligned}$$

This method is similar to CQC, but is slightly more conservative.

Note: Refer to Table 7-1 for the basic equation for obtaining the total modal response, and for definition of terms not provided on this table.

(1) The PGA associated with the vertical component. In some instances the vertical component PGA may be as great or greater than the horizontal component PGA. Refer to paragraph 5-6*a*.

(2) The shape of the vertical component design response spectrum. The frequency content of the vertical component of ground motion is usually higher than the frequency content of the horizontal component. This causes the vertical spectrum shape to be different than the horizontal spectrum shape. The vertical component will excite modes in the lower frequency range less than will the horizontal component.

(3) The depth of the reservoir. Vertical ground motion causes hydrodynamic pressure waves to be generated which exert a lateral load against the face of the dam (this hydrodynamic load is in addition to that discussed in paragraph 7-5). When considering stresses caused by the vertical component of ground motion, the stress induced by the hydrodynamic pressure waves can be larger than the stress caused by the inertia response associated with the mass of the dam. For a nonabsorptive reservoir bottom, the hydrodynamic load theoretically reaches infinity at the natural vibration frequencies of the reservoir. This is in contrast to stresses caused by the horizontal component of ground motion where the stress caused by the hydrodynamic load is small compared to the stress caused by the inertia response associated with the mass of the dam.

(4) Reservoir bottom absorption. Reservoir bottom absorption greatly reduces the added hydrodynamic load due to vertical ground motion and eliminates the unbounded peaks in the response, described above, at excitation frequencies equal to the natural vibration frequencies of the reservoir.

b. Method of analysis. Except for the hydrodynamic load contribution which is discussed later, determining the response due to the vertical component of ground motion follows the same general procedures and recommendations that apply in determining the horizontal component response. The vertical component design response spectrum, and the PGA associated with vertical excitation are used to define the design earthquake. It should be noted that for vertical direction excitation, the fundamental mode and some or all of the significant higher modes are often different than for horizontal excitation. The participation factor and the mode coefficient for a particular mode and direction of excitation may be used to judge the order of importance of the modes, and which modes will make a significant contribution to the dynamic response.

c. Equivalent added mass system. The added mass associated with the equivalent mass system discussed in paragraph 7-5*c* should be active in the horizontal direction, and inactive in the vertical direction. Added mass representing backfill or silt deposits against vertical or near vertical surfaces of the dam should also be active horizontally and inactive vertically. If the backfill is placed on the sloping face of the dam, the magnitude of the added mass acting vertically should be determined as described in paragraph 6-3*b*.

d. Hydrodynamic loading. The vertical component of ground motion causes hydrodynamic pressure waves to be generated from the reservoir bottom into the impounded water above. These pressure waves act horizontally against the vertical or near vertical face of the dam. In the composite finite element method, the equivalent mass system discussed in paragraph 7-5 accounts for the hydrodynamic reservoir effects caused by the horizontal component of ground motion, but it does not account for the effect of the hydrodynamic pressure waves generated by the vertical component of ground motion. To account for the effect of the pressure waves, a finite element-substructure model configuration is required as discussed in Chapter 8.

e. Combining component responses. The individual vertical and horizontal component dynamic responses are not in phase. They are independent maximum component responses that do not occur at the same point in time during the period of ground motion activity. Each pair of horizontal and vertical ground motion records representing a single earthquake event would have a unique phase relationship. Since the response spectrum method encompasses many possible ground motion events which make up the design earthquake, the maximum vertical and horizontal component responses are combined by a statistical method to produce a total dynamic response with reasonable probability of occurrence. It is recommended that the phasing of the two maximum component responses be treated as two unrelated random occurrences, and they be combined by the square root of the sum of the squares method (SRSS).

f. Conclusions. Under certain critical seismic conditions, the response to the vertical component of ground motion may be significant when compared to the response to the horizontal component; however the phase relationship will greatly moderate the vertical component contribution to the total response. On this basis, the vertical component of ground motion may be ignored in the preliminary design of new dams not subject to critical seismic conditions. The vertical component of ground motion shall be included for preliminary designs subject to critical seismic conditions, all final designs, and evaluation of existing dams.

