II. DESIGN AND SAFETY EVALUATION EARTHQUAKES

A. Terminology

The earthquake or earthquakes used for the design or safety evaluation analysis of a dam have been specified by a variety of terms such as the maximum credible earthquake (MCE), the maximum design earthquake (MDE), or the Safety Evaluation Earthquake (SEE). An operating basis earthquake (OBE) is often used in addition to the MCE, MDE, or SEE.

1. Maximum Credible Earthquake (MCE). The MCE is the largest earthquake magnitude that could occur along a recognized fault or within a particular seismotectonic province or source area under the current tectonic framework. The loading resulting from the MCE can and often is exceeded for Probabilistic Methods for high return period faults close in, such as the San Andreas Fault in California.

2. Maximum Design Earthquake (MDE) or Safety Evaluation Earthquake (SEE). This is the earthquake that produces the maximum level of ground motion for which a structure is to be designed or evaluated. The MDE or SEE may be set equal to the MCE or to a design earthquake less than the MCE, depending on the circumstances. Factors to consider in establishing the size of MDE or SEE are the hazard potential classification of the dam (FEMA 1998), criticality of the project function (water supply, recreation, flood control, etc.), and the turnaround time to restore the facility to operability. Guidance on selecting the MDE and SEE is contained in paragraph C.3. In general, the associated performance requirement for the MDE or SEE is that the project perform without catastrophic failure, such as uncontrolled release of a reservoir, although significant damage or economic loss may be tolerated. If the dam contains a critical water supply reservoir, the expected damage should be limited to allow the project to be restored to operation in an acceptable time frame.

3. Operating Basis Earthquake (OBE). The OBE is an earthquake that produces ground motions at the site that can reasonably be expected to occur within the service life of the project. The associated performance requirement is that the project function with little or no damage, and without interruption of function. The purpose of the OBE is to protect against economic losses from damage or loss of service. Therefore, the return period for the OBE may be based on economic considerations.

B. Conducting Seismotectonic Studies

1. Seismotectonic Studies. A seismotectonic study evaluates the geologic and seismologic history for the site. It defines the earthquake source or sources, determines the maximum earthquake potential for each source, develops magnitude-recurrence relationships, and provides information for use in the seismic attenuation studies. A seismotectonic study includes the identification and characterization of faults and other geologic structures that may be the sources of the earthquakes that could affect the site. This effort typically starts with literature searches in the fields of tectonics, seismology, and geology, and may include analyses of remote sensing imagery, geophysical data, and field mapping and exploration programs to verify and further evaluate documented or suspected tectonically active structures. Activities in seismotectonic studies typically include the following:

- Documenting and evaluating the type, location, areal extent, displacement, and age of displacement of faults in the area of interest. Evaluating the age of Quaternary surfaces and deposits may also be required in areas where active, concealed faults may be present.

- Characterizing located faults by sense of movement, degree and age of activity, recurrence, and slip rate (Page 1997). Potential for surface faulting in the immediate vicinity of the dam should also be addressed and may require detailed evaluation.

- Characterizing other seismotectonic structures that are not correlative with surface features using geophysical techniques, regional geologic data, and historical seismicity characteristics.

- Analyzing recent earthquakes or data from micro-earthquake monitoring to gain information on current stress orientations.

- Establishing relationships between identified faults and other seismotectonic features and the historical seismicity by (1) integrating the data for identified faults and other structures with the seismicity data to develop event/source correlations; and (2) developing and evaluating conceptual models to explain significant earthquake events that do not correlate with identified faults or other seismic sources or geologic structures.

- Establishing earthquake magnitude and distance pairs that would be expected to produce the largest ground motions at the site.

- Developing magnitude-recurrence relationships.

2. Factors to Consider. The factors enumerated below, ranging from regional to site-specific, should be considered in conducting a seismotectonic study. Bureau of Reclamation (Reclamation 1993), U.S. Committee on Large Dams (USCOLD 1999), and U.S. Geological Survey (Hays 1980) are recommended as source documents for additional information on this subject. Locating the project area in published information of source zones and seismic hazard areas (FEMA 1997) may indicate the level of anticipated seismicity as well as provide a starting point for a more comprehensive evaluation of seismic activity.

a. Regional Tectonic Setting. The regional geology and late Cenozoic tectonic history study area should cover, at a minimum, a 100 km radius around the site, but may be extended to include any major fault or structure that could affect the ground motions at the site. This study should include such items as the following:

- Identification of the physiographic and seismotectonic province(s) relevant to the project.

- Geologic history of the project area.

- Description of the regional geologic formations, rock types, and soil deposits to evaluate seismic energy transmission, effects, and duration of ground motions.

- Location of major regional geological structural features, including folds and fracture or joint patterns.

- Interpretation of the regional tectonic mechanism(s) and associated type(s) of faulting.

- Location and description of faults and shear zones and assessment of the capability of faults to generate earthquakes or to be displaced by earthquakes.

- Estimation of the relative degrees of fault activity for each of the faults of concern to the study area.

b. Historic Seismicity. Compilation of historical earthquake data helps to identify the seismicity patterns of an area and provides a basis for estimating a lower bound of the severity of possible future earthquake motion at the site considered. The lack of historic earthquakes, however, does not necessarily imply that the area considered is aseismic. Historical seismological information exists primarily within governmental and academic references and the media record. The National Earthquake Information Center of the U.S. Geological Survey, California Division of Mines & Geology, and the Engineering Research and Development Center of the U.S. Army Corps of Engineers compile source and magnitude information for earthquake events. The collection and evaluation of historic seismicity data involves the following activities:

- Defining the limits of the historical data search area (considering boundaries of seismotectonic sources and provinces with potential significance to the site).

- Conducting a search of the historic seismological records to determine the location, date, and characteristics (e.g., earthquake source parameters and wave propagation effects) of seismic events within the area of interest. The completeness of the historic record, the location accuracy, and the independence of events are considered in a thorough evaluation.

- Developing frequency, magnitude, and recurrence relationships considering the relationships between regional province-wide recurrence and recurrence for individual faults or sources. A probabilistic estimate of the peak acceleration and, if required, the peak velocity ground motion and/or spectral accelerations expected to occur at the site within a given time period may be based on the historic record of seismicity and appropriate attenuation relationships.

- Displaying the seismic history information using tabulations, graphs, and maps. Historical records of any surface effects of past earthquake events are included in the Seismotectonic study.

c. Local or Site Geology. Site-specific geologic information is required to determine the potential for surface rupture and to evaluate local site response effects on earthquake ground motions. This information may be obtained through the evaluation of published geologic reports, field observations, and detailed site-specific investigations that may include geologic mapping, trenching, shear wave velocity measurements, drilling, material sampling, and laboratory testing. The site geology investigation could cover the following items:

- The geotechnical character, depositional history, orientation, lateral extent, and thickness of soil units beneath the site and on adjacent slopes.

- The character, lateral extent, and thickness of rock units at the site area.

- The structural geology of the site to include rock unit attitudes, faults and joint systems, folding, and intrusive bodies.

- The age and activity level of faults in the dam and reservoir area.

- The geohydrology of the site area to include water table conditions, soil and rock transmissivity coefficients, and recharge areas.

- The existing and potential ground failure and subsidence, including rock and soil stability, dispersive soil conditions, and soil units exhibiting characteristics for potential liquefaction.

- The potential for seiche action by examination of fault location, reservoir shape, topography, and slope stability conditions.

d. Seismic Attenuation. The seismotectonic study can provide support to determine ground motions at the site and the attenuation of seismic ground motion with distance. Because of the inhomogeneity of areal and cross-sectional geology, the assessment of local or regional attenuation is difficult, and to a large extent is based upon historical, empirical, or theoretical data. The advancements in attenuation assessment take place through the development and testing of better mathematical models and the incorporation of new data. The seismotectonic study may require the following information for use in seismic attenuation studies:

- Instances where ground motion data are available, to develop site-specific attenuation relationships or scattering functions.

- Isoseismal maps constructed from historical earthquake data provide an indirect measure of ground motion attenuation.

The most common approach is to use published attenuation relationships applicable to the general site conditions and type of expected faulting. Recent attenuation relationships are summarized in a series of papers published by the Seismological Society of America (SSA 1997). Attenuation relationships are frequently modified and refined. As a result, the most recent published research should be consulted to determine the appropriate attenuation models.

e. Reservoir-Induced Seismicity. USCOLD published a report (USCOLD 1997) examining the topic of reservoir-triggered seismicity. From a worldwide standpoint, only a small number of reservoirs impounded by large dams have triggered known seismic activity. It is generally accepted that reservoir filling will not cause damaging earthquakes where they would not otherwise occur; however, reservoir filling can cause earthquakes to occur more frequently than they would otherwise occur. Thus, the MCE is considered to be the same, with or without reservoir filling, but reservoir filling can alter the magnitude-recurrence relationship, at least temporarily. Seismic activity associated with reservoir filling has been reported at more than 40 reservoirs. Included in the USCOLD report is a list of the more frequently cited cases.

C. Selecting Maximum Design and Safety Evaluation Earthquakes

1. General. There are two general approaches for conducting site-specific analyses for determining site ground motions (Reiter 1991). Either or both of these approaches may be used for a given site. The selection would be based upon the framework of the engineering assessments being conducted. It is also important to understand that in some cases different sources will control the design for different structural elements depending on the structures response. For example, the piers on a spillway structure may be controlled by nearby smaller events whereas a more massive structure, such as a high embankment dam subject to liquefaction, may be controlled by larger more distant sources. The two approaches are discussed below.

a. Deterministic Seismic Hazard Analysis (DSHA). The DSHA approach uses the known seismic sources near the site and available historical seismic and geological data to generate discrete single-valued events or models of ground motion at the site. Typically, one or more earthquakes that will produce the greatest ground motion at the site are specified by magnitude and location with respect to the site. Usually, the earthquakes are assumed to occur on the portion of the source closest to the site. The site ground motion parameters (peak ground acceleration/velocity, spectrum intensities, duration of strong shaking, etc.) are estimated deterministically for each source, given the magnitude, source-to-site distance, and site conditions, using an attenuation relationship and/or theoretical models.

b. Probabilistic Seismic Hazard Analysis (PSHA). The PSHA approach uses the elements of the DSHA and adds an assessment of the likelihood that ground motions of a given magnitude would occur. The probability or frequency of occurrence of different magnitude earthquakes on each significant seismic source and inherent uncertainties are directly accounted for in the analysis. The possible occurrence of each magnitude earthquake at any part of a source (including the closest location to the site) is directly incorporated in a PSHA. The results of a PSHA are used to select the design earthquake ground motion parameters based on the probability of exceeding a given parameter level during the service life of the structure or for a given return period. Results from the PSHA approach can also be used to identify which combinations of magnitudes and distance (or specific seismic sources) are the largest contributor to hazard. Identification of these controlling earthquakes can then be used in scenario or DSHA analyses. Several of the Federal agencies are currently developing guidelines on procedures to follow when performing a PSHA study.

2. Determining Maximum Credible Earthquakes. The MCE for each potential earthquake source, judged to have a significant influence on the site, is established by a DSHA based on the results of a seismotectonic study (site-specific investigations and/or literature review). The MCE for each seismotectonic structure or source area within the region examined is defined preferably by magnitude, but in some cases in terms of epicentral Modified Mercalli Intensity, distance, and focal depth. Earthquake recurrence relationships (i.e., the frequency of occurrence of earthquakes of different sizes if appropriate for the fault) should also be established for the significant seismic sources. For source zones consisting of random seismicity, an MCE can be determined by finding the magnitude and distance that best matches the equal hazard response spectrum from a PSHA in the design earthquake frequency range appropriate for the structure. Judgments on activity of each potential fault source are generally based on recency of the last movement. For high-hazard potential dams, movement of faults within the range of 35,000 to 100,000 years BP is considered recent enough to warrant an "active" or "capable" classification. All of the above MCE assessments for the various earthquake sources are candidates for one or more controlling MCEs at the site. It is also important to look at earthquakes that have a long duration but not necessarily the highest peak acceleration at the dam site. For embankment dams and foundations subject to liquefaction, this longer duration earthquake may be the controlling event if it triggers liquefaction of the embankment/foundation materials. Other appurtenant structures should be evaluated to determine if a higher magnitude distant earthquake is critical to the overall stability of the structure.

3. Selecting Maximum Design Earthquakes or Safety Evaluation Earthquakes

a. Criteria. The final selection of the MDE considers whether or not the dam must be capable of resisting the controlling MCE without catastrophic failure, such as uncontrolled release of a reservoir, although severe damage or economic loss may occur. For high-hazard potential dams, the MDE usually is equated with the controlling MCE. However, for low or significant-hazard dams, the MDE may be determined based on faults active in Holocene time, or according to other agency specified criteria.

b. Combining Seismic Hazard Analyses. The combined application of deterministic and probabilistic seismic hazard analyses is an effective approach for determining the MDE ground motions. The probabilistic analysis allows the probabilities or return periods for exceeding different levels of site ground motion to be evaluated. This information can then be used to complement the deterministic analysis. For example, the probabilistic results can aid in deciding whether mean, or mean-plus-one standard-deviation, or greater estimates of ground motion would be justified for the MDE from the deterministic ground motion analysis to achieve an acceptably low probability of exceedance or long expected return period for the ground motion.

4. Selecting Operating Basis Earthquakes. The second level of design earthquake, the OBE, represents the maximum level of ground shaking that corresponds to the desired level of protection for the project from an operations standpoint. Factors to be considered are earthquake-induced structural and mechanical damage and loss of service during the project's service life or remaining service life for existing dams. The OBE is generally determined from a PSHA.

III. CHARACTERIZING SITE GROUND MOTIONS

A. General

The next requirement for the seismic evaluation or design of structures is to establish an adequate representation of the earthquake ground motions to which the structure might be subjected. Earthquake ground motions are characterized for design, analysis, or evaluation in one or more of the following ways: by a set of ground motion parameters (e.g., peak ground acceleration, peak ground velocity, peak ground displacement, and duration of ground shaking); by response spectral values; or by one or more accelerograms. The spectral characteristics of the ground motion are normally presented in the form of response spectra. The response spectrum is very useful for the design and analysis of structures because it directly provides the structure dependent response to the design earthquake excitation. In practice, time-history representation of the seismic input is generally presented in terms of the ground accelerations (accelerograms), although velocity and displacement records may also be developed. Time-history records are used to perform linear or nonlinear time-history analyses.

B. Ground Motion Parameters

1. General. Peak ground acceleration, velocity, and displacement are estimated using seismic history data, site-specific observations, and/or magnitude-distance attenuation relations if specific earthquake sources have been identified.

2. Attenuation Relationships. Values of peak ground acceleration, velocity, displacement, spectral acceleration, and duration at various distances from an earthquake source have been compiled from past earthquakes. These values are used to develop relationships between distance from the source of energy release and earthquake magnitude or MM intensity for various subsurface conditions (bedrock, stiff soil, and soft sites).

3. Components. Previous practice commonly assumed that accelerations in the two horizontal directions are equal or used the acceleration pairs from actual records with common scaling. The maximum vertical value has generally been taken as one-half to two-thirds or more of the maximum horizontal value. Current practice is to use attenuation relations for vertical acceleration which are now available. Ratios of vertical to horizontal ground motions are strongly dependent on earthquake source-to-site distance, local site conditions, and on the particular period range of interest. Detailed fault specific ground motion studies take into account directional aspects of the rupture and thus provide different motions for each of the horizontal components. Near-fault ground motions require special considerations. Rupture directivity on near-fault ground motions can significantly alter the amplitude and duration of the ground motion. The component of ground motion normal to the fault plane is often more severe than the component parallel to the fault plane. Peak vertical acceleration may also equal or exceed the peak horizontal acceleration. The effects of directivity in near fault ground motions are discussed in detail by Somerville (et al 1997 & 1998).

4. Duration. Duration of shaking has been identified as one of the most important parameters of ground motion that causes damage to a structure. Some earthquakes have produced short, high-frequency accelerograms, but did not cause structural damage, even though the peak ground accelerations were very large. Other earthquakes where damage occurred had lower peak ground accelerations but had a long duration of ground shaking. Duration plays an important role in determining if liquefaction is triggered, determining deformations, and in conducting nonlinear dynamic analyses. Specific definitions of duration are discussed in the glossary.

C. Elastic Response Spectra

Elastic response spectra can be defined by using standard or site-specific procedures. Elastic response spectra represent maximum responses of a series of single-degree-of-freedom systems of different natural periods to a given ground-motion excitation (Newmark and Rosenblueth1971). The response spectrum amplifications vary with the value of damping. Typically, they are plotted at a damping ratio of 5 to 10 percent. Damping ratios of up to 10 percent should generally only be allowed in dams showing energy dissipation through joint opening and tension cracking. Response spectra are significantly influenced by earthquake time history, source-to-site distance, and site conditions. These factors must be considered in developing site-specific design response spectra.