Chapter 8
Dynamic Analysis Methods and Procedures

8-1. Attributes of Dynamic Analysis Methods

A dynamic analysis method is identified by four attributes: (1) material behavior, (2) design earthquake definition, (3) dimensional representation of project conditions, and (4) model configuration. The first two attributes have been discussed in preceding chapters. They are briefly summarized below, followed by a more detailed discussion of the latter two attributes.

a. Material behavior. This attribute defines material behavior as either (1) linear-elastic or (2) nonlinear. Associated with each of these two types of material behavior is a unique criterion for establishing acceptable response. Refer to paragraphs 2-2*d*, 2-2*e*, and 3-10.

b. Design earthquake definition. This attribute establishes which of two options will be used to specify the free field ground motion for the design earthquakes. The options are (1) design response spectra and (2) ground motion time-history records. Refer to Chapter 5 for details.

c. Dimensional representation of project conditions. This attribute defines whether project conditions will be represented in (1) two dimensions or (2) three dimensions. Project conditions refer to the geometry of the dam, the foundation, and the reservoir that have an affect on the seismic response. Examples of features governing which of these two options is appropriate include such things as layout of the dam axis, shape of the dam monoliths, foundation conditions, and orientation of potential fault slips if applicable.

(1) Two-dimensional (2-D) analysis. In the analysis of most gravity dams, it is assumed that the dam is composed of individual transverse vertical elements or cantilevers each of which carry loads to the foundation without transfer of load between adjacent elements. This assumption also applies to most RCC dams including dams with transverse joints that separate the dam into several monoliths, and dams with monolithic construction that contain no transverse joints. This assumption is usually valid, and stress analyses including the dynamic stress analysis phase can be based on 2-D representation of the dam cross-section. The design example provided in Appendix D presents a typical 2-D analysis. It demonstrates the most common procedure where a 2-D cross section of the structure is analyzed. However, most principles and procedures applying to the 2-D analysis also apply, or may be adapted to a 3-D analysis discussed below.

(2) Three-dimensional (3-D) analysis. Occasionally there are exceptions to the assumption justifying 2-D analysis. Dams in narrow canyons with a large enough ratio of height of the dam to distance between abutments may cause significant two-way distribution of stresses. Dams which are aligned on a curved axis may also allow significant transfer of stress into the abutments by arch action. Unusual shaped monoliths where there is substantial variation in the transverse cross section across the width of the monolith also may not be analyzed satisfactorily by 2-D methods. Another exception occurs when the trace of a potential fault slip is not parallel or nearly parallel to the dam axis. In this situation, a 2-D foundation fault displacement analysis will not adequately represent project conditions. All of these situations indicate the need for 3-D analysis if the response is to be determined to a reasonable degree of accuracy.

(a) Ground motion direction. The 3-D analysis introduces additional variables into the dynamic analysis. One important variable is determining the critical direction of the horizontal ground motion. This introduces a second horizontal component of ground motion into the dynamic analysis. The critical direction is defined by transforming the design earthquake ground motion into a pair of orthogonal components. Since no method exists to determine the critical direction directly, it usually becomes necessary to make some rough approximations.

(b) Simplified approach. This approach to determining the critical horizontal direction of ground motion is to select two orthogonal direction vectors (in the horizontal plane), and assume that the critical tensile stress at various locations on the dam will occur when the direction of ground motion is near one or the other vector. Since the accompanying orthogonal ground motion component is small, the stresses are assumed negligible and are neglected. Often the direction vectors are assumed to be the upstream-downstream direction, and the cross-stream direction. This approach requires performing separate, independent dynamic analyses for the two orthogonal ground motion directions.