1. Standard or Normalized Response Spectra. The standard response spectra commonly used is Newmark-Hall Spectrum (Newmark and Hall 1982). The spectra is developed using the peak or effective ground motion parameters in conjunction with a standard spectral shape that ignores the influence of earthquake magnitude and distance and site-type on the shape of the spectra. For critical structures located in regions with moderate or high seismic activity, or at sites close to the epicentral region or founded on soft soils, the site-specific spectra described in Paragraph 3 below are used in lieu of standard response spectra.

2. Response Spectra from U.S. Geological Survey (USGS) Probabilistic Maps. USGS probabilistic maps of the United States are now available to construct preliminary design response spectra. These maps give ground acceleration and spectral response acceleration with 10 percent, 5 percent, and 2 percent probabilities of exceedance in 50 years. These probabilities of exceedance correspond to return periods of approximately 500, 1000, and 2500 years, respectively. The use of spectral ordinates to construct design response spectra directly is a marked improvement to using peak ground accelerations and standard scaled spectral shapes, as discussed in paragraph 1 above.

3. Site-Specific Response Spectra. Site-specific procedures are used to produce response spectra that correspond closely with those expected on the basis of the seismological and geological conditions at the site. These procedures use either the deterministic or probabilistic method to develop site-specific spectra.

a. Deterministic Approach. Deterministic estimates of response spectra can be obtained either by 1) establishing the accelerograms to be used for the site and directly computing their response spectra; 2) estimating the response spectrum directly; or 3) anchoring a response spectral shape to the estimated peak ground acceleration. If approach 2 is used, estimating the response spectrum directly, this involves one or more of the following procedures: using response spectral attenuation relationships; performing statistical analyses of response spectra of a suite of ground motion records; and/or applying theoretical (numerical) ground motion modeling (USACE 1999). Approach 3, anchoring a response spectral shape to the estimated peak ground acceleration, is the least desirable approach for high consequences dams.

Response spectra developed from a deterministic seismic hazard analysis are normally provided as median and median-plus-one standard deviation (84^{th} percentile) spectra. The decision on the spectrum to use for design should be based on site-specific and agency criteria. In particular, for sites with either high rates of nearby seismicity or nearby high slip rate fault(s), using mean-plus-one standard deviation spectra may be prudent.

b. Probabilistic Approach. Site ground motions are estimated for selected values of the probability of ground motion exceedance in a design time period. These values can also be represented by values of the annual frequency or return period of ground motion exceedance. A probabilistic ground motion assessment incorporates the probability or frequency of occurrence of earthquakes of different magnitudes from the various seismic sources, the uncertainty of the earthquake locations of the sources, and the ground motion attenuation including its uncertainty. In a probabilistic approach, response spectral attenuation relationships for each of several periods of vibration are used. By drawing a curve that connects the response spectral values to the same probability of exceedance, a response spectrum having an equal probability of exceedance at each period of vibration (termed equal or uniform hazard spectrum) is obtained. The resulting spectrum is preferred because of the direct incorporation into the analysis of the influence of different earthquake magnitudes and distances and source-site geometry on the results for each period of vibration.

Response spectra developed from a probabilistic seismic hazard analysis are provided, generally, as mean equal hazard spectra. A comprehensive PSHA incorporates not only the inherent randomness, or variability, of earthquake generation and seismic wave propagation, but also the uncertainty associated with the choice of particular models and model parameters for characterizing seismic sources and estimating ground motions. Accordingly, no additional conservatism is normally added to mean equal hazard spectra from a comprehensive PSHA. The degree of conservatism is explicitly defined by the choice of annual exceedance probability.

c. Site-Specific Characteristics. The site-specific characteristics to be considered in developing response spectra are the foundation conditions (soil or rock, deep or shallow soil, rock type, basin configuration, and impedence contrast), magnitude of the design earthquake, distance of the site from the earthquake, attenuation characteristics of the earthquake to the site, and the source mechanisms of the earthquake, including identifying if the site fault is on the hanging wall versus footwall (Sommerville et al 1997, Abrahamson and Silva 1997), depth of faulting, and other factors.

D. Time Histories of Ground Motions

1. General. When acceleration time histories (also referred to as accelerograms) of ground motions are required for the dynamic analysis of a structure, they should be developed to be consistent with the design response spectrum, as well as have an appropriate strong motion duration for the particular design earthquake. In addition, whenever possible, the acceleration time histories should be representative of the design or safety evaluation earthquake in all the following aspects: earthquake magnitude, distance from source-to-site, fault rupture mechanisms (fault type, focal depth), transmission path properties, and regional and geological conditions. Since it is not always possible to find empirical records that satisfy all of the above criteria, it is often necessary to modify existing records or develop synthetic records that meet most of these requirements.

2. Approaches to Developing Time Histories. There are two general approaches to developing acceleration time histories: selecting a suite of recorded motions and synthetically developing or modifying one or more motions. These approaches are discussed below. For either approach, when modeling near-source earthquake ground motions (i.e., minimum source-site distance less than 10 km), it is desirable that the motions include a strong intermediate- to long-period pulse to model this particular characteristic of ground motion often observed in the near field and generally accepted to be responsible for significant damage. Of specific importance at distances less than 10 km are the effects of directivity in developing fault normal and fault parallel components (Somerville et al 1997).

a. Selecting Recorded Motions

(1) Typically, in selecting recorded motions, it is necessary to select a suite of time histories (typically 3 or more) such that, in aggregate, valleys of individual spectra that fall below the design (or "target") response spectrum are compensated by peaks of other spectra and the exceedance of the design response spectrum by individual spectral peaks is not excessive (preferably at least within the bandwidth of interest for structures specific analysis). For nonlinear analyses, it is desirable to have additional time histories because of the importance of phasing (pulse sequencing) to nonlinear response. In the past, when using selected recorded motions, simple scaling of acceleration time histories was frequently performed to enhance spectral fit. However, scaling should be done with caution. The ramifications of significant scaling of acceleration time-histories on velocity, displacement, and energy can be profound.

(2) The advantage of selecting recorded motions is that each accelerogram is an actual recording; thus, the structure is analyzed for motions that are presumably most representative of what the structure could experience. The disadvantages are: multiple dynamic analyses are needed for the suite of accelerograms selected; although a suite of accelerograms is selected, there will typically be some exceedances of the smooth design spectrum by individual spectrum peaks; and although a reasonably good spectral fit may be achieved for one horizontal component, when the same simple scaling factors are applied to the other horizontal components and the vertical components for the records selected, the spectral fit is usually not as good for the other components.

b. Synthetically Developing or Modifying Motions

(1) Techniques. A number of techniques and computer programs have been developed to either completely synthesize an accelerogram or modify a recorded accelerogram so that the response spectrum of the resultant waveform closely matches the design or target spectrum. Recent advances have used either (a) frequency-domain techniques with an amplitude spectrum based upon band-limited white noise and a simple, idealized source spectrum combined with the phase spectra of an existing record; or (b) kinematic models that produce three components of motion using complex source and propagation characteristics. Such motions have the character of recorded motions since the modeling procedures are intended to simulate the earthquake rupture and wave propagation process. Recent research suggests dynamic and three-dimensional models may be important in estimating engineering ground motions in the future.

(2) Comments. The natural appearance and duration of strong motion can be maintained using these techniques. A good fit to the target spectrum may or may not be possible with a single component of motion. However, for non-linear applications, it is particularly desirable to have multiple accelerograms because different accelerograms may have different phasing (pulse sequencing) characteristics of importance to nonlinear response yet have essentially identical response spectra. For near-field situations, the characteristics of the motions should reproduce the coherent velocity pulses ("fling") commonly observed in near-field recordings.

(3) Advantages and Disadvantages. The advantages of synthetic techniques for developing time-histories are: the natural appearance and strong motion duration can be maintained in the accelerograms; three component motions (two horizontal and one vertical) each providing a good spectral match can be developed; and the process is relatively efficient. The disadvantage is that the motions are not "real" motions. Real motions generally do not exhibit smooth spectra. Although a good fit to a design spectrum can be attained with a single accelerogram, it may be desirable to fit the spectrum using more than one accelerogram. Such motions have the character of recorded motions since the modeling procedures are intended to simulate the earthquake rupture and wave propagation process.

3. Application. Ground motion parameters should be specified in a manner that is consistent with the analyses to be performed. Where ground motions are specified at one location (e.g., a rock outcrop) and are used in the analysis at a different location (e.g., at the base of a soil layer), the motions need to be adjusted accordingly. Where magnitude and distance are used in empirical procedures, it is important to verify that distance-attenuation definitions in the procedure are consistent with those inferred for the site of interest.

IV. SEISMIC ANALYSES

A. Type and Extent of Analyses

1. Consistency. The extent and type of analysis required for the seismic design or evaluation of a dam depends on the hazard potential classification (FEMA 1998), criticality of project function, level of seismic loading, the site conditions, type and height of dam, construction methods, as-built as well as current material properties, and engineering judgment. Consistency should be maintained between the level of analysis and the level of effort given to the development of seismotectonic data, the ground motion parameters, and the site investigation. For example, a highly refined structural analysis based on an assumed earthquake loading is not reasonable in most cases. Likewise, a highly refined structural analysis should use site-specific tested material properties and not assumed values.

2. Progressive Analyses. In general, it is the most cost-effective for seismic analyses to begin with the simplest conservative method appropriate to the problem. If the structure is judged able to resist the earthquake loading within certain safety margins from the initial analysis, then further analysis should not be necessary. If further studies are needed, they would be progressively more detailed and the structure designed or evaluated accordingly. Regardless of the method of analysis, the final evaluation of the seismic safety of the dam should be based on engineering judgment and past experience, not just on the numerical results of the analyses.

B. Concrete Dams

1. Safety Concerns

a. Instability. The most important safety concern of concrete dams subjected to earthquakes is excessive cracking, which can lead to potential instability from sliding or overturning. Sliding can be on an existing plane of weakness in the dam or foundation or along planes of weakness formed by excessive cracking of the concrete above or at the foundation-dam interface. Although no concrete dam has failed as a result of earthquake loadings, failure modes can be postulated and tested on shake tables (Payne 2002). For concrete dams, sliding instability is possible due to an earthquake-induced vibratory motion on a plane of weakness at, above, or below the foundation-dam interface. For an arch dam, sliding instability is more likely to occur by failure of the abutment support because the arching (shell) effect provides additional resistance to sliding within the dam. In general, instability of gravity and arch dams caused by excessive cracking of the concrete is most likely to occur in the upper half of the dam. Buttress dams also are particularly vulnerable to cross-valley shear motions that can result in tipping of the buttresses and loss of support for the reinforced concrete slab.

b. Importance of Foundation. Of the two possible types of instability discussed above, historical experience shows that foundation (abutment) induced failure is the chief source of concern for concrete dams. In contrast to the dam itself, the supporting medium consists of natural materials of varying composition, irregular joints, and planes of weakness. The strength of this medium is generally estimated from exploratory borings and tests on only a small fraction of the material present. Key zones of weakness are critical and often difficult to detect.

c. Field Performance. No major concrete dam is known to have failed due to earthquake-induced ground motion, although some have experienced strong ground motion and some damage. The field performance of concrete dams during earthquakes is further discussed in Section V.

2. Defensive Design Measures. The application of defensive design measures when designing new dams is the most dependable approach to alleviate safety concerns. Defensive design measures for concrete dams include the following:

a. Sufficient foundation and abutment exploration, material testing, and strengthening to assure foundation and abutment integrity. The importance of foundation and abutment integrity cannot be overemphasized. Adequate drainage is usually the first line of defense against foundation instability, in part because it is the most economical.

b. Use of the best geometric design and structural detailing appropriate to the structure. The structural configuration should have minimum geometric irregularities and gradual variations in structural stiffness. Examples of good geometric designs are curved transitions, minimal mass at the crest, gradual changes in arch and cantilever stiffness at the top half of arch dams, and a downstream face slab for buttress dams. Continuous load paths, load path redundancy, and ductile behavior are important safeguards to ensure that structures loaded past their elastic limit will continue to perform adequately and will function after extensive cracking. Any necessary structural irregularities should be properly detailed to account for the localized effects of stress concentrations.

c. Effective quality control during construction to ensure adequate foundation preparation, strength of the concrete and appropriate cleaning and preparation of lift joints, and placement of the reinforcement when used. For existing dams where the seismic loading has substantially increased and engineering analysis demonstrates poor performance of the structure, remediation schemes will depend on site-specific conditions. However, general types of seismic remediation of existing concrete structures where the dam or foundation has been determined to be inadequate are post-tensioned anchors, additional mass concrete, buttressing, and drainage.

3. Structural Modeling

a. General. Structural modeling should be undertaken by an engineer (seismic structural analyst) knowledgeable in the basic theory of structural dynamics, materials, seismology, and finite-element structural analysis. The structural engineer, materials engineer, geologist, geotechnical engineer, and seismologist should all work closely together to understand the level of study and the level of certainty required from each discipline. The team should develop failure modes and describe the level of certainty for each failure mode. Concrete dams are very complex structures. The dynamic interactions among various components of the structure must be considered when developing the failure modes and the analysis models. In particular, interaction effects of the structure with its foundation and the impounded, surrounding, or contained water should be included in the analysis. However, the structural model selected should be consistent with the level of refinement used in specifying the earthquake ground motion, and should begin with the simplest model that can provide the required information and level of certainty for evaluation. Information required for modeling the rock foundation and any backfill soils or soil foundation should be provided by the project team's geotechnical engineer and/or engineering geologist. The geotechnical engineer must know how the information will be used in the structural analysis. For example, if the analysis uses an approach that incorporates mass in the foundation, the dam response can be highly sensitive to the modulus values being used. The material engineer needs to know where in the dam the concrete properties are most critical for the structural engineer and what properties are desired. If lift line strength is critical, a core extraction and testing program should be devised to try and obtain the best possible in situ values possible. For dynamic stability, the dynamic tensile strength may have to be measured along with the static tensile strength. The direct tensile strength may be desired along with the splitting tensile strength.

b. Types of Models. The development of an appropriate structural model requires consideration of the geometry, stiffness, damping, and mass distributions of the structure, all of which affect its dynamic characteristics. Engineering judgment and knowledge of dynamics of structures are required to develop a satisfactory model that is both simple and representative of the most important dynamic behavior of the structure. Depending on its nature and the level of complexity, a hydraulic structure may be represented by a planar or two-dimensional model whose deformations are restricted in a plane, or by a more extensive three-dimensional model to account for three-dimensional behavior.

4. Methods of Analysis

a. General. For a concrete dam on a rock foundation with no major known structural deficiencies, such as significant cracks in the structure, major weak joints, or adversely oriented discontinuities in the foundation, stability during an earthquake loading based on previous case histories and/or analyses should be satisfactory if all the following conditions are satisfied:

- The dam is well-constructed (quality concrete and lift joints) and is in good condition.
- Peak bedrock accelerations are 0.2g or less.
- The resultant location for a gravity dam under static conditions is within the middle 1/3 of the base and the factor of safety against sliding is acceptable for static conditions.

If these conditions are not satisfied, the methods of dynamic analysis currently being applied for various levels of studies and types of dams, in the order of complexity, are linear-elastic response spectrum, linear-elastic time-history, and non-linear time-history methods.

b. Dynamic Analysis Methods. Dynamic analysis methods are those that determine the structural response based on the dynamic characteristics of the structure and the earthquake ground motions established for the site. The dynamic method most commonly used is the linear-elastic modal superposition analysis method. This method is based on the fact that, for certain forms of damping that are reasonable models for many structures, the response in each natural mode of vibration can be computed independently of the others and the modal responses can be combined to determine the total response. Either a response spectrum or an acceleration-time record can be used with the analysis technique. Time-history responses can also be calculated using step-by-step direct integration.

(1) Response Spectrum Method

(a) Description. In this method, the maximum response in each mode of vibration is directly computed from the design response spectrum and the dynamic characteristics of the structure. These modal responses are then combined to obtain estimates (but not the exact value) of the maximum modal response. Adding the absolute values of each mode gives an upper limit for the total response, while using the square root of the sum of the squares yields a more probable total response. Either a simple beam analysis (P/A +/- Mc/I) or finite element analysis can be used to compute the stresses. Dynamic stresses are combined with the static stress to obtain the total stress. The response spectrum method does not account for stress variation with time. See U. S. Army Corps of Engineers (USACE 1999) for guidance on the use of the response spectrum method for concrete dams.

(b) Evaluation of Response. In most cases, when the dynamic response of the structure is entirely within the linear elastic range, the response spectrum analysis will suffice. However, in the case of earthquakes for which the calculated stresses for mass concrete structures may approach or exceed the tensile strength of the concrete, a time-history linear dynamic analysis provides valuable information for the approximation of the potential damage, or the expected inelastic response behavior that may occur and the potential need for a nonlinear analysis.

(2) Time-History Method

(a) Linear-elastic Analysis. In this method, the response of the dam is calculated for the entire duration of an acceleration time-history, starting with initial static conditions and computing the response at the end of each time interval. The stresses and/or deflections of each mode are added together for each time interval to yield the total time-history response. The dynamic responses are added algebraically to the static responses. Stress histories are developed to show the maximums, the number and duration of excursions beyond the tension or compression limits, and the areal distribution of stresses in the dam at particular times during the earthquake.