(c) Conservative approach. Another more conservative approach accounts for both orthogonal components of ground motion. It is necessary to perform the two dynamic analyses described above, but the first analysis includes the full magnitude design earthquake ground motion component acting in an assumed direction with a fraction of the design earthquake ground motion acting orthogonally. The second analysis includes the fractional part of the ground motion acting in the assumed direction and the full magnitude ground motion acting orthogonally. The fractional part of the design earthquake ground motion is usually assumed to be 30 percent of the design earthquake ground motion. In a response spectrum analysis, stresses produced by the two horizontal components of ground motion are added directly to produce the resultant stress component for horizontal ground motion. This resultant stress component is then combined with the stress component produced by the vertical component of ground motion using SRSS.

(d) Complexity of analysis. A 3-D analysis requires considerably greater effort to create the 3-D model as compared to a 2-D model, and may require a main frame computer and a substantial amount of computer time to perform the analysis. It also produces a large amount of output to evaluate and interpret. However, the general purpose structural finite element programs are continuously being improved and are much more user oriented than they were in the past. They have refined graphics capabilities which help greatly in checking for errors in the computer model input, and in displaying the stress output. Also, specialized post-processors are being developed so that results can be evaluated much more efficiently. These advances greatly enhance the practicality of the 3-D analysis.

d. Model configuration. This attribute of the dynamic analysis method is dependent on the type of model used to represent the dam-foundation-reservoir system. The three types of models used for dynamic analysis of gravity dams are (1) the "standardized" model developed by Chopra and used in his Simplified Method of Analysis, (2) the finite element-substructure model, and (3) the composite finite element-equivalent mass system model.

(1) Standardized model. This type of model is used in Chopra's Simplified Method. It is based on standardizing certain parameters that define the dam-foundation-reservoir system. It recognizes the fact that these parameters have little variation within the range of geometry common to gravity dams. For example, the normalized fundamental mode shapes for six sample dam cross sections were studied and found to be almost identical. A standardized mode shape was then developed for use in the calculation procedure.

(a) Factors considered. In the latest version, the standardized model considers dam-foundation rock interaction, dam-reservoir effects, and reservoir bottom absorption. All of these factors are based on standard curves and formulae.

(b) Model limitations. The standardized model is the simplest of the three types of models. A computer is not required to formulate the model or even to perform the dynamic analysis. However, standardizing the mode shape, frequency, and other parameters makes this an approximate method limited strictly to the typical nonoverflow monolith shape.

(2) Finite element-substructure model. In this type of model, different techniques are used to represent the dam, foundation, and reservoir; however, by using common node points at the interfaces, a computer model is formulated that can be analyzed by conventional matrix methods.

(a) Dam. The dam is modeled as an assembly of discrete finite elements. Either solid quadrilateral plane stress or plane strain elements are used for a 2-D model.

(b) Foundation. The foundation is idealized as a viscoelastic half-plane. The elastic properties of the foundation are formulated into a substructure matrix using the theory of elasticity. This matrix is combined with the structural stiffness matrix developed from the finite element representation of the dam. The substructure matrix introduces the foundation stiffness to the equations associated with the degrees-of-freedom of the node points at the dam-foundation interface. There is no finite element model of the foundation. The dimensions of the structural stiffness matrix are set by the finite element model of the dam.

(c) Reservoir. The impounded water of the reservoir is idealized as a fluid domain of constant depth and infinite length. This can be interpreted as a series of subchannels of infinite length discretized to match the common upstream nodal points of the dam. The reservoir bottom absorption is modeled by adjusting the boundary condition at the reservoir bottom. This substructure representation of the reservoir produces more accurate hydrodynamic response to horizontal and vertical ground motion than does an equivalent mass system representation as described in paragraph 8-1*d*(3)(a).

(d) Specialized computer program. This type of model requires a specialized computer program to allow the foundation and the reservoir effects to be formulated in the manner described above. Also, the substructure method requires the foundation to be modeled as a uniform homogeneous material. Presently, a computer program is available which develops a 2-D finite element-substructure model for gravity dams. Refer to paragraph 8-2*b*.

(3) Composite finite element-equivalent mass system model. This method models both the dam and the foundation as an assembly of discrete finite elements. Either solid quadrilateral plane stress or plane strain elements are used for 2-D models or 3-D isoparametric solid elements are used for 3-D models. The foundation consists of a rectangular block with a width in the upstream-downstream direction about 3 times the base width of the dam at the foundation plane, and with a height about 1.5 times the height of the dam.

(a) Reservoir effects. The reservoir effects are modeled by developing an equivalent mass system which consists of adding mass to the finite element model to correctly alter the dynamic properties. The added mass is active in the direction normal to the vertical upstream face of the dam. This method also allows the reservoir bottom absorption characteristics to be incorporated into the analysis by using Chopra's standard hydrodynamic pressure function curves to determine the added mass. Although use of these curves in developing the equivalent mass system is only approximate, it has been shown to be reasonably accurate. Refer to paragraphs 7-5*c* and 7-5*d* and Appendix D for details.

(b) Boundary conditions. With this type of model, the earthquake ground motion is introduced at the rigid boundary. This boundary is along the sides and bottom of the rectangular foundation block rather than at the ground surface (dam-foundation interface) where the design earthquake ground motion is specified. To account for this, the foundation is assumed massless. Therefore, no wave propagation takes place in the massless foundation so the ground motion is transmitted to the dam-foundation interface without modification.

(c) Flexibility in modeling. The composite finite element model may be formulated to represent a variety of design conditions for both 2-D and 3-D models. For example, most any geometric shape may be accommodated, various zones of superior RCC mix may be incorporated in the dam model, and discontinuities such as fault zones or changes of deformation modulus in the foundation may also be included.

8-2. Comparison of Dynamic Analysis Methods

This section will describe the attributes associated with the most commonly used dynamic analysis methods, and the methods will be evaluated and compared.

a. Chopra's simplified method. This method uses the standardized model described in paragraph 8-1*d*(1). Other attributes include 2-D representation, linear-elastic material behavior, and response spectrum definition of the design earthquake. This method is not flexible because all of these attributes are fixed.

(1) Equivalent lateral force. The simplified method develops the maximum response to the first mode as a set of equivalent lateral forces. It also approximates the equivalent lateral forces associated with the higher vibration modes using a "static correction" method. The two sets of equivalent lateral forces are treated as statically applied distributed lateral loads. At present, response to a vertical component of ground motion is not possible with this type of model. Stresses may be hand calculated by beam theory treating the dam as a simple cantilever beam, or the static load may be applied to a finite element model of the dam to gain a more realistic stress distribution pattern.

(2) Advantages and limitations. The simplified method is easy to use and can be done without a

computer. However, it takes less time and effort to perform a simple 2-D analysis using a general purpose finite element program on a personal computer (PC) and the results of the finite element analysis will be more accurate. Also, comparative studies have indicated that as the flexibility of the foundation increases, the response calculated by the simplified method tends to diverge from the response determined by more refined methods, and the simplified method is not always conservative.

(3) Recommended use. Because of the limitations of the simplified method, it should be used only for preliminary design work as described in paragraph 8-4*a*. However, appropriate equations and design figures used in this method are helpful in checking the results from other more refined analyses and to prepare the computer input for these methods.

b. EAGD-84 Analysis Method. EAGD-84, A Computer Program for Earthquake Analysis of Concrete Gravity Dams (Fenves and Chopra 1984), is a specialized computer program that allows the foundation and the reservoir effects to be characterized by the substructure model described in paragraph 8-1*d*(2).

(1) Other attributes. Other attributes that define the EAGD-84 analysis method include 2-D representation, linear-elastic material behavior, and time-history ground motion definition of the design earthquake. All attributes of EAGD-84 are fixed and cannot be changed.

(2) Advantages and limitations. When compared to either a standardized model or a finite element-equivalent mass system model, the EAGD-84 substructure model is a better representation of the foundation and reservoir, as long as the project conditions properly fit the program requirements. Also, the time-history definition of ground motion is a level of refinement beyond response spectrum definition. Therefore, the EAGD-84 method is capable of producing the most accurate response, and the time-history response output provides additional information often needed to evaluate acceptable performance. The biggest disadvantage of EAGD-84 is the lack of attribute flexibility.

c. General purpose finite element program analysis methods. This comprises a number of methods each with a different combination of attributes, but all having the composite finite element-equivalent mass system model as a common attribute. These methods use any one of several proven general purpose finite element computer programs to perform the dynamic analysis. Examples are ANSYS, SAP6, GT-STRUDL, and STAAD III. The material behavior attribute for most of the general purpose programs is linear-elastic; however, some programs such as ANSYS and ADINA have nonlinear capability.