(b) Evaluation of Response. For major earthquakes occurring close to some concrete dams, it is probable that the elastic capacity of the mass concrete would be exceeded, and some damage or yielding could occur. For these cases, the prediction of the actual response and estimation of the expected damage and inelastic behavior of the dam can only be judged in a linear analysis. Evaluating actual damage and failure requires a more involved nonlinear analysis. However, linear analysis can still be very valuable for a preliminary assessment of the damage and the level of post-elastic response, and can aid in the decision of whether a nonlinear analysis should be performed. As part of this evaluation, the results of a linear analysis for hydraulic structures should be examined in a systematic manner to identify the extent of over-stressed regions at any particular point in time. This will aid in the production of plots to show time-histories of stresses and other response quantities of interest, and to establish statistics on the number of stress cycles exceeding the allowable values and the corresponding excursions of these stress cycles beyond the specified limits. Minor, local damages, indicated from an elastic analysis, would have little effect on the overall integrity of the structure and can still be evaluated by proper interpretation of the results of linear analyses. An alternative evaluation technique that has been used by some agencies to evaluate a structure indicating distress using a linear elastic analysis is to assume cracking along the total width of the plane being investigated and evaluate structural stability in the post earthquake (static) condition. Stability during the earthquake would also have to be checked or assured if cracking took place prior to the conclusion of the event.

(c) Nonlinear Analysis. A complete nonlinear analysis should take into account all sources of nonlinearity that contribute significantly to the nonlinear behavior. The damage caused by earthquake shaking is normally associated with significant loss in the structural stiffness, a result of concrete cracking, yielding of steel, opening of contraction joints, slippage across the construction joints or cracking planes, and nonlinear material behavior. Additional sources of nonlinearity arise from the nonlinear response of the foundation supporting the structure, as well as the separation of the structure and the foundation at the contact surface. A complete and reliable nonlinear dynamic analysis that includes tensile cracking of concrete, yielding of reinforcements, opening of joints, and foundation/abutment displacements is becoming more practical. Having a complete and reliable nonlinear analysis for the seismic safety evaluation of dams depends on continuing developments in the following areas:

- Definition of spatially varying seismic input.

- Energy absorption factors for reservoir sides and bottom and at infinite boundaries at the reservoir and foundation extents of the models.

- Boundary identification and specification of significant nonlinear mechanisms (joint opening, tensile cracking, steel yielding, nonlinear material behavior under dynamic loads, etc.).

- Development of idealized models representing the nonlinear behavior (contact surfaces for modeling contraction joints and sliding along discontinuities, and validated constitutive models for concrete cracking)

- Development of efficient and numerical sound techniques and solution strategies for computing the nonlinear response.

- Development of criteria for acceptable performance.

- Identification of possible modes of failure.

(d) See Canadian Electrical Association (CEA 1990a) for additional guidance on using linear and nonlinear dynamic methods of analysis for concrete dams.

(3) Foundation Stability Dynamic Analysis

(a) General. A dynamic stability analysis should generally be performed on the same foundation blocks and wedges that were identified and used in the static stability analysis. The same methods of foundation analysis used for static stability calculations are used for dynamic evaluations with the following additions, modifications, and exceptions:

- The analysis must include dynamic loads from both the dam acting on the foundation blocks and the inertial loads from the blocks themselves.

- The inertial load from the mass of the foundation block is included. If a time-history analysis is being performed, the load is equal to the mass multiplied by the acceleration at any instant of time. If a response spectrum analysis is being performed, appropriate effective peak acceleration is used.

- If the analysis indicates that the sliding resistance on the potential sliding plane (frictional resistance plus intact rock cohesion) is exceeded at some point during the earthquake loading, a strength reduction (typically reduction or elimination of cohesion) is necessary. Dynamic strengths of the intact rock and/or concrete must also be accounted for in making the analysis.

- Deformations can be computed for the foundation blocks by a Newmark-type rigid-block sliding analysis, and the dam's performance can be evaluated by comparison with acceptable-deformation criteria. This approach is especially valuable for cases where blocks can be assumed to be formed by continuous joints, faults, and shear zones. For atypical dams, commonly-used criteria may not be applicable, and there can be separate issues for appurtenant features.

- Research has shown that dynamic loading can affect water pressures in joints and shear zones. At this time, however, water pressure adjustments to account for that are not routine in practice.

(b) Seismic Stability Criteria. Seismic stability criteria for foundations depend on the method of analysis and the basic assumptions made for the geologic discontinuities. The three general cases are:

- A specified minimum factor of safety in a pseudo-static analysis at the time of critical dynamic loading from the dam, in which the rock is considered to be intact.

- A specified minimum factor in pseudo-static analysis of blocks assumed to be bounded by fully continuous joints, shears, or faults.

- Acceptable deformations calculated by time-history analysis with or without consideration of the strength of intact rock, in which sliding during excursions are evaluated below a factor of safety less than 1.0 (if any).

(c) For guidance on the use of two- and three-dimensional rigid block and finite element methods of dynamic analysis for rock foundations see Canadian Electrical Association (CEA 1990b).

(d) Coupled dam and foundation analysis. Typically, the foundation stability analysis is performed separately from the dam analysis, as discussed above. A more realistic approach is to analyze the dam and foundation together in a coupled analysis. This way, the interaction between the dam and the rock wedges in question can be modeled. Because a coupled analysis can model non-linearity caused by changes in geometry, explicit coupled methods can be very effective.

(4) Material Properties for Dynamic Analysis. The ability to analyze a dam often exceeds the ability to define material properties. This is particularly true for nonlinear analysis where many times the material models in the finite element codes have theoretical inputs that can only be estimated and not tested. Concrete and rock are not homogeneous materials. The material properties can be quite variable and possess a degree of uncertainty. This should be kept in mind when performing investigations. Sensitivity studies using variations on material properties should be performed where appropriate.

(a) Concrete Properties for Analysis. The concrete properties required for input into a linear dynamic analysis are the unit weight, Young's modulus of elasticity, damping, and Poisson's ratio. The concrete properties used should account for, as nearly as practicable, the effects of aging and existing cracks and the expected rate of loading.

(b) Concrete Strengths for Evaluation of Results. The concrete properties needed to perform dynamic analysis are the compressive, tensile, and shear strengths. The standard unconfined uniaxial compression test, excluding creep effects, is generally acceptable as a first test for the compressive strength. Usually, this test suffices, even though it does not account for the rate of loading, since compressive generally does not control the outcome of the analysis. The tensile strength of concrete generally governs because it is a small fraction of the compressive strength.

(c) Concrete Tensile Strength. Tensile strengths of the lift joints and the parent concrete are determined from tests on cores taken from test fill placements for new dams and the actual concrete in existing dams. The modulus-of-rupture test and the splitting tensile test are commonly used for the parent concrete. Direct tensile tests are preferred, but less-expensive splitting tensile tests can be used, with the results adjusted to reflect the results of a smaller number of direct tensile tests. The direct tensile test is also used to measure the strength of the lift lines. From these tests, allowable design tensile stresses can be established for both lift joints and parent concrete can be adjusted using factors to account for the high strain rate associated with dynamic loading. However, the best approach is to actually test the strength of the parent concrete and lift lines under rapid loading. Tensile and bond strengths of concrete dams are as much related to mix consistency, placement methods, and compaction procedures as they are to mix proportions. Maximum aggregate size, consistency, and mortar bedding also influence tensile strength of the joints. The effects of pre-existing cracks, lift joints, and construction joints on the tensile strength must also be accounted for, whether assessing the results of linear-elastic analysis or performing nonlinear analysis. If dynamic analyses indicate high tensile stresses, the effects of cracks and weakened joints can be modeled conservatively by assuming that cracking occurs, and reanalyzing the dam to determine whether the remaining capacity of the dam in the cracked state is sufficient.

(d) Rock Properties for Analysis. When the foundation is included in the seismic analysis, elastic moduli and Poisson's ratios for the foundation materials are required. If the foundation is not considered as massless, rock densities and damping characteristics are also required. Determining the elastic moduli for a rock foundation may be accomplished using one of several approaches that account for the effects of rock inhomogeneity and discontinuities on foundation behavior. The determination of foundation compressibility should consider both elastic and inelastic (plastic) deformations. The resulting "modulus of deformation" is lower than the elastic modulus of intact rock. The effects of the rate of loading on the foundation moduli is considered insignificant relative to the other uncertainties involved in determining rock foundation properties; rate effects are generally not measured. To account for the uncertainties, lower and upper limits for the foundation moduli should be used in the structural analysis to determine the sensitivity of the results to variation in moduli. For dynamic loading, the upper limit is generally the most realistic estimate because the inelastic "set" has already occurred. However, the shear strength used for static analysis is usually used for dynamic analysis without adjustment. The shear strength of discontinuities is generally a critical element in the stability analysis.

(e) Damping Ratio. For concrete gravity dams built on competent rock, where cracking of the concrete does not occur, the viscous damping ratio is usually assumed to be 5 percent of critical. If calculated stresses indicate that cracking would occur, a value from 7 to 10 percent of critical can be used (depending on the severity of cracking expected). Some analysis codes use hysteretic damping and Rayleigh damping, which require different values specific to the form of damping. It may also be necessary to account for radiation damping and the effects of reservoir-bottom reflection (Nuss et al 2003, Ghanaat 1995).

c. Sliding Stability

(1) Methods of Analysis. The sliding stability evaluation of hydraulic structures under earthquake loading can be made according to the traditional pseudostatic (seismic coefficient) and permanent displacement approaches (Ebeling and Morrison 1992). In the traditional approach, the sliding stability is expressed in terms of a prescribed factor of safety against sliding, whereas in the permanent displacement approach, the structure is permitted to slide along its base but the accumulated displacement during the ground shaking is limited to a specified allowable value. Some federal agencies have encouraged the use of conducting a post earthquake stability analysis to demonstrate seismic stability. Under this type of analysis, conservative assumptions are used to reduce the amount of information needed of the ground motions, and the exploration and testing required of the dam/foundation material properties. For example, a typical analysis would assume the seismic event has created a crack that extends entirely through the base of the dam. Sliding along deep seated failure surface in the foundation should also be considered or ruled out. An analysis is then conducted, assuming no cohesion and a reduced friction angle, to determine if the dam has an adequate post earthquake factor of safety or if remediation or additional analysis is warranted.

(2) Pseudostatic (Seismic Coefficient) Method. In this method, seismic inertia forces due to weight of the structure and to hydrodynamic pressures are included as part of the driving force, and a static analysis of sliding is performed (Ebeling and Morrison 1992). Treating the system above the failure surface as a rigid block, the inertia force associated with the mass of the structure is computed as the product of the assumed earthquake acceleration (seismic coefficient x g) and the mass of the block. Similarly, the product of the earthquake acceleration and the added-mass of water that is moving with the structure, according to the theory of Westergaard (1933), produce inertia force due to the hydrodynamic pressures. The motion of the structure relative to the failure surface is resisted by the shear strength mobilized between the structure and failure surface by the friction and cohesion. When the earthquake forces are included in the sliding stability analysis, the calculated factor of safety against sliding (i.e., ratio of the resisting to driving shear forces) may become less than one. When this occurs, the sliding is assumed to occur for as long as the ground acceleration is greater than the critical value required for the driving force to exceed the resistance. However, due to oscillatory nature of the earthquake ground motion, the sliding displacement is limited and can be estimated by the permanent displacement approach described below.

(3) Permanent Displacement Method. The factor of safety against sliding required by the pseudostatic method (e.g., >1.0-1.5 at all times) may not be attainable for larger seismic forces representative of moderate to high intensity earthquake ground motions. Where the sliding factor of safety drops below 1.0, sliding can occur during periods of time when the acceleration cycles exceed a critical acceleration period of time associated with the acceleration cycles exceeding a critical acceleration, a_c, and the relative velocity between the structure and the base is greater than zero (Chopra and Zhang 1991). Knowing the critical acceleration a_c, the permanent sliding displacements can be estimated using Newmark's rigid block model (Newmark 1965). Newmark's model provides an easy means for approximate estimation of the upper bounds for permanent sliding displacements, but is based on certain assumptions that ignore the true dynamic response behavior of the sliding. More accurate estimates of the sliding displacements can be made from the response history analysis proposed by Chopra and Zhang as referenced above.

d. Rotational Stability. Hydraulic structures subjected to large lateral forces produced by major earthquakes may tip and start rocking when the resulting driving moment becomes so large that the structure breaks contact with the foundation or cracks all the way through at an upper elevation. However, under earthquake excitation large driving moments occur for only a fraction of a second in each cycle, with intermediate unloading. Stability studies of structures (Housner 1963, Scholl 1984) infer that dynamic rotational stability for rigid structures with height to base width ratios typical of gravity dams is not a problem. In addition, the study by Chopra and Zhang (1991) indicate that a concrete dam under earthquake loading will slide downstream before tipping will occur. U. S. Army Corps of Engineers (USACE 1999) discusses each of the above references. However, if the dynamic analysis indicates that extensive cracking, block rocking, or translational displacement is expected, the dam in its post-earthquake condition should be checked for sliding stability to assess if it is capable of containing the reservoir for a sufficient period of time to allow for strengthening of the dam, if necessary, to restore the dam to acceptable factors of safety for the various credible loading conditions. The post-earthquake analysis should use residual shear strength parameters.

C. Embankment Dams

1. Safety Concerns. The first step in the design or evaluation of any dam is the development of an understanding of the way the dam can fail. Several earthquake-induced conditions that can cause failure of an embankment dam are described below.

a. Overtopping of the Embankment. The evaluation of the seismic safety of embankment dams often depends on determining, directly or indirectly, the magnitude of expected deformations. If the embankment crest falls below the elevation of the reservoir surface, erosion from overtopping can cause the dam to fail. Direct methods of assessing deformation model the design earthquake, the dam, and foundation to calculate the expected deformation. There are also indirect methods to predict the response of the embankment and foundation based on empirical observations. Post-earthquake stability analysis can, in a sense, also be considered an indirect prediction of deformation – if the post-earthquake factor of safety is high, the deformations should be limited to a few feet except under very severe loading. Deformation analyses are discussed further below.

The magnitude of deformations is very dependent on the strengths of the materials involved. During strong shaking, permanent deformations (usually small) may occur simply because the dynamic stresses temporarily exceed the available strength. In saturated soils, there is frequently some loss of shearing resistance due to an increase in pore water pressure when shaken. This increases the dynamic deformations over what they would be with no strength loss. In very loose, contractive soils, the strength may become a small fraction of its static, drained value due to excess pore-water pressure, a process called "liquefaction." If the strength drops below that required for static stability, very large deformations can result, driven by gravity even after the shaking ends. These can be anywhere from a few feet to hundreds of feet, depending on the driving forces, geometry, and post-earthquake strength. Liquefaction evaluations are described in 3.b. of this section. There is an intermediate condition known as "cyclic mobility," in which the shearing resistance is initially very low due to excess pore pressure, but increases with larger shear strains, helping to prevent gross instability but still permitting significant deformation.

Overtopping leading to failure of the dam can also result from the following:

- Movement on a fault through the reservoir or through the embankment foundation, causing the reservoir elevation to rise above the crest of the dam (or the dam crest to fall below the reservoir surface).

- An earthquake-induced landslide that displaces a significant volume of water.

- A large seiche wave generated by an earthquake.

b. Cracking and Internal Erosion. If a dam is deformed by earthquake excitation or fault displacement, the deformations can cause cracks in the dam and/or disrupt internal filters, either of which could lead to failure of the dam by erosion. Cracks are most likely to occur at interfaces with concrete structures (e.g., spillway walls) or at abrupt changes in the embankment's cross section. There is also evidence that shaking could precipitate piping even without formation of a crack if the dam is already on the verge of piping. The amount of deformation a dam can withstand without risk of failure by erosion through cracks depends on the materials in the dam and foundation, the details of internal zoning (filters, drains, and cutoff), the reservoir elevation at the time of the earthquake, and the nature and location of appurtenant structures. Should there be conduits through the embankment, deformation of the dam can rupture them or cause joints to separate, leading to erosive failure by either creating an unfiltered exit for seepage or exposing the embankment or foundation to full reservoir head where not intended. Erosion along intact conduits has also caused dam failures.

2. Defensive Design Measures. Many situations that could develop into a dam safety emergency or failure of the dam if not properly designed do not always require extensive analytical evaluation. The simple application of defensive measures will sometimes give the evaluator assurance that the structure will perform satisfactorily, assuming even the worse scenario. Conversely, poorly designed defensive measures may invalidate the beneficial effects of the defensive measures. Such defensive measures include the following:

- Remove foundation materials that may present problems.

- Use wide core zones of plastic materials resistant to erosion.

- Use well-graded wide filter zones upstream of the core to help choke-off any cracks that may open and downstream of it to prevent movement of particles eroded from the core.

- Construct chimney drains downstream of the embankment core to lessen saturation.

- Use crest details and downstream slope protection that will prevent or greatly inhibit erosion in the event of moderate overtopping.

- Flare the embankment core at abutment contacts.

- Locate the core to attain the lowest practicable phreatic line within the embankment.