(1) Primary advantage. Attribute flexibility is the primary advantage of the general purpose finite element methods. Except for the common attribute mentioned above, design methods are possible which feature most of the other possible combinations of the remaining attributes. This allows the dynamic analysis phase to start with a simple method such as the 2-D, linear-elastic, response spectrum method. If the results of the simple analysis or the project conditions indicate the need of a more refined analysis, the procedure may transition conveniently into a more refined analysis by modifying or adding to the input to the same general purpose program.

(2) Other advantages. The general purpose finite element programs discussed above are large, comprehensive programs developed for main frame computers. In addition to these programs are several smaller general purpose finite element programs specifically developed for PC's. Since these desk-top PC's are now a standard item in most design offices, a considerable amount of the dynamic analysis phase may be completed without the need or expense of a large main frame computer.

8-3. Dynamic Analysis Procedure

The dynamic analysis procedure described hereafter is derived with the objective of arriving at a reasonable and economic design of a new dam, and evaluating the seismic resistance of existing dams using an analysis method with the simplest attributes possible. In general the procedure is to perform a dynamic stress analysis and evaluate the results to determine if the RCC dam response to the design earthquakes is acceptable. If not acceptable, the design of a new dam may be modified and reanalyzed, or a more refined analysis method may be employed when analyzing either new dams or existing dams.

a. Evaluating acceptable response. The response is judged acceptable for a linear-elastic analysis when the tensile stresses are within the established allowables and the analysis method provides a reasonably accurate or conservative representation of project conditions. Should the analysis method utilize an extremely simplified representation of project conditions, the response may not necessarily be conservative and will likely be of relatively low order of accuracy. However, the response may still be judged acceptable without pursuing more refined analyses on the basis that the tensile stresses are far enough below the established allowables to clearly infer that the response satisfies the requirements and criteria described above. Refer to paragraphs 2-2*e*, 2-2*f*, and 2-2*g* for information on allowable tensile stress criteria for various methods of analysis.

b. Modifying the design of a new dam. When the response from a dynamic stress analysis for a new dam is judged not acceptable, consideration shall be given to modifying the design, adjusting the computer model to reflect the modifications, and reanalyzing. Modifications include:

(1) Modify geometric configuration.

(2) Superior mixes. Use richer, higher strength superior RCC mixes in overstressed areas.

(3) Reducing aggregate size. Increase tensile strength by reducing the maximum size aggregate.

(4) Mortar bedding. Provide mortar bedding to increase tensile strength at lift joints.

(5) Zone boundaries. Adjust the zone boundaries of the superior RCC mixes to better fit the tensile stress pattern.

c. Refining the dynamic analysis methods. When the response from a dynamic stress analysis of an existing dam is judged not acceptable, the next step in the procedure shall be to reanalyze using an analysis method with more refined attributes. In contrast to this, there is no clearly defined point in the design procedure for new dams that indicates when the analysis method should be refined. The design conditions and results of the design procedure already completed must be evaluated to determine when it is appropriate to suspend the design modification process, and pursue a more refined analysis of the latest modified design. When the attributes of the dynamic analysis method are to be refined, it is recommended that the refinements be considered in the following order:

(1) 3-D representation. Consider refining the analysis from two to three dimensions when the accuracy of the response from a 2-D analysis cannot lead to a confident judgment that the response is acceptable.

(2) Time-history analysis. Consider defining the design earthquakes with appropriate ground motion time-history records, and performing a time-history analysis when additional insight into the structural behavior beyond that provided by the response spectrum analysis is needed. A time-history analysis yields additional information regarding the excursions of tensile stress cycles beyond the allowables and provides a better understanding of the response. This applies both to existing dams or to the design of a new dam when all practical and economical modifications to the design of a new dam have been exhausted.