- Stabilize slopes around the reservoir rim to prevent slides into the reservoir.

- Provide special details for treating the embankment-foundation interface if the potential for fault movement in the foundation exists.

- Provide high quality, free draining rockfill shells.

- Provide ample freeboard to allow for settlement, slumping, or fault movements.

- Shape foundation contacts to avoid abrupt changes in profile, overhangs, or large "stairsteps."

- Compact embankment fill materials adequately to prevent or minimize generation of excess pore pressures.

- Provide filters or other measures to prevent erosion along the outside of conduits or other structures within the embankment.

3. Methods of Analysis

a. General. For a dam and foundation not subject to liquefaction, minor deformation may take place but should not lead to failure if all of the following conditions are satisfied:

- Dam and foundation materials are not subject to liquefaction and do not include loose soils or sensitive clays.

- The dam is well built and compacted to at least 95% of the laboratory maximum dry density, or to a relative density greater than 80%.

- The slopes of the dam are 3:1 (H:V) or flatter, and/or the phreatic line is well below the downstream slope of the embankment.

- The peak horizontal acceleration at the base of the embankment is no more than 0.2 g.

- The static factors of safety for all potential failure surfaces (other than shallow surficial slides) are greater than 1.5 under loading and pore-pressure conditions expected immediately prior to the earthquake.

- The freeboard at the time of the earthquake is at least 3% to 5% of the embankment height and not less than 3 feet (0.9 m). (Freeboard requirements to accommodate reservoir seiche waves or coseismic movement of faults at the dam site or in the reservoir must be considered as a separate issue.)

- There are no critical appurtenant features that would be harmed by small movements of the embankment, or that have the potential to cause cracks that would allow internal erosion.

If these conditions are not satisfied, more detailed study is required. This may include assessment of liquefaction potential, post-earthquake stability analysis, and/or deformation analysis. If there are no potentially liquefiable materials present, this can usually be done by the simple Newmark sliding-block approach, discussed in c. below. In situations where excess pore pressure could develop, it may be necessary to conduct more rigorous (but more time-consuming) finite-element or finite-difference analyses. The objective of deformation analysis is to determine whether plausible movements would be sufficient to allow overtopping by the reservoir, or if cracking at critical locations could result in failure by internal erosion. From these results and historic performance of embankment dams subjected to earthquake loading, one must make an overall assessment of whether the dam and foundation can be expected to safely withstand the loading.

b. Liquefaction Evaluation

(1) General. For existing dams/foundations, as well as for the foundations for proposed new dams, the most important part of a liquefaction investigation is adequate subsurface investigations (mapping, drilling, sampling, geophysical) so that the extent of any weak material is identified. In general, sands, gravels, and fine-grained nonplastic soils should be evaluated for susceptibility to liquefaction. The basic steps for the procedure are described below. It is important to determine whether a continuous weak layer could exist over some large area of the foundation or embankment. If the investigation is unable to rule this out, the analysis should be made, assuming that the layer is continuous.

Before beginning an evaluation of liquefaction potential, appropriate ground motions must be established.

Evaluations of liquefaction potential are usually made by indirect reference to the considerable body of case histories of earthquake-induced liquefaction and absence of liquefaction under earthquake loading. The most widely used procedures are based on correlations between results of standard penetration tests (SPTs) or cone penetration tests (CPTs), and field performance under various levels of loading. Note, however, that to verify the material types predicted by CPT test data, samples must be obtained, preferably by SPTs, which would also provide corroboration for the indications of the CPT regarding liquefaction potential. Soils that contain gravels can be tested using the Becker Hammer penetration test, with appropriate correlations to estimate equivalent SPT values from the Becker blowcounts (Harder and Seed 1986, Sy and Campanella 1994, Youd and Idriss 1997). As with the CPT, separate sampling holes are required because the BPT does not retrieve a sample. Down-hole or cross-hole measurements of shear-wave velocity (SWV) can also be used for gravelly soils (Andrus and Stokoe 1997).

(2) Analysis for Static Factor of Safety Immediately After Shaking. If liquefaction would be triggered by the evaluation earthquake, a conventional slope stability evaluation using post-earthquake properties and conditions should be made to determine whether instability would result. The critical slide masses typically consist of active and passive wedges with a neutral block in between (sliding on a thin horizontal layer of liquefied soil), but they can also be defined by circular slip surfaces. The strength of the liquefied material is generally estimated from correlations between SPT blowcount and strengths back-calculated from historic failures (Seed and Harder 1990), or by other in-situ or laboratory strength testing. If the factor of safety against sliding is greater than 1.0 for expected conditions following the earthquake, gross instability will presumably not occur. However, because of the great uncertainty in such calculations, post-liquefaction factors of safety are generally required to be a minimum of 1.2 to 1.3, along with some limit being placed on the expected deformation. If the factor of safety is below 1.0, the only conclusion that can be drawn is that large deformations would occur; it is not possible to predict where the slide would stop without sophisticated deformation analysis by nonlinear finite-element or finite-difference codes. Section 3 below discusses methods for calculating the expected deformation.

(3) Post-Earthquake Deformation Analysis

(a) General. If the post-earthquake stability analyses indicate factors of safety against sliding above 1.0, the expected amount of deformation can be estimated using several methods. The most rigorous method is to use finite element or finite difference programs such as TARA (Finn et al 1986), FLAC (Itasca Group 2002), or PLAXIS (PlaxisBV 2002) in which dynamic response, pore-pressure development, and deformations can be fully coupled. Simpler, more common methods for estimating deformation are the Newmark method (Newmark 1965) and the Makdisi-Seed simplified method based on the Newmark method (Makdisi and Seed 1977). These two are applicable only to slopes with post-earthquake factors of safety significantly above 1.0. Simpler still are deformation estimates based on historic performance with procedures (Byrne 1991, Swaisgood 1993). Included below is a step-by-step procedure to determine embankment/foundation liquefaction resistance.

(b) Practical Approach. A practical engineering approach to analyzing an embankment/foundation for liquefaction resistance and resulting seismic stability using the techniques developed by the late Professor H. B. Seed and other researchers (Seed and Harder 1990) is summarized in the following steps:

- Conduct SPTs and/or CPTs at close spacing, as continuously as practical. In addition, obtain samples for laboratory index tests (grain size and Atterberg limits), and to confirm predictions of soil type from the CPT.

- Establish the effective stresses existing in the embankment and foundation before the earthquake, preferably using a two-dimensional finite element program such as FEADAM (Duncan et al 1985). Well below the foundation contact, the one-dimensional approximation may be reasonable.

- Select ground motions (acceleration vs. time) to be used in analysis.

- Establish dynamic shear moduli of the soils in the embankment and foundation. These can be determined by *in-situ* geophysical measurements (down-hole and cross hole testing) and by comparison with published values for similar materials. Other needed dynamic properties, such as modulus degradation and damping ratio as a function of shear strain, have been investigated by Seed (et al 1984) and others. Available "generic" curves for damping and modulus degradation can often be used.

- Calculate the earthquake-induced stresses in the embankment and its foundation. This can be performed in a one-dimensional columnar analysis using the computer program SHAKE (Schnabel et al 1972) if the suspected liquefiable material is in the foundation or near the lower part of the embankment. For high embankments or other site conditions where a two-dimensional dynamic analysis is warranted, a two-dimensional FEM program such as QUAD4M (Hudson et al 1994) may be needed.

- Evaluate the liquefaction resistance of the embankment and foundation using the adjusted SPT, CPT, and/or SWV data using available correlations.

- If the factor of safety against triggering liquefaction from the previous two steps is near or below 1, residual strength values should be assigned. This is done using (Seed and Harder 1990) correlation with SPT data or laboratory testing. It may also be necessary to reduce the strength of other zones below their drained strength due to excess pore pressure from cyclic loading or simply from undrained shearing, as in the case of normally consolidated clays.

- Perform a static post-earthquake slope-stability analysis using the *lower bound* residual strength values for the liquefied zones. If this shows the factor of safety against sliding is below 1.0, the only conclusion that can be made is that instability could occur. If the embankment has a factor of safety against sliding above 1.0, a Newmark analysis, or preferably a large-strain finite-element or finite-difference analysis, can be used to evaluate the dynamic performance of the embankment, as described below.

c. Deformation Analysis for Non-Liquefying or Non-Sliding Conditions. If instability is not expected or no liquefaction is anticipated, deformations should be estimated by one or both of the following methods, depending on the nature of the materials at the site.

(1) Newmark Deformation Analysis

(a) General. The Newmark deformation analysis (Newmark 1965) is the most common method for evaluating the dynamic performance of an embankment dam. The method assumes that the mode of deformation in response to earthquake shaking will be the sliding of rigid blocks (masses) of dam and foundation material. Sliding is assumed to occur whenever the base acceleration exceeds the yield acceleration, which is the horizontal seismic acceleration that results in a factor of safety of exactly 1.0 using conventional slope stability analysis. Through the course of the earthquake ground motion, sliding stops and starts as the factor of safety rises above and falls below 1.0.

(b) Input Motion. To perform this analysis, a time-history representation of the motions along the potential sliding surface is required. This motion is obtained based on a dynamic response analysis of the dam.

(c) Makdisi-Seed Simplified Method. A simplified method, based on application of the Newmark method to several dams using several different earthquakes, was developed by Dr. Faiz I. Makdisi and Dr. H. Bolton Seed (Makdisi and Seed 1977). The resulting nomograph procedure is commonly referred to as the Makdisi-Seed Method. This approach provides an estimate of expected net displacement as a function of (1) pseudostatic horizontal yield coefficient for a potential slide mass; (2) peak effective horizontal acceleration within the slide mass; and (3) duration of strong shaking, approximated by an empirical correlation with earthquake magnitude. Makdisi and Seed also developed a simplified response analysis to find the peak acceleration of the slide mass that treats the dam embankment as a triangular prism of elastic material and approximates its response by a Bessel series.

(d) Other "Newmark-Type" Methods. Several other methods based on the Newmark procedure have been developed. The most basic of these is the Sarma analysis, which represents the earthquake by a horizontal acceleration and computes the horizontal movement of the mass. Subsequent computation of loss of crest elevation, based on the shape of the potential sliding surface, is required. More rigorous analyses include both horizontal and vertical accelerations, as well as variations of pore pressures and shear strength with time during the earthquake (Von Thun and Harris 1981). A detailed case study on the deformation analysis for Eastside Reservoir Project, Riverside County, California is available (USCOLD 1996). Another study (Hynes and Franklin 1984) concluded from a series of Newmark analyses that deformations of an embankment dam would be fairly minor as long as the peak horizontal ground acceleration is less than 3 times the yield acceleration (provided that there would be no excess pore pressures from shaking and that undrained shear strengths are properly accounted for). This is consistent with observed performance of dams in earthquakes.

(e) Analysis Results. Once one has obtained an estimate of deformation from the Newmark (or other) method, the expected performance of the dam under seismic loading is judged considering the severity of deformation (loss of available freeboard and potential for cracking leading to failure of the embankment or foundation by internal erosion). Limiting the amount of allowable deformations along critical failure surfaces to two feet has been used as a criterion by some agencies.

(2) Generalized Settlement. In addition to deformation of the embankment from slipping in response to earthquake shaking, the dam may settle in response to the stresses developed in each soil element. This generalized settlement can be estimated by using soil mechanics consolidation, empirical, and/or finite element procedures. An empirical relationship has been developed (Jansen 1988) for estimating settlement of an embankment dam under seismic loading if the dam and foundation are not susceptible to liquefaction.

d. Pseudostatic Analysis. A pseudostatic analysis (sometimes called seismic coefficient analysis) should only be considered as an index of the seismic resistance available in a structure not subject to build-up of pore pressure from shaking. It is not possible to predict failure by pseudostatic analysis, and other types of analysis are generally required to provide a more reliable basis for evaluating field performance. However, if there are no soils present that would develop excess pore pressure from shaking, and undrained shear strengths are properly incorporated where needed, a pseudo-static factor of safety greater than 1.0 is very strong evidence that there would be little or no damage to the dam from an earthquake.

V. EVALUATION OF STRUCTURAL ADEQUACY FOR EARTHQUAKE LOADING

A. Performance Criteria

Dams should be capable of withstanding the MDE without failure resulting in a catastrophic loss of the reservoir, or should present an acceptably low risk of failure if that is how the relevant agency approaches decision making. Damage without catastrophic release of the reservoir may be acceptable in some cases, provided the risk of release is suitably low. However, the extent of damage allowed should not impair the ability to quickly and safely draw down the reservoir to alleviate any potential threat to downstream life and property. Even if there is no potential for sudden loss of the reservoir, determining the acceptable level of damage to a project may be linked to the ability to quickly undertake repairs and return the project to full operation. If drawing down the reservoir to undertake repairs cannot be done easily, then significant damage may not be acceptable. In addition, agencies typically require that dams should be capable of resisting an OBE without sustaining serious damage (or present a suitably low risk of damage), so that they remain operational and do not require extensive repair work. All systems and components necessary to maintain the project should be designed to remain operable during and after the OBE (or have sufficiently low risk of becoming inoperable). Other factors that may play a role in determining the acceptability of the performance of the dam following an earthquake are (1) the use of the reservoir; and (2) the ability or lack thereof to quickly repair a damaged structure. A primary water supply facility damaged from an earthquake may cause severe economic and health consequences while projects with lost freeboard and cracking may be vulnerable to further damage from after shocks or the next significant flood.

B. Evaluating Analysis Results for Concrete Dams

1. Dynamic Analyses. Results of dynamic analyses are generally evaluated in terms of compressive and tensile strengths of the concrete. The compressive stresses resulting from the combination of static and earthquake loads usually remain below the dynamic strength of the concrete. However, since the mere occurrence of tensile stresses does not necessarily lead to failure, the significance of predicted tensile stresses is not evaluated as easily. The number and amplitudes of stress cycles that exceed the dynamic tensile strength are taken into account for this purpose in linear analysis. To evaluate the effects of stresses that exceed the tensile strength, sound engineering judgment is required and should be based on the expected effects of nonlinear behavior and the past performance of dams under similar earthquake loadings. To estimate the extent of cracking, one must consider nonlinear behavior resulting in stiffness degradation and energy absorption. Nonlinear behavior from cracking reduces the stiffness of the dam and shifts the dam's response into other frequency contents of the ground motion. The energy level of the earthquake corresponding to the frequency of the cracked structure may or may not be more severe than when uncracked. As a result, the peak values of tensile stress and the extent of tensile zones may increase or decrease, and large tensile stresses given by linear elastic analysis may or may not necessarily indicate an unsafe condition. They may, in fact, be "artifacts" of the analysis rather than real behavior. The past performance of concrete dams under large earthquake loading is discussed later in this section. A nonlinear analysis should be performed if the response of the dam would be influenced significantly by nonlinearity from material behavior or changes in geometry. For an arch dam, this might include 1) cantilever tensile stresses larger than the tensile strength of the concrete over significant areas of the dam; 2) a long duration earthquake; 3) opening and closing of contraction joints indicated by simultaneous arch tensions on the upstream and downstream faces; and 4) large displacements or distortions of the arch.

2. Sliding and Overturning Stability. Pseudostatic methods are generally discouraged and should only be used for screening from further consideration those dams where a seismic stability failure is highly improbable. Structures that fail to meet the prescribed pseudostatic stability requirements (i.e., sliding safety factors and resultant location) should be subjected to in-depth study, but should not be considered unsafe until the in-depth analysis is performed. A better screening devise used by some federal agencies, discussed above under section c. Sliding Stability (1) Methods of Analysis, is to conduct a post earthquake stability analysis to demonstrate seismic stability. Under this type of analysis, conservative assumptions are used to reduce the amount of information needed of the ground motions and the exploration and testing required of the dam/foundation material properties. The failure to meet these requirements should only suggest the need for dynamic analyses that fairly assess the demands placed on the dam and foundation during a major earthquake.

Dynamic time-history analyses are used to determine the displacements and stresses experienced by the dam and foundation. Evaluation of the results is used to determine if there is a risk of a stability failure. Physical model studies of the Koyna Dam conducted at the University of California at Berkeley (Niwa and Clough 1980) demonstrated the inherent capability of a cracked dam to withstand severe earthquake shaking without a sliding or overturning failure even though extreme rocking of the cracked upper section of the model occurred. However, this is not to say that all gravity dams are stable during an earthquake. Concrete dams (gravity, arch, or buttress dam) and sites are very unique and should all be evaluated for stability. Consequences from a dam failure can be too large for anything less to be acceptable.

C. Evaluating Analysis Results for Embankment Dams

1. General. In some cases, the analyses may indicate that the dam is either clearly safe or clearly unsafe. Frequently, however, judgments concerning the safety of the dam must take into account not only the results of the analyses but also the level of confidence that can be put in those analyses and underlying assumptions and, to some extent, the consequences of misjudging the level of uncertainty.