(3) Nonlinear analysis. The analysis based on nonlinear material behavior represents the greatest possible refinement and it produces the most accurate results. However, it is also the most complex and the most costly. It requires time-history ground motion input, direct integration solution, a large main frame computer, specialized computer programs, and a considerable amount of computer time. As such, it is the last recourse in the attribute refining process. The nonlinear analysis should only be undertaken under the guidance of an expert in the field of fracture mechanics and finite element methods.

8-4. Preliminary Design of New Dams

Preliminary design includes engineering and design through the Feasibility Phase, or through the General Design Memorandum (GDM) phase if a GDM is prepared for the project.

a. Initial dynamic analysis. The initial dynamic stress analysis shall use the simplest analysis method which is identified by the following attributes: (1) linear-elastic material behavior, (2) 2-D representation, and (3) design response spectrum definition of the design earthquake. The analysis shall be performed using the cross-section of the critical transverse element of the dam which usually consists of a section of the nonoverflow monolith with the greatest height. The dam-foundation-reservoir system shall be represented by a composite finite element-equivalent mass system model for RCC dams subject to critical seismic design conditions. For other conditions, the dam-foundation-reservoir system may be characterized by either the standardized model using Chopra's simplified method or the composite finite element model described above.

b. Seismic and foundation investigations. Appropriate investigations of the regional tectonics and site seismicity shall be conducted at the preliminary design stage. When required, the site-specific design response spectra shall be developed in accordance with paragraph 5-5*c*. Preliminary dam site and reservoir geology investigations shall be conducted including exploratory corings and load testing to determine foundation conditions and deformation modulii.

c. Tensile strength. For preliminary design, the tensile strength may be taken from Figures 3-1 through 3-6 for the proposed basic RCC mix and for superior RCC mixes in the critical zones.

d. Satisfying criteria. The preliminary design procedure shall progress to the point where it becomes evident that the preliminary design will lead to a final design that fully satisfies established performance requirements and criteria.

8-5. Final Design of New Dams

The final design of an RCC dam shall result in a design that satisfies the provisions of this EP. The dynamic analysis phase for RCC dams under critical seismic design conditions shall be presented in an appropriate feature design memorandum.

a. Final design analysis method. The dynamic analysis method for the final design shall evolve from the simple initial method described in paragraph 8-4*a* to more refined methods of design conditions as warranted. RCC dams analyzed by Chopra's simplified method during the preliminary design phase shall be reanalyzed using a composite finite element-equivalent mass system model and general purpose finite element program in the final design.

b. Foundation and material investigations. The foundation conditions for the final design shall reflect the latest exploratory coring and other foundation and geology investigations. The final design shall be based on the RCC material properties obtained from tests on core samples taken from test fill placements made with the proposed design mixes.

8-6. Evaluating Existing Dams

The dynamic analysis procedure for evaluating existing dams is essentially the same as the combined preliminary design and final design procedures for a new dam, except modification of the design discussed in paragraph 8-3*b* does not apply to existing dams. As with the design of new dams, the dynamic analysis procedure shall utilize an analysis method with the simplest attributes possible to determine if the existing dam is capable of responding to the design earthquakes in an acceptable manner.

a. Material properties. Material properties of the RCC for an existing dam, including tensile strength, shall be obtained from tests on core samples taken directly from the dam.

b. Using available records. Exploratory coring logs, laboratory test data, and field geologic test results conducted during design and construction should be used for an existing dam and to provide information needed to model the foundation. Reservoir data should be used to determine the reservoir and tailwater elevations for earthquake load cases.

c. Special requirements and analysis methods. The regional tectonics and site geology and seismicity shall be investigated as required to develop a site-specific design response spectra in accordance with paragraph 5-5*c*. The initial analysis of an existing dam shall utilize a composite finite element-equivalent mass system model. Existing dams shall not be analyzed by Chopra's simplified method.