2. Stability Analyses. If the results of post-earthquake sliding stability analyses for critical failure surfaces indicate a safety factor well above 1.0 (e.g., 1.25 or greater) using the strengths expected after the earthquake, experience from past earthquakes suggests that deformations will be small and the dam will perform satisfactorily. Confidence in the safety of the dam decreases when the factor of safety against triggering of liquefaction is 1.0 or less and a post-earthquake sliding factor of safety less than or approaching 1.0 is calculated using residual shear strengths for materials assumed to be liquefied. In general, many analyses have shown that when a wedge or circular sliding surface has a low post-earthquake sliding factor of safety, the deformations on these sliding planes will be excessive. If these failure planes are critical to the overall integrity of the dam, deformations may lead to failure of the dam by overtopping or internal erosion.

3. Deformation Analysis. Predicted deformations, whether from a Newmark analysis or nonlinear finite-element or finite-difference analysis, should not be greater than what the dam can safely withstand without catastrophic release of the reservoir. Important factors to consider in making this determination are discussed below. Deformation analyses can be made for three conditions: 1) liquefaction would not occur; 2) liquefaction may occur but instability would not; and 3) liquefaction may occur, resulting in instability. In the first two conditions, judgment is required to determine whether the predicted deformations along the critical failure surfaces are small enough that cracking of the embankment/foundation materials does not occur that could eventually cause a piping failure of the dam. A determination also must be made whether the post-earthquake sliding factors of safety and available freeboard are adequate to ensure the dam would not be overtopped and would be able to safely retain the reservoir. The predicted deformations are very sensitive to the post-earthquake strength that is used in the analysis. The critical question is often will liquefaction be triggered and, if it is, what is the appropriate residual shear strength parameter to use in the analysis. Confidence, or lack thereof, in the strengths that are used, and hence confidence in the predicted deformations, are important factors to consider in evaluating the results of a deformation analysis. In the case where large deformations or relatively low post-earthquake sliding factors of safety are predicted, the question is generally whether a lowered reservoir level can be safely maintained without the potential loss of the reservoir. A comprehensive review of the factors to be considered in the earthquake resistance design of dams, as well as a review and commentary on the field performance of dams during earthquakes, can be found in Seed (1979). A compilation of relevant papers can also be found in Duncan (1990).

D. Past Experience of Dams Shaken by Earthquakes

USCOLD published a report (USCOLD 1992) listing the observed performance of various types of dams during earthquakes. Some of these case studies are listed in the following sections. In addition to these case studies, a summary of the performance of embankment dams was documented by Swaisgood (1993). This information was updated by for the Eastside Reservoir Project (Ebasco et al 1994).

1. Concrete Dams. No major concrete dam is known to have failed due to earthquake-induced ground motion, although several are known to have experienced strong ground motion. The following is a review of some of these experiences.

a. Gravity Dams

(1) The Crystal Springs Dam, a 154-foot-high gravity-arch dam located only one-fourth of a mile from the San Andreas Fault, survived the 8.3 magnitude (estimated) 1906 San Francisco earthquake with no apparent damage.

(2) Several 200-foot-high gravity dams in Japan experienced earthquakes producing an MM intensity of VIII in the area of the dams and were not damaged. However, some examples exist of partial earthquake damage to concrete dams in Japan. In 1923, the piers of spillway gates at the top of a hydroelectric intake dam cracked at their bases. These piers were of plain concrete, indicating that reinforcing steel should be used in piers of dams built in areas where large earthquakes are anticipated. In 1943, a gravity dam for deposition of muck from a mine was sheared at a horizontal section at the elevation of approximately two-thirds up the dam height. However, the cross section of this dam was smaller than that of a water storage dam, probably because the pressure exerted by deposited muck was less than the water pressure.

(3) In 1967, the Koyna Dam, a 340-foot-high gravity dam in India, survived a near-field magnitude 6.4 earthquake, with an MM intensity of VIII or IX at the site. A peak horizontal acceleration of 0.51g perpendicular to the dam axis and a peak vertical acceleration of 0.36g, were measured at the dam. The dam experienced major cracking on both upstream and downstream faces of the non-overflow monoliths. The overflow monoliths were not damaged. A dynamic analysis of the Koyna Dam was performed using two-dimensional finite element methods that incorporated hydrodynamic interaction and assumed linear elastic behavior in the concrete and foundation. The analysis indicated tensile stresses in the upper part of the non-overflow monoliths exceeded the tensile strength of the concrete up to three times. Analysis of the overflow monolith indicated maximum tensile stresses approximately equal to the concrete tensile strength.

(4) Hoover Dam, a 726-foot-high curved gravity dam, has been suspected of being the cause of moderate reservoir-triggered seismicity of Richter Magnitude M5 or less (USCOLD 1992). No damage to the dam has occurred due to these earthquakes.

(5) In 1995, the concrete dams of Sengari and Aono, located 15 km and 30 km north of the earthquake source in Kobe, Japan, were shaken by the Great Hanshin earthquake of Richter magnitude M7.2, but suffered no damage (Krinitzsky et al 1995). These concrete dams were founded on rock and received relatively low levels of ground motion. The level of ground motion is based on the fact that there was little damage to residences with tile roofs and unstable grave markers in a cemetery nearer the closest dam were not overturned. Overturning of grave marker stones approximately 4-feet high with a 4-inch-square base indicated horizontal ground motion of approximately 0.2g. Peak rock acceleration within the source area in Kobe registered an equivalent 0.45g.

b. Arch Dams

(1) In 1971, the Pacoima Dam, a 372 foot high arch dam, survived the magnitude 6.6 San Fernando earthquake, the epicenter of which was only four miles north of the dam. Despite the estimated 0.6 to 0.8g foundation acceleration, there was no evidence of distress within the dam. However, a separation of the arch dam proper and the left abutment thrust block occurred in the upper part of the dam; the thrust block and upper abutment moved away from the dam. A three-dimensional linear finite element analysis of this arch dam was performed using the accelerograms (modified) recorded on the rock above the left abutment of the dam during the

San Fernando event. The analysis indicated tensile stresses in the dam that should have caused cracking due to the earthquake; however, on-site visual examination of the dam did not reveal cracking of the concrete. Similar studies of the Big Tujunga concrete arch dam, which was also shaken by the San Fernando event, have been conducted using three-dimensional finite element procedures. The analysis results showed peak tensile stresses that should have caused cracking, but again there was no physical evidence of any structural damage to the dam from that earthquake.

(2) In 1976, several Italian arch dams (Ambiesta, Maina di Sauris, and Barcis) were subjected to a series of earthquakes with reported intensities as high as MM IX in the vicinity of the dams. At Ambiesta Dam, a 194 foot high arch dam, the peak ground acceleration was recorded to be 0.33g at the right abutment (USCOLD 1992). No structural damage was reported at any of these dams during inspection after the earthquakes.

(3) In 1994, the Pacoima Dam was again severely shaken by the Northridge/San Fernando Earthquake, a Moment Magnitude of M6.7 with its epicenter approximately 11 miles southwest of the site (Morrison Knudsen 1994). Peak horizontal acceleration values of up to 0.5g horizontal and 0.4g vertical were measured near the base of the dam. Cross-canyon accelerations on the abutments were measured at 2.0g horizontal and 1.3g vertical. There were three significant consequences of the earthquake on the dam: (1) cracking in the left block of the arch which extends from the upstream to the downstream face; (2) permanent upstream deflection of the crest of the dam (on the order of 1.8 inches); and (3) opening of the contraction joint between the dam and the left thrust block. Two rock masses, one critical to the stability of the left abutment of the dam, permanently displaced away from the dam (on the order of inches) during the earthquake. The spillway tunnel and covered downstream chute suffered considerable damage as a result of rockslides. The Pacoima Reservoir, which is maintained low to provide storage for floodwater, was at about 65 percent of the height of the dam (at station 0+00) at the time of the earthquake.

(4) In 1992, Bear Valley Dam survived the M6.7, Big Bear Lake Earthquake, located approximately 8 miles from the epicenter. The Bear Valley Dam was originally constructed as a multiple arch dam. However, in 1989 the multiple arch bays were filled with mass concrete. No damage was reported to the dam.

(5) Some of the observed ability of arch dams to resist large earthquakes, even though calculations show large tensile stress, may be due to the ability of arch dams to transfer load. When cracking occurs as a result of large tensile cantilever stresses, the decrease in flexibility of the section will cause loads to transfer to adjacent arches and cantilevers, thereby reducing the areas that are over-stressed. Vertical cracking may be pre-empted by additional damping, due to nonlinearity of material properties or the computed, but nonexistent, formation of horizontal tensile stresses across contraction joints. The implementation of good defensive design mechanisms in the construction of arch dams may have contributed to their sound performance. These factors were discussed at the beginning of this chapter and include adequate abutment and foundation investigations, good design practices that alleviate local stress concentrations, and an effective quality control program to assure the dam is constructed as designed.

c. Buttress Dams

(1) In 1962, the Hsingfengkiang Dam, a 345 foot high buttress dam in China, survived a near-field magnitude 6.1 earthquake, but developed horizontal cracks at a change of section in the non-overflow blocks on each side of the spillway, although no leakage occurred (USCOLD 1992, Clough and Ghanaat 1993). No instrumentation was installed at the time of the main shock, but strong motion seismographs were brought in to record the aftershocks and a magnitude 4.5 aftershock produced a peak acceleration of 0.54g at the dam crest.

(2) In 1990, the Sefid Rud Dam, a 106-meter-high buttress dam in northern Iran, was damaged by an earthquake of Richter Magnitude of M7.3 at a distance of 5 km (Ahmadi et al 1992, Clough and Ghanaat 1993). The principal damage to the central monoliths were cracks at lift joints extending from the dam face through the buttress face and web. These occurred adjacent to a change of slope near the crest of the dam and were accompanied by a 20 mm shear-displacement toward downstream. Offsets between buttresses indicated about 50 mm of distortion of the whole valley (USCOLD 2000). There was minor damage and displacement on all of the gates. This event was approximately equivalent to the MCE for the site. By extrapolation from the 15 accelerograms from nearby sites, the peak horizontal ground acceleration at the dam was estimated as 0.71g, greatly exceeding the design earthquake for the dam.

2. Embankment Dams. The behavior of earth and rock-fill dams when subjected to earthquake shaking depends strongly on the design of internal drainage features and the method of construction used. A comprehensive review of past experience with numerous embankment dams shaken by six earthquakes produced the following conclusions (Seed et al 1978, Seed 1979).

a. Hydraulic fill dams have been found to be vulnerable to failures under unfavorable conditions, in particular shaking produced by strong earthquakes. The near failure of the Lower Van Norman Dam during the 1971 San Fernando earthquake is the most famous case history in that it triggered the development of modern methods for the dynamic analysis of embankment dams (USCOLD 1992). This earthquake, with a Richter Magnitude of M6.5, was centered approximately 11.2 km from the dam.

b. Virtually any well-built compacted embankment dam can withstand moderate earthquake shaking, with peak accelerations of 0.2g and more, with no detrimental effects.

c. Dams constructed of clay soil on clay or rock foundations have withstood extremely strong shaking, ranging from 0.35 to 0.8g, from a magnitude 8.25 earthquake with no apparent damage. This conclusion is based on the results of the performance of 33 embankment dams that were shaken during the 1906 San Francisco earthquake. These dams range in height from 15 to 140 feet, and are located within 37 miles (60km) of the San Andreas Fault. Almost one-half (16) were located within 5 miles (8km) of the causative fault. Explorations performed in the 1980's and observations of piezometers indicate that puddled cores in some of the higher dams may have kept the downstream slopes from being saturated. Performance of embankment dams in earthquakes subsequent to the Seed publication (Seed et al 1978) has also been good, except when liquefaction or unusual circumstances have been involved. See Matahina Dam below.

d. Two rockfill dams have withstood moderately strong shaking with no significant damage. If the rockfill dam is kept dry by means of an impervious facing, they should be able to withstand extremely strong shaking with only small deformations. The strongest shaking to which a concrete face rockfill (CFR) dam has been subjected to is around 0.2g. In areas of moderate seismic shaking, CFR dams have been built with slopes of the order of 1H:1.3V. Analysis of the dams under these conditions show the dam should have an acceptable level of seismic performance. When the design earthquake is as high as Magnitude 7.5 and capable of producing a peak ground acceleration of 0.5g, the desirable slope is on the order of 1H:1.65V (Seed et al 1985). However site-specific analysis should always be performed for critical structures.

e. The performance of modern compacted embankment dams was further demonstrated in 1994 when the Los Angeles Reservoir, a modern reservoir constructed in 1979 comprising two rolled fill embankments, was severely shaken by the Northridge Earthquake, a magnitude M_L of 6.8 with its epicenter approximately 7 miles south of the site (Davis and Sakado 1994). The Los Angeles Dam (LAD), on the south side of the reservoir, is 155 feet high. The North Dike (ND), on the northern side of the reservoir, is 117 feet high. Both dams are founded on Saugus Formation bedrock, are zoned with shell material on the upstream and downstream slopes, and contain a chimney drain at the center section. The LAD also has a clay zone upstream of the chimney drain. The degree of compaction of fill, clay zone, and blended sand and gravel was specified to be compacted to 93 percent modified Proctor (ASTM D1557 2002) and was carefully controlled during construction. Peak horizontal acceleration values of up to 0.43g were measured on the right abutment and 0.56g on the crest of the LAD. A survey of the reservoir following the earthquake showed 3-1/2 in. of crest settlement and lateral downstream movement of over 1 in. on the crest of LAD. The ND showed no more than 1-1/4 in. of crest settlement and lateral downstream movement of less than 1/3 in. on the crest. The reservoir was within nearly 80 percent of its capacity, as defined by the high water elevation, at the time of the earthquake.

f. A few recent case histories have provided opportunities to verify analysis procedures for estimating earthquake-induced deformations. In each of these cases, instrumental measurements have been made of accelerations and displacements during and after strong ground shaking, allowing comparison with numerical model predictions of the same quantities.

(1) Elgamal (et al 1990) describe the dynamic response and permanent deformations recorded at La Villita Dam in Mexico consequent to five earthquakes, most recently the September 19, 1985 Michoacan Earthquake. Yielding was confirmed to have occurred within the embankment by asymmetry of acceleration records recorded at various points on the dam, i.e., some of the peaks in the acceleration history were truncated as material slipped in the vicinity of the accelerograph. The authors used simplified displacement estimation techniques involving a “sliding rock” analogy (Newmark 1965) to successfully predict stick-slip type deformations and to evaluate the accumulation of displacement over several earthquake events.

(2) Matahina Dam, New Zealand was shaken and deformed by the 1987 Edgecumbe Earthquake (M6.7)and the dynamic response and displacements were recorded by extensive instrumentation (Finn et al 1994). A nonlinear finite element computer code, TARA-3 (Finn et al 1986), was employed to simulate the performance of the Matahina Dam, using engineering properties carefully determined by laboratory and field-testing. Deformations were generally small (less than 0.5m), but the results of this study boosted the confidence of geotechnical engineers to analytically estimate earthquake behavior of embankment dams for stronger events. About 10 months later, a sinkhole appeared near one abutment, and fairly extensive erosion damage was found in the low-plasticity core (Gillon and Newton 1991). The earthquake was thought to have accelerated or reactivated the erosion damage that was discovered and repaired shortly after first filling.

g. To summarize, experience has shown that well-compacted, impervious rolled-fill dams are resistant to earthquake forces, provided they are constructed on rock or overburden foundations resistant to liquefaction. The same is true of well-drained, compacted rockfill dams or dumped rockfill dams with impervious cores, although some surface deformation can be expected on steep slopes. Rockfill dams with membrane facing (e.g., concrete) have performed well under strong shaking; however, permanent displacement or cracking of the facing can be expected which may require remediation following the seismic event. Low-density embankments built of low plasticity granular soils, especially hydraulic or semihydraulic fills, are highly susceptible to earthquake damage due to the potential for liquefaction. Existing dams that have been constructed on foundations of low density cohesionless materials formed in continuous layers also may be subject to excessive deformations during the seismic event due to liquefaction.

E. Evaluating Existing Dams

The criteria and analysis methods cited in these guidelines may differ significantly from those in use when many older dams were designed. Therefore, some existing structures may be found deficient using the procedures outlined. If the analyses indicate the dam is over-stressed or unstable under the earthquake conditions examined, the consequences of failure should be determined and assessed. A decision analysis, scoped to the magnitude of potential consequences, should be used as guidance for further action. If the safety evaluation indicates that failure of the dam does not pose a hazard to downstream life or property, remedial action may not be necessary. However, this position should be fully documented.

F. Seismic Remediation of Earth Dams Subject to Liquefaction

Present methods available for engineering remediation of seismic deficient earth dams are listed below.

1. Excavate and Replace. This method provides positive assurance that what is designed is actually constructed in the field. This approach is viable when the suspect material is near the surface and it can be removed down to a firm foundation.

2. *In Situ* Densification or Cementing. Stone columns, vibrocompaction, compaction grouting, and other methods of soil mixing and densification. Compaction grouting requires extreme caution because of the possibility of hydraulic fracturing in the embankment/foundation.

Verification of the amount of improvement should be required when using these methods.

3. Berms/buttresses. These are used to increase the effective overburden pressures and thus decrease the potential for liquefaction. It also increases the factor of safety by lengthening the potential failure surface.

4. Drains. Use of drains to lower the phreatic surface in the embankment can be an effective way to treat potentially liquefiable material. However, dependence on drains to reduce the buildup of excess pore pressure in potentially liquefiable soils during an earthquake creates many uncertainties and is generally not a recommended practice. One unknown that is a major factor in the effectiveness of the drains is the permeability of the problem material.

5. Additional Freeboard. Additional freeboard can be obtained by either raising the embankment crest or restricting the reservoir surface. This would lower the potential for loss of the reservoir if large deformation of the dam were to occur during the seismic event. It should be recognized, however, that a crest raise to increase the freeboard adds weight to the crest, increasing the driving force for instability or deformation.

APPENDIX A
REFERENCES

Abrahamson and Silva 1997
Abrahamson, N. A., Silva, W. J., 1997. “Empirical Response Spectral Attenuation Relationships for Shallow Crustral Earthquakes”, Seismology Research Letters, Seismological Society of America, Vol. 68, No. 1, Jan. 1997, pp 94-127

Ahmadi et al 1992
Ahmadi, M. T., Khoshrang, G. H., Mokhtarzadeh, A., and Jalalzadeh, A., 1992. “Behavior of a Large Concrete Dam to an Actual Maximum Credible Earthquake”, Madrid, Spain, Vol. 7, pp. 3995-4000

Andrus and Stokoe 1997
Andrus, R. D. and Stokoe II, K. H., 1997. "Liquefaction Resistance Based on Shear Wave Velocity," Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance, Salt Lake City, 1996, National Center for Earthquake Engineering Research, Technical Report 97-0022

ASTM 2002
American Society for Testing and Materials, “Standard Test Method for Laboratory Compaction Characteristics of Soil Using Modified Effort”, ASTM D1557-02, ASTM International, P. O. Box C700, West Conshohocken, PA, 19428.

Bryne 1991
Byrne, P. M., 1991. "A Model for Predicting Liquefaction Induced Displacement," Proceedings: Second International Conference on Recent Advances in Geotechnical Engineering & Soil Dynamics, March, 1991

CEA 1990a
Canadian Electrical Association, 1990. Safety Assessment of Existing Dams for Earthquake Conditions, Vol. C-4, Report for the Canadian Electrical Association, One Westmount Square, Suite 500 Montreal, Quebec, H3Z 2P9, pg. 39

CEA 1990b
Canadian Electrical Association, 1990. Safety Assessment of Existing Dams for Earthquake Conditions, Vol. C-4, Report for the Canadian Electrical Association, One Westmount Square, Suite 500 Montreal, Quebec, H3Z 2P9, pg.57

Chopra and Zhang 1991
Chopra, A.K. and Zhang, L., May 1991, "Base Sliding Response of Concrete Dams to Earthquakes" Report No. UCB/EERC-91/05, Earthquake Engineering Research Center, University of California, Berkeley, CA

Clough and Ghanaat 1993
Clough, R.W. and Ghanaat, Y., 1993. Concrete Dams: “Evaluation for Seismic Loading in Dam Safety Evaluation Proceedings,” Vol. 4, Grindewald, Switzerland, pp. 137-169

Davis and Sakado 1994
Davis, C.A. and Sakado, M.M., 1994. “Response of the Van Norman Complex to the Northridge Earthquake”, Association of Dam Safety Officials Conference Proceedings, Boston, MA, pp. 241-255

Duncan et al 1985
Duncan, J. M., Seed, R. B., Wong, K. S., Ozawa, Y., Feb. 1984. “FEADAM84: A Computer Program for the Finite Element Analysis of Dams”, Virginia Polytechnical Institute

Duncan 1990
Duncan, J. M., editor, 1990. "H. Bolton Seed Selected Papers, Volume 1 and 2”, BiTech Publishers

Ebasco et al 1994
Ebasco, Black & Vetch, Woodward-Clyde, 1994. "West Dam Ground Design Memorandum" Metropolitan Water District of Southern California, Eastside Reservoir

Ebeling and Morrison 1992
Ebeling R. M. and Morrison, Jr., E. E., 1992. "The seismic design of waterfront retaining structures," Technical Report ITL-92-11, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Elgamal et al 1990
Elgamal, A. M., Scott, R. F., Succarish, M. F., and Yan, L., 1990. “La Villita Dam Response During Five Earthquakes Including Permanent Deformation”, Journal of Geotechnical Engineering, American Society for Civil Engineers, Vol. 116, No. 10, pp. 1443-1462

FEMA 1997
Federal Emergency Management Agency, 1997. "NEHRP, Recommended Provisions for the Development of Seismic Regulations for New Buildings", National Earthquake Hazards Reduction Program

FEMA 1998
Federal Emergency Management Agency, 1998. “Federal Guidelines for Dam Safety: Hazard Potential Classification Systems for Dams”, National Dam Safety Program, FEMA 333, October 1998, reprinted April 2004

Finn et al 1986
Finn, W. D. L., Yogendrakumar, M., Yoshida, N., Yoshida, H., 1986. “TARA-3: A Computer Program to Compute the Response of 2-D Embankments and Soil Structure Interaction Systems to Seismic Loadings”, Department of Civil Engineering, University of British Columbia, Vancouver, Canada

Finn et al 1994
Finn, W. D. L., Ledbetter, R. H., and Marcuson III, W. F.,January 12-16, 1994. “Seismic Deformations in Embankments and Slopes,” Proceedings, Symposium on Developments in Geotechnical Engineering (From Harvard to New Delhi, 1936-1994), ed. by A.S. Balasubramaniam, S.W. Hong, N. Phien-Wej, and P. Nutslaya, , Bangkok, Thailand, pp. 233-264

Ghanaat 1995
Ghanaat, Yusof, 1995. “Measurements of Reservoir-Bottom Reflection Coefficients at Seven Concrete Damsites”, Report Number QS95-01, Quest Structures, Orinda, CA

Gillon and Newton 1991
Gillon, M. and Newton C., 1991. "Abutment Repairs at Matahina Dam", Proceedings of the 17th Congress on Large Dams, ICOLD, Vienna

Harder and Seed 1986
Harder, L. F., Jr. and Seed, H. B., 1986. "Determination of Penetration Resistance for Coarse-Grained Soils Using the Becker Hammer Drill", Report No. UCB/EERC-86/06, University of California, Berkeley, CA

Hays 1980
Hays, W. W., 1980. Procedures for Estimating Earthquake Ground Motions, U.S. Geological Survey Professional Paper 1114

Housner 1963
Housner, G.W., 1963. "The behavior of inverted pendulum structures during earthquakes," Bulletin of Seismological Society of America, Vol. 53, No. 2, pp. 403-417

Hudson et al 1994
Hudson, M., Idriss, I. M., Beikae, M., May 1994. “QUAD4M: A Computer Program to Evaluate the Seismic Response of Soil Structures Using Finite Element Procedures and Incorporating a Compliant Base”, Center for Geotechnical Modeling, University of California, Davis, CA

Hynes and Franklin 1984
Hynes, M. E. and Franklin, F. G.,1984. “Rationalizing the Seismic Coefficient Method," U.S. Army Corps of Engineers, Waterways Experiment Station, Miscellaneous Paper GL-84-13, Vicksburg, MS

Itasca Group 2002
“FLAC, Fast Lagrangian Analysis of Continua”, 2002. Itasca Consulting Group, Inc., Thresher Square East, 708 South Third Street, Suite 310, Minneapolis, MN 55412.

Jansen 1988
Jansen, R. B., Editor, 1988. Advanced Dam Engineering for Design, Construction, and Rehabilitation, Van Nostrand Reinhold, 115 Fifth Avenue, New York, NY

Krinitzsky et al 1995
Krinitzsky, E. L., Koester, J. P., and Hall, R. L., 1995. Memorandum for Record Summarizing Reconnaissance by U.S. Army Corps of Engineers Waterways Experiment Station Team of the Great Heshin Earthquake of 17 January 1995.

Makdisi and Seed 1977
Makdisi, F. I. and Seed, B. H., August 1977. "A Simplified Procedure for Estimating Earthquake-Induced Deformations in Dams and Embankments", Earthquake Engineering Research Center Report No. UCB/EERC-77/19, University of California, Berkeley, CA

Morrison Knudsen 1994
Morrison Knudsen Corporation 1994. "Report on the Initial Assessment of the Effects of the January 17, 1994 Northridge/San Fernando Earthquake on Pacoima Dam, Phase I", Report to the Dept. of Public Works, County of Los Angeles, CA

Newmark 1965
Newmark, N.M. 1965. "Effects of Earthquakes on Dams and Embankments," Geotechnique, Vol. 15, No. 2, pp. 139-160

Newmark and Hall 1982
Newmark, N. M. and Hall, W. J., 1982. "Earthquake spectra and design; Engineering monographs on earthquake criteria, structural design, and strong motion records", Vol. 3: Earthquake Engineering Research Institute, University of California, Berkeley, CA

Newmark and Rosenblueth 1971
Newmark, N. M. and Rosenblueth, E., 1971. "Fundamentals of earthquake engineering," Prentice-Hall, Englewood Cliffs, N.J.

Niwa and Clough 1980
Niwa, A. and Clough, R. W. 1980. Shaking Table Research on Concrete Dam Models. Report No. UCB/EERC 80/05, University of California, Berkeley

Nuss et al 2003
Nuss, L. K., Chopra, A. K., and Hall J. F., 2003. "Comparison of Vibration Generator Tests to Analyses Including Dam-Foundation-Reservoir Interaction for Morrow Point Dam", 21st ICOLD Conference, Montreal, Canada, June 2003

Page 1997
Page, W. D., Lettis, W. R., and Abrahamson, N. A., 1997. Characterization of Seismic Sources and Estimated Ground Motions for Lake Almanor and Butt Valley Dams, Plumas County, CA, WATERPOWER Conference 1997

Payne 2002
Payne, T., 2002. "Shaking Table Study to Investigate Failure Modes of Arch Dams" DSO-02-01, Bureau of Reclamation, U. S. Department of Interior, Denver, CO

PLAXIS BV 2002
PLAXIS BV, 2002. “PLasticity AXISymmetry”, PLAXIS BV, P.O. Box 572, 2600 AN DELFT, The Netherlands

Reclamation 1993
Bureau of Reclamation,. 1993. "Development of Seismotectonic and Ground Motion Parameters," ACER Technical Memorandum No. 20 (Working Draft) U.S. Dept. of Interior, Denver, CO

Reiter 1991
Reiter, Leon, 1991. Earthquake Hazard Analysis, Issues and Insights, Columbia University Press, New York, NY

Schnabel et al 1972
SHAKE, Schnabel, P. B., Lysmer, J., Seed, H. B., Dec. 1972. “SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites”, Report Number EERC 72-12, Earthquake Engineering Research Center, University of California, Berkley, CA

Scholl 1984
Scholl, 1984. "Critical Aspects of Earthquake Ground Motion and Building Damage Potential," Applied Technology Council Publication ATC 10-1, pp. 105-111

Seed et al 1978
Seed, H. B., Makdisi, F. I., and De Alba, P., July 1978. “Performance of Earth Dams During Earthquakes,” Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol., 104, No. GT7, pp. 967-994

Seed 1979
Seed, H. B, Sept. 1979. “Considerations in the Earthquake-Resistant Design of Earth and Rockfill Dams”, 19th Rankine Lecture of the British Geotechnical Society, Geotechnique, Vol. XXIX, No. 3, pp. 215-263

Seed et al 1984
Seed, H. B., Wong, R. T., Idriss, I. M., Tokimatsu, K., 1984. “Moduli and Damping Factors for Dynamic Analyses of Cohesionless Soils”, Report Number EERC 84-14, Earthquake Engineering Research Center, University of California, Berkley, CA, Sept. 1984

Seed et al 1985
Seed, H. Bolton, Seed, Raymond B., Lai, S.S., Khamenehpour B., 1985. "Seismic Design of Concrete Faced Rockfill Dams," Concrete Face Rockfill Dams-Design, Construction and Performance Symposium Proceedings, ASCE, October 1985, pp. 459-478

Seed and Harder 1990
Seed, R. B. and Harder Jr., L. F., May 1990. "SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Residual Strength", H. Bolton Seed Memorial Symposium Proceedings

Somerville et al 1997
Somerville, P. G., Smith, N. F, Graves, R. W., Abrahamson, N. A., 1997. "Modification of Empirical Strong Motion Attenuation Relations to Include the Amplitude and Duration Effects of Rupture Directivity", Seismology Research Letters 68

Somerville 1998
Somerville, P. G., August 1998. "The Amplitude and Duration Effects of Rupture Directivity in Near Fault Ground Motions," Proceedings of the 1998 ASCE Specialty Conference on Geotechnical Earthquake Engineering and Soil Dynamics, Seattle, WA, August 3-6, 1998

SSA 1997
Seismological Society of America, 1997. Research Letters, Volume 68, Number 1, January/February 1997

Swaisgood 1993
Swaisgood, J. R., 1993. "Historical Behavior of Embankment Dams During Earthquakes," Proceedings of the Association of State Dam Safety Officials National Conference, Kansas City, MO

Sy and Campanella 1994
Sy, A., Campanella, R.G., 1994. "Becker and Standard Penetration Tests (BPT-SPT) Correlations with Consideration of Casing Friction," Canadian Geotechnical Journal, Vol. 31, No. 3, pp. 343-356

USACE 1995
U.S. Army Corps of Engineers, July 31, 1995. “Earthquake Design for Civil Works Projects,” Engineering Regulation No.1110-2-1806, Washington, D.C.

USACE 1999
U. S. Army Corps of Engineers, June 30, 1999. “Response Spectra and Seismic Analysis for Concrete Hydraulic Structures”, Engineering Manual 1110-2-6050, Washington, D.C.

USCOLD 1992
United States Committee on Large Dams, July 1992. “Observed Performance of Dams during Earthquakes”, USCOLD Committee on Earthquakes

USCOLD 1996
United States Committee on Large Dams, 1996. “Sixteenth Annual Lecture Series, Seismic Design and Performance of Dams”

USCOLD 1997
United States Committee on Large Dams, April 1997. “Reservoir Triggered Seismicity”, USCOLD Committee on Earthquakes

USCOLD 1999
United States Committee on Large Dams, 1999. “Guidelines for Selecting Seismic Parameters for Dam Projects”, USCOLD Committee on Earthquakes

USCOLD 2000
United States Committee on Large Dams, 2000. “Observed Performance of Dams during Earthquakes, Volume II”

Von Thun and Harris 1981
Von Thun, J. L., Harris, C.W., 1981. “Estimated Displacements of Rockfill Dams Due to Seismic Shaking”, Proceedings of the International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Vol. 1, St. Louis, MO, April 26-May 3, 1981, pp. 417-423

Westergaard 1933
Westergaard, H. M., 1933. “Water Pressure on Dams during Earthquakes”, Transactions of ASCE

Youd and Idriss 1997
Youd, T. L., Idriss, I. M., Editors, 1997. “NCEER Workshop on Evaluation of Liquefaction Resistance of Soils, January 5-6, 1996, Salt Lake City, UT”, National Center for Earthquake Engineering Research, NCEER 97-0022

APPENDIX B-1
BIBLIOGRAPHY

SEISMOLOGY, DETERMINISTIC SEISMIC HAZARD ANALYSIS, PROBABILISTIC SEISMIC HAZARD ANALYSIS, & GROUND MOTIONS

Algermissen 1983
Algermissen, S. T. 1983. An Introduction to the Seismicity of the United States. Engineering Monograph (MNO-7), Earthquake Engineering Research Institute, Oakland, CA

Ambraseys 1988
Ambraseys, N. N. 1988, "Engineering Seismology," Earthquake Engineering and Structural Dynamics, Vol. 17, No. 1, pp. 1-105

Bolt 1988
Bolt, B.A., 1988, Earthquakes: W. H. Freeman and Co., New York

Department of the Army 1995
Department of the Army. 1995. Engineering Manual-Response Spectra and Seismic Analysis for Concrete Hydraulic Structures, EC l110-2-6050, Office, Chief of Engineers, Washington, D.C.

Green 1992
Green, R. 1992. Selection of Ground Motions for the Seismic Evaluation of Embankments, ASCE Specialty Conference on Stability and Performance of Slopes and Embankments 11, A 25-Year Perspective, Geotechnical Special Publication No. 31, Vol. 1, pp. 593-607

Hudson 1979
Hudson, D. E. 1979. Reading and Interpreting Strong Motion Accelerograms, Earthquake Engineering Research Institute, 499 14th Street, Oakland, CA, p. 112

Idriss 1993
Idriss, I. M. 1993 (Mar). Procedures for Selecting Earthquake Ground Motions at Rock Sites, A Report to the National Institute of Standards and Technology, U.S. Dept. of Commerce, Gaithersburg, MD

Joyner and Boore 1988
Joyner, W. B. and Boore, D. M. 1988. Measurement, Characterization and Prediction of Strong Ground Motion, Earthquake Engineering and Soil Dynamics 11 -Recent Advances in Ground Motion Evaluation, ASCE Geotechnical Special Publication No. 20, pp. 43-102

Krinitzsky and Chang 1987
Krinitzsky, E. L. and Chang, F. K. 1987. "State-of-the-Art for Assessing Earthquake Hazards in the United States, Report 25 Parameters for Specifying Intensity-Related Earthquake Ground Motions," MP S-73-1, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Krinitzsky and Others 1987
Krinitzsky, E. L., Chang, F. K., and Nuttli, 0. W. 1987. "State-of-the-Art for Assessing Earthquake Hazards in the United States, Report 26-Parameters for Specifying Magnitude-Related Earthquake Ground Motions," MP S-73-1, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Krinitzsky and Others 1993
Krinitzsky, E. L., Gould, J. P., and Edinger, P. H. 1993. Fundamentals of Earthquake Resistant Construction, John Wiley and Sons, Inc. New York, NY

Leeds 1992
Leeds, D. L. 1992. "State-of-the-Art for Assessing Earthquake Hazards in the United States, Report 28 Recommended Accelerograms for Earthquake Ground Motions," MP S-73-1, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

National Research Council 1988
National Research Council. 1988. Probabilistic Seismic Hazard Analysis: National Academy Press, Washington, D.C., p. 97

Newmark and Hall 1983
Newmark, N.N., and Hall, W.J. 1983. Earthquake Spectra and Design, Earthquake Engineering Research Institute, 499 14th Street, Oakland, CA, p. 103

Nuttli 1981
Nuttli, O.W., “Similarities and Differences Between Western and Eastern United States Earthquakes and their Consequences for Earthquake Engineering,” Earthquakes and Earthquake Engineering: Earthquakes in the Eastern United States, Vol. 1, ed. J. E. Beavers, 1981, pp. 25-52

Reiter 1991
Reiter, L. 1991. Earthquake Hazard Analysis -Issues and Insights. Columbia University Press. New York, NY

Schneider 1993
Schneider, J. F. 1993, Guidelines for determining design basis ground motions. Volume 1: Method and guidelines for estimating earthquake ground motion in eastern North America, EPRI TR-102293: Electric Power Research Institute, Palo Alto, CA

Schwartz 1988
Schwartz, D.P. 1988. Geologic Characterization of Seismic Sources: Moving Into The 1990's, Proceedings, Earthquake Engineering and Soil Dynamics II --Recent Advances in Ground-Motion Evaluation, ASCE Geotechnical Special Pub. No. 20, pp. 1-42

APPENDIX B-2
BIBLIOGRAPHY

CONCRETE GRAVITY DAMS, ARCH DAMS, BUTTRESS DAMS AND STRUCTURES APPURTENANT TO DAMS

Chandrashaker and Humar 1993
Chandrashaker, R. and Humar, J.L. 1993. Fluid-Foundation Interaction in the Seismic Response of Gravity Dams, Earthquake Engineering and Structural Dynamics, Vol. 22, pp. 1067-1084

Chavez and Fenves 1994
Chavez, J.W. and Fenves, G.L. 1994. EGAD-SLIDE: A Computer Program for the Earthquake Analysis of Concrete Gravity Dams Including Base Sliding, Rept. UCB/SEMM-94/02, Dept. of Civil Engrg., Univ. of California at Berkeley, Berkeley, CA

Chavez and Fenves 1995
Chavez, J.W. and Fenves, G.L. 1995. Earthquake Response of Concrete Gravity Dams Including Base Sliding, Journal of Structural Engineering, ASCE, Vol. 121, No. 5, pp. 865-875

Chopra 1967
Chopra, A.K. 1967 (Dec). Hydrodynamic Pressures on Dams During Earthquakes, ASCE, Journal of Engineering Mechanics Division, Vol. 93, No. EM6, pp. 205-223

Chopra 1978
Chopra, A.K. 1978 (Jun). Earthquake Resistant Design of Concrete Gravity Dams, ASCE, Journal of the Structural Division, Vol. 104, No. ST6, pp. 953-971

Chopra 1992
Chopra, A.K. 1992 (July). Earthquake Analysis, Design, and Safety Evaluation of Concrete Arch Darns, paper presented at Tenth World Conference on Earthquake Engineering, Madrid, Spain, and Published in report entitled Earthquake Engineering Research at Berkeley -1992, Rept. UCBfEERC-92/13, University of California, Berkeley CA.

Chopra 1995
Chopra, A. 1995. Dynamics of Structures -Theory and Applications to Earthquake Engineering, Prentice-Hall, p. 29

Chopra and Tan 1989
Chopra, A.K. and Tan, H. 1989 (Mar). Simplified Earthquake Analysis of Gated Spillway Monoliths of Concrete Gravity Darns, U.S. Army Corps of Engineers, Waterways Experiment Station Technical Report SL-89-4, Vicksburg, MS

Chopra and Tan 1992
Chopra, A. K. and Tan, H. 1992. Modeling of Dam Foundation Interaction in Analysis of Arch Dams, Proc. 10th World Conference on Earthquake Engineering, Madrid, Spain

Chopra and Zhang 1991
Chopra, A.K. and Zhang, L. 1991 (May). Base Sliding Response of Concrete Gravity Dams to Earthquakes, Rept. UCBiEERC-91/05, Univ. of California, Berkeley, CA

Chopra and Zhang 1991
Chopra, A. K. and Zhang, L. 1991 (Dec). Earthquake-Induced Base Sliding of Concrete Gravity Dams, Journal of Structural Engineering, ASCE, Vol. 117, No. 12, pp. 3698-3719

Clough and Ghanaat 1993
Clough, R. W. and Ghanaat, Y. 1993. Concrete Dams: Evaluation for Seismic Loading, in Dam Safety Evaluation Proceedings, Vol. 4, Grindelwald, Switzerland, pp. 137-169

Clough and Zienkiewicz 1982
Clough, R. W. and Zienkiewicz, O. C. 1982. Finite Element Methods in Analysis and Design of Dams, Committee on Analysis and Design of Dams

Damay and Adegha 1993
Danay, A. and Adeghe, L. N. 1993 (Jan). Seismic-Induced Slip of Concrete Gravity Dams, Journal of Structural Engineering, ASCE, Vol. II 9, No. 1, pp. 108-129

Department of the Army 1993
Department of the Army 1993. Engineering Manual-Arch Dam Design, EM 1110-2-2201, Office, Chief of Engineers, Washington, D.C.

Department of the Army 1995
Department of the Army 1995. Engineering Manual-Gravity Dam Design, EM 1110-2-2200, Office, Chief of Engineers, Washington, D.C.

Dowling and Hall 1989
Dowling, M. J. and Hall, J. F. 1989 (Apr). Nonlinear Seismic Analysis of Arch Dams, Journal of Engineering Mechanics, ASCE, Vol. II 5, No. 4, pp. 768-789

El-Aidi and Hall 1989
El-Aidi, B. and Hall, J. 1989. Non-Linear Earthquake Response of Concrete Gravity Dams Part 1: Modeling, Earthquake Engineering and Structural Dynamics, Vol. 18, pp. 837-851

El-Aidi and Hall 1989
El-Aidi, B. and Hall, J. 1989. Non-Linear Earthquake Response of Concrete Gravity Dams Part 2: Behavior, Earthquake Engineering and Structural Dynamics, Vol. 18, pp. 853-865

Fenves and Chopra 1983
Fenves, G. and Chopra, A. K. 1983 (Nov-Dec). Effects of Reservoir Bottom Absorption on Earthquake Response of Concrete Gravity Dams, Vol. 11, No. 6, pp. 809-829

Fenves and Chopra 1984
Fenves, G. L. and Chopra, A. K. 1984. EGAD-84: A Computer Program for the Earthquake Analysis of Concrete Gravity Dams, Rept. UC/EERC-84/11, Univ. of California, Berkeley, CA

Fenves and Chopra 1984
Fenves, G. and Chopra, A. K. 1984 (Sept-Oct). Earthquake Analysis of Concrete Gravity Dams Including Reservoir Bottom Absorption and Dam-Water-Foundation Rock Interaction, Earthquake Engineering and Structural Dynamics, Vol. 12, No. 5, pp. 663-680

Fenves and Chopra 1985
Fenves, G. and Chopra, A. K. 1985 (Jan-Feb). Effects of Reservoir Bottom Absorption and Dam-Water-Foundation Rock Interaction on Frequency Response Functions for Concrete Gravity Dams, Earthquake Engineering and Structural Dynamics, Vol. 13, No. 1, pp. 13-31

Fenves and Chopra 1985
Fenves, G. and Chopra, A.K. 1985. Reservoir Bottom Absorption Effects in Earthquake Response of Concrete Darns, Journal of Structural Engineering, ASCE, Vol. 111, No. 3, pp. 545-562

Fenves and Chopra 1986
Fenves, G. and Chopra, A.K. 1986 (June). Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams, Rept. UCB/EERC-85/10, Univ. of California, Berkeley, CA

Fenves and Chopra 1985
Fenves, G. and Chopra, A. K. 1985 (June). Simplified Earthquake Analysis of Concrete Gravity Dams: Separate Hydrodynamic and Foundation Interaction Effects, Journal of Engineering Mechanics, ASCE, Vol. 111, No. 6, pp. 715-735

Fenves and Chopra 1985
Fenves, G. and Chopra, A.K. 1985 (June). Simplified Earthquake Analysis of Concrete Gravity Dams: Combined Hydrodynamic and Foundation Interaction Effects, Journal of Engineering Mechanics, ASCE, Vol. 111, No. 6, pp. 736-756

Fenves and Chopra 1987
Fenves, G.L. and Chopra, A.K. 1987. Simplified Analysis of Concrete Gravity Dams, Journal of Structural Engineering, ASCE, Vol. 113, No. 8, pp. 1688-1708

Fenves and Others 1992
Fenves, G.L., Mojtahedi, S. and Reimer, R.B. 1992 (Apr). Effect of Contraction Joints on quake Response of an Arch Dam, Journal of Structural Engineering, ASCE, Vol. II 8, No. 4, pp. 1039-1055

Ghanaat 1993
Ghanaat, Y. 1993. Theoretical Manual for Analysis of Arch Dams, U.S. Army Corps of Engineers, Waterways Experiment Station Technical Report ITL 93- 1, Vicksburg, MS

Ghanaat 1993
Ghanaat, Y. 1993. User's Manual-GDAP, Graphics-Based Dam Analysis Program Version 3.25, U.S. Army Corps of Engineers, Waterways Experiment Station Instruction Report IT 93-3, Vicksburg, MS

Hall and Others 1991
Hall, J.F., Dowling, M.J. and El-Aidi, B. 199 1. Defensive Design of Concrete Gravity Darns, Report No. EERL 91-02, California Inst. of Technology, Pasadena, CA

Janson 1988
Janson, R. B. (editor) 1988. Advanced Dam Engineering for Design, Construction and Rehabilitation, Van Nostrand Reinhold, p. 811

Leger and Katsoull 1989
Leger, P. and Katsoull, M. 1989. Seismic Stability of Concrete Gravity Dams, Earthquake Engineering and Structural Dynamics, Vol. 18, pp. 889-902

Panel on Earthquake Engineering for Concrete Dams 1990
Earthquake Engineering for Concrete Dams: Design, Performance, and Research Needs, National Academy Press, Washington, D.C., p. 143

Priscu and Others 1985
Priscu, R., Popovici, A., Stematiu, D. and Stere, C., 1985. Earthquake Engineering for Large Dams, John Wiley, 2nd edition

Scott and Dreher 1983
Scott, G. A., and Dreher, K. J. 1983. Dynamic Stability of Concrete Darn Foundations, 5th International Society for Rock Mechanics Congress, Melbourne, Australia, Proceedings, pp. C227-C233

Touhei and Ohmachi 1993
Touhei, T. and Ohmachi, T. 1993. A FE-BE Method for Dynamic Analysis of Dam-Foundation-Reservoir Systems in the Time Domain, Earthquake Engineering and Structural Dynamics, Vol. 22, pp. 195-209

Yang and Others 1996
Yang, R., Tsai, C. S. and Lee, G. C. 1996 (Feb). Procedure for Time-Domain Seismic Analyses of Concrete Dams, Journal of Engineering Mechanics, ASCE, Vol. 122, No. 2, pp. 116-126

USCOLD 1995
Guidelines for Earthquake Design and Evaluation of Structures Appurtenant to Dams

APPENDIX B-3
BIBLIOGRAPHY

EARTHEN DAMS, ROCKFILL DAMS & LIQUEFACTION HAZARD ANALYSIS

Ambraseys and Sanna 1967
Ambraseys, N. N. and Sanna, S. K. 1967. "The Response of Earth Dams to Strong Earthquakes," Geotechnique, Vol. 17, No. 2, pp. 181-213

Banerjee and Others 1979
Banerjee, N. G., Seed, H. B., and Chan, C. K. 1979. "Cyclic Behavior of Dense Coarse-Grained Materials in Relation to the Seismic Stability of Dam," Report No. UCB/EERC-79/13, University of California, Berkeley, CA

Bureau and Other 1985
Bureau, G., Volpe, R.L., Roth, W. H., and Udaka, T. 1985. Seismic Analysis of Concrete Face Rockfill Dams, Proc. of Geotechnical Engineering, Division of ASCE Symposium on Concrete Face Rockfill Dams-Design, Construction, and Performance, pp. 479-508

Byrne 1991
Byrne, P. M., "A Model for Predicting Liquefaction Induced Displacement," Proceedings: Second International Conference on Recent Advances in Geotechnical Engineering & Soil Dynamics, March, 1991

Castro and Others 1985
Castro, G., Poulos, S. J., and Leathers, F. D. 1985. "A Re-examination of the Slide of the Lower San Fernando Dam," Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 111, No. 9, pp 1093-1107

Chugh 1982
Chugh, A.K. 1982. Slope Stability Analysis for Earthquakes, International Journal for Numerical and Analytical Methods for Geomechanics, Vol. 6, No. 3, pp. 307-322

Department of the Army 1970
Department of the Army 1970. Laboratory Soil Testing, Engineer Manual EM II 10-2-1906, 1980 Revision, Office, Chief of Engineers, Washington, D.C.

Department of the Army 1979
Department of the Army. 1979. Geophysical Exploration, Engineer Manual EM II I0-I-1802, Office, Chief of Engineers, Washington, D.C.

Department of the Army 1993
Department of the Army. 1993. Slope Stability Manual, Engineer Manual EM II I0-I-1902, Office, Chief of Engineers, Washington, D.C.

Duncan and Others 1980
Duncan, J. M., Byrne, P., Wong, K. S., and Mabry, P. 1980. "Strength, Stress-Strain and Bulk Modulus Parameters for Finite Element Analysis of Stresses and Movements in Soil Masses," Report UCB/GT/80-0 1, University of California, Berkeley, CA

Duncan and Others 1984
Duncan, J. M., Seed, R. B., Wong, K. S., and Ozawa, Y. 1984. "FEADAM84: A Computer Program for Finite Element Analysis of Dams," Research Report SU/GT/84-03, Stanford University, Stanford, CA

Edris and Wright 1992
Edris, E. V., Jr. and Wright, S. G. 1992. "User's Guide: UTEY-AS3 Slope stability package," Technical Report GL-87- 1, Vol. IV, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Evans 1993
Evans, M.D. 1993 (May). Gravelly Soil Liquefaction, Proceedings, 1993 National Earthquake Conference -Earthquake Hazard Reduction in the Central and Eastern United States: A Time for Examination and Action, Memphis, pp. 57-66

Evans and Seed 1987
Evans, M.D., and Seed, H.B. 1987. "Undrained Cyclic Triaxial Testing of Gravels--The Effects of Membrane Compliance," Report No. UCB/EERC-87/08, University of California, Berkeley, CA

Federal Highway Administration 1978
Guidelines for Cone Penetration Test Performance and Design, Report No. FHWA-TS-78-209, Prepared for the US Department of Transportation by Dr. John H. Schmertmann, University of Florida

Fear and McRoberts 1995
Fear, L. and McRoberts, E. “Reconsideration of Initiation of Liquefaction in Sandy Soils,” ASCE Geotechnical Journal, Vol. 121, #3, March 1995

Finn 1990
Finn, W. D. L. 1990. "Analysis of Post-Liquefaction Deformations in Soil Structures" Proceeding H. Bolton Seed Memorial Symposium, Vol. 2, University of California, Berkeley, CA, May 10 and 11, 1990, pp. 291-311

Finn and Others 1994
Finn, W.D.L., Ledbetter, R.H. and Marcuson, W.F. 1994. Seismic Deformations in Embankments and Slopes, Proceedings Symposium on Developments in Geotechnical Engineering from Harvard to New Delhi 1936-1994, Bangkok, Thailand, edited by A.S. Balasubramaniam, and others, Balkema Publication, ISBN 90 5410 522 4, pp. 233-264

Finn and Others 1986
Finn, W. D. L., Yogendrakumar, M., Yoshida, N., and Yoshida, H. 1986. "TARA-3: A Program to Compute the Response of 2-D Embankments and Soil-Structure Interaction Systems to Seismic Loadings," Department of Civil Engineering, University of British Columbia, Vancouver, B.C.

Franklin and Chang 1977
Franklin, A. G. and Chang, F. K. 1977. "Earthquake Resistance of Earth and Rock-Fill Dams; Permanent Displacements of Earth Embankments by Newmark Sliding Block Analysis," Miscellaneous Paper S-71-17, Report 5, U. S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Gazetas and Uddin 1994
Gazetas, G. and Uddin, N. 1994 (Nov). Permanent Deformation on Preexisting Sliding Surfaces in Dams, Journal of Geotechnical Engineering, ASCE, Vol. 120, No. I 1, pp. 2041-2061

Houston and Others 1987
Houston, S. L., Houston, W. N., and Padilla, J. M. 1987. "Microcomputer-Aided Evaluation of Earthquake-Induced Permanent Slope Displacements," Microcomputers in Civil Engineering, Elsevier Science Publishing Co., Inc., Vol. 2, pp. 207-222

Hynes and Franklin 1984
Hynes, M. E. and Franklin, A. G. 1984. "Rationalizing the Seismic Coefficient Method," Miscellaneous Paper GL-84-13, US Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Idriss and Others 1992
Idriss, I.M. and Sun, J.I, 1992 (Nov). User's Manual for SHAKF-91-A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits, Center for Geotechnical Modeling, Dept. of Civil & Environmental Engineering, University of California, Davis, CA

Ishihara 1993
Ishihara, K. 1993. Liquefaction and Flow Failure During Earthquakes, 33rd Rankine Lecture, British Geotechnical Society, Geotechnique, Vol. 43, No. 3, pp. 351-415

Itasca Consulting Group 2002
Itasca Consulting Group, Inc. 2002, User's Guide for FLAC3D - Fast Lagrangian Analysis of Continua in 3 Dimensions, Version 2.1. Minneapolis

Koester 1992
Koester, J.P. 1992 (Dec). The Influence of Test Procedure on Correlation of Atterberg Limits With Liquefaction in Fine-Grained Soils, Geotechnical Testing Journal, ASTM, GTODJ, Vol. 15, No. 4, pp. 352-361

Koester 1994
Koester, J.P. 1994 (Jul). Pre-and Post-Liquefaction Behavior of Fine-Grained Soils, SF session paper No. 00347, presented at the Fifth US National Conference on Earthquake Engineering, EERI, Chicago, IL

Koester 1994
Koester, J.P. 1994 (Oct). The Influence of Fines Type and Content on Cyclic Strength, Geotechnical Special Publication No. 44, ASCE Convention Specialty Session on Ground Failures in Silty Soils

Kramer 1996
Kramer, S.L., 1996. Geotechnical Earthquake Engineering, Prentice-Hall, Upper Saddle River NJ, 653 pages

Liao and Whitman 1985
Liao, S. C. and Whitman, R. V. 1985. "Overburden Correction Factors For SPT in Sand," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 112, No. 3, pp 373-377

Liao and Others 1988
Liao, S., Veneziano, D., and Whitman, R.V., “Regression Models for Evaluating Liquefaction Probability,” ASCE Journal Geotechnical Engineering, Vol.114, #4, April 1988

Lysmer and Others 1973
Lysmer, J., Udaka, T., Tsai, C., and Seed, H. 1973. "FLUSH: A Computer Program for Approximate 3-D Analysis of Soil-Structure Interaction Problems," Report No. EERC 75-30, Earthquake Engineering Research Center, Berkeley, CA

Makdisi and Seed 1979
Makdisi, F.I., and Seed, H.B. 1979 (Jul). Simplified Procedure for Estimating Dam and Embankment Earthquake-Induced Deformations, ASCE Journal of the Geotechnical Engineering Division, Vol. 104, No. GT7, pp. 849-867

Marcuson and Others 1990
Marcuson, W.F., Hynes, M.E. and Franklin, A.G. 1990 (Aug). Evaluation and Use of Residual Shear Strength in Seismic Safety Analysis of Embankments, Earthquake Spectra, pp. 529-572

Mejia and Others 1982
Mejia, L.H. and Seed, H.B. and Lysmer, J.L. 1982 (Dec). Dynamic Analysis of Earth Dams in Three Dimensions, Journal of Geotechnical Engineering Division, ASCE, Vol. 108, No. 12, pp. 1586-1604

Mejia and Seed 1983
Mejia, L.H. and Seed, H.B. 1983 (Sep). Comparison of 2-D and 3-D Dynamic Analysis of Earth Dams, Journal of Geotechnical Engineering Division, ASCE, Vol. 109, No. I 1, pp. 1383-1398

Mitchell and Tseng 1990
Mitchell, J.K., and Tseng, D.J. 1990. Assessment of Liquefaction Potential by Cone Penetration Resistance, Proceedings of the H.B. Seed Memorial Symposium, Bi Tech Publishing, Vol. II, pp. 335-350

National Research Council 1985
National Research Council. 1985. Liquefaction of Soils During Earthquakes: National Academy Press, Washington, D.C., p. 240

Okamoto 1984
Oakamoto, S. 1984. Introduction to Earthquake Engineering, second edition, University of Tokyo Press, p. 629

Olsen 1994
Olsen, R.S. 1994. Normalization and Prediction of Geotechnical Properties Using the Cone Penetrometer Test (CPT), Technical Report GL-94-29, U.S. Army Corps of Engineers, Waterways Experiment Station, Vicksburg, MS

Olsen and Koester 1995
Olsen, R.S., and Koester, J.P. 1995 (Oct). "Prediction of Liquefaction Resistance Using the CPT," Proceedings, CPT ‘95 -International Conference on Cone Penetration Testing, Linkopping, Sweden

Prevost 1989
Prevost, J. 1989. DYNARD-A Computer Program for Nonlinear Seismic Site Response Analysis, XI Technical Report NCEER-89-0025, National Center for Earthquake Engineering Research, University of Buffalo

Robertson and Others
Robertson, P. K., Woeller, D. J. and Finn, W. D. L., “Seismic Cone Penetration Test for Evaluating Liquefaction Potential Under Cyclic Loading,” Canadian Geotechnical Journal, Vol. 29, pp. 686-695

Sarma 1975
Sarma, S. K. 1975. "Seismic Stability of Earth Dams and Embankments," Geotechnique, Vol. 25, No. 4, pp 743-761

Sarma 1979
Sarma, S. K. 1979. Response and Stability of Earth Dams During Strong Earthquakes, Miscellaneous Paper GL-79-13, USAE Waterways Experiment Station, Vicksburg, MS

Schnabel and Others 1972
Schnabel, P. B., Lysmer, J., and Seed, H. B. 1972. "SHAKE, A Computer Program for Earthquake I Response Analysis of Horizontally Layered Sites," Report EERC 72-12, Earthquake Engineering Research Center, University of California, Berkeley, CA

Seed 1979
Seed, H. B. 1979. "19th Rankine Lecture: Considerations in the Earthquake Resistant Design of Earth and Rockfill Dams," Geotechnique, Vol. 29, No. 3, pp 215-263

Seed 1987
Seed, H. B. 1987 (Aug). Design Problems in Soil Liquefaction, Journal, Geotechnical Engineering Division, ASCE, Vol. 11 3, No. 8, pp.827-845

Seed and Idriss 1970
Seed, H. B., and Idriss, I.M. 1970. Soil Moduli and Damping Factors for Dynamic Response Analysis, Report EERC-70/1 0, Earthquake Engineering Research Center, Berkeley, CA

Seed and Idriss 1982
Seed, H. B., and Idriss, I. M. 1982. Ground Motions and Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute, 499 14th Street, Oakland, CA, pp. 134

Seed and Others 1983
Seed, H. B., Idriss, I. M., and Arango, I. 1983. "Evaluation of Liquefaction Potential Using Field Performance Data." Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. 109, No. 3, pp 458-482

Seed and Others 1975
Seed, H. B., Lee, K. L., Idriss, I. M., and Makdisi, F. I. 1975. "The Slides in the San Fernando Dams During The Earthquake of February 9, 1971," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 101, No. GT7, pp 651-688

Seed and Others 1985
Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. 1985. "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 111 No. 12, pp 861-878

Seed and Others 1986
Seed, H. B., Wong, R., Idriss, I. M., and Tokimatsu, K. 1986. Moduli and Damping Factors for Dynamic Analysis of Cohesionless Soils, Journal, Geotechnical Engineering Division, ASCE, Vol. 112, No. 11, pp. 1016-1032

Seed and Harder 1990
Seed, R. B. and Harder, L. F. 1990. SPT-Based Analysis of Cyclic Pore Pressure Generation and Undrained Strength, Proceedings of the H. B. Seed Memorial Symposium, Bi Tech Publishing, Vol. 11, pp. 351-376

Sherard and Dunnigan 1985
Sherard and Dunnigan 1985. Filters and Leakage Control in Embankment Dams, Proceedings, Symposium on Seepage and Leakage from Dams and Impoundments, R. L. Volpe and W. E. Kelly, eds, ASCE, pp. 1-30, reprinted in Embankment Dams-James L. Sherard Contributions, S. Singh, ed, ASCE Geotechnical Special Publication No. 32, 1992

Stark and Mesri 1994
Stark, T. D. and Mesri, G. 1994. Undrained Shear Strength of Liquefied Sands for Stability Analysis, Journal Geotechnical Engineering Division, ASCE, Vol. 118, No. 11, pp. 1727-1747, and Field Case Histories

Stark and Olson 1995
Stark, T.D. and Olson, S.M. 1995. Liquefaction Resistance Using CPT Journal Geotechnical Engineering Division, ASCE, Vol. 121, No. 12, pp. 856-869

Sun and Others 1988
Sun, J. I., Golesorkhi, R., and Seed, H. B. 1988. Dynamic Moduli and Damping Ratios for Cohesive Soils, Report EERC-88/15, Earthquake Engineering Research Center, Berkeley, CA

Sykora 1987
Sykora, D. W. 1987. "Examination of Existing Shear Wave Velocity and Shear Modulus Correlations in Soils," U.S. Army Corps of Engineers, Waterways Experiment Station, Miscellaneous Paper GL-87-22

Tokimatsu and Seed 1988
Tokimatsu, K. and Seed, H. B. 1988. "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 113, No. 8, pp 861-878

Tokimatsu and Yoshimi 1983
Tokimatsu, K. and Yoshimi, Y. 1983. "Empirical Correlation of Soil Liquefaction Based on SPT N-value and Fines Content," Soils and Foundations, Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 15, No. 4, pp 81-92

Velesa and Others 1994
Velesa, J. E., Traubenik, M., Egan, J., and Kaneshiro, M., A Practical Perspective on Liquefaction of Gravels," Proceedings of Geotechnical Engineering Division Session on Ground Failures under Seismic Conditions, ASCE Special Publication AS 44, October, 1994

Yule and Others 1994
Yule, D. E., Wahl, R. E., Sykora, D. W. and Wallace, D. C. 1994 (July). WESHAKE for Personal Computers (Version 1.2), USE Geotechnical Earthquake Engineering Software, Report 2, USAE Waterways Experiment Station Instruction Report GL-94, Vicksburg, MS

Zen and Higuchi 1984

Zen, K., and Higuchi, Y. 1984. Prediction of Vibratory Shear Modulus and Damping Ratio for Cohesive Soils, Proc., 8th International Conference on Earthquake Engineering, San Francisco, CA, Vol. 3, pp. 23-30

APPENDIX C
GLOSSARY

Active Fault - A fault, which, because of its present tectonic setting, can undergo movement from time to time in the immediate geologic future.

> ***Comment:*** For the purpose of earthquake engineering application, it is a seismogenic or earthquake fault and is distinct from other types of faults such as landslides, ice thrusting, groundwater withdrawal effects, etc.

Attenuation - A decrease in amplitude of the seismic waves with distance due to geometric spreading, energy absorption, and scattering.

Bedrock - Any sedimentary, igneous, or metamorphic material represented as a unit in geology; being a sound and solid mass, layer, or ledge of mineral matter; and with shear wave threshold velocities greater than 2500 feet/second.

Bedrock Motion Parameters - Numerical values representing vibratory ground motion, such as particle acceleration, velocity, and displacement, frequency content, predominant period, spectral intensity, and a duration which define a design earthquake. (These may also be used in a more general sense for ground motion.)

Body Waves - Waves propagated in the interior of the earth, i.e., the compression (P) and shear (S) waves of an earthquake.

Capable Fault - An active fault that is judged capable of producing macroearthquakes and exhibits one or more of the following characteristics:

- Movement at or near the ground surface at least once within the past 35,000 years.
- Macroseismicity (3.5 magnitude or greater) instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.
- A structural relationship to a capable fault such that movement on one fault could be reasonably expected to cause movement on the other.
- Established patterns of microseismicity which define a fault, with historic macroseismicity that can reasonably be associated with the fault.

Critical Damping - the minimum amount of damping that prevents free oscillatory vibration.

Cyclic Mobility - A phenomenon in which a cohesionless soil loses shear strength during earthquake ground vibrations and acquires a degree of mobility sufficient to permit intermittent movement up to several feet as contrasted to liquefaction where continuous movements of several hundred feet are possible.

Damping - Resistance which reduces vibrations by energy absorption. There are different types of damping such as viscous, Coulomb, and geometric damping.

Damping Ratio - the ratio of the actual damping to the critical damping.

Duration of Strong Ground Motion - The "bracketed duration" or the time interval between the first and last acceleration peaks that are equal to or greater than 0.05g.

Comment: There are other definitions that, if used, should be clearly defined.

Epicenter - The point on the earth's surface located vertically above the point where the first rupture and the first earthquake motion occur.

Fault - A fracture or fracture zone in the earth along which there has been displacement of the two sides relative to one another and which is parallel to the fracture.

Hypocenter - The location where the slip responsible for an earthquake originates; the focus of an earthquake.

Intensity - A numerical index describing the effects of an earthquake on man, manmade structures,or other features of the earth's surface.

***Comment*:** There are several intensity scales, but the most commonly used scale in the U.S. is the Modified Mercalli Intensity scale (Richter, C. F., Elementary Seismology, W.H. Freeman and Company, San Francisco, 1958). Historically, the epicentral intensity (I_o) has been used to designate the size of an earthquake due to the lack of instrumental data.

Limits of Near-Field Motion:

Richter Magnitude, M	Modified Mercalli, To Maximum Intensity	Radius of Near Field, Km
5.0	VI	5
5.5	VII	15
6.0	VIII	25
6.5	IX	35
7.0	X	40
7.5	XI	45

Comment: These limits of near-field motions are for earthquakes in the Western United States. There are no established limits of near-field motions in the Eastern United States.

Liquefaction - Denotes a condition where a soil will undergo continued deformation at a constant low residual stress or with low residual resistance due to the buildup and maintenance of high pore water pressures, which reduces the effective confining pressure to a very low value. Pore pressure buildup leading to liquefaction may be due either to static or cyclic stress applications and the possibility of its occurrence will depend on the void ratio or relative density of a cohesionless soil and the confining pressure.

Magnitude - A measure of the earthquake size related to the strain energy released by it, based on the displacement amplitude, period of the seismic waves, and distance from the earthquake epicenter.

***Comment*:** There are several magnitude designations, some of which are listed below. Therefore, the type of magnitude used and how it was determined should be clearly designated.

- **Body Wave Magnitude (m_b) -** The magnitude of an earthquake measured as the common logarithm of the maximum displacement amplitude (microns) and period (seconds) of the body waves.

  Comment: Developed to measure the magnitude of deep focus earthquakes, which do not ordinarily set up detectable surface waves with long periods. Magnitudes can be assigned from any suitable instrument whose constants are known. The body waves can be measured from either the first few cycles of the compression waves (mb) or the 1-second period surface waves ($M_{b,\ g}$).

- **Richter or Local Magnitude (M_L) -** The magnitude of an earthquake measured as a common logarithm of the displacement amplitude, in microns, of a standard Wood Anderson seismograph located on firm ground 100 km from the epicenter and having a magnification of 2800, a natural period 0.8 second and a damping coefficient of 80 percent.

  ***Comment*:** Empirical charts and tables are available to correct for epicentral distances from 100 km, other types of seismographs, and various conditions of the ground. The correction charts and the definition itself apply strictly only to earthquakes having focal depths smaller than about 30 km. The correction charts are relatively accurate up to epicentral distances of about 600 km. These correction charts are site dependent and have to be developed for each recording site. This is commonly used up to magnitude ML equal to 6.5. Above the M_S is used to define magnitude.

- **Surface Wave Magnitude (M_S) -** The magnitude of an earthquake measured as the common logarithm of the resultant of the maximum mutually perpendicular horizontal displacement amplitudes, in microns, of the 20-second period surface waves.

  ***Comment*:** Developed to measure the magnitude of shallow focus earthquakes at relatively long distances. Magnitudes can be assigned from any suitable instrument whose constants are known.

Maximum Credible Earthquake (MCE) - The earthquake(s) associated with specific seismotectonic structures, source areas, or provinces that would cause the most severe vibratory ground motion or foundation dislocation capable of being produced at the site under the currently known tectonic framework. It is determined by judgment based on all known regional and local geological and seismological data.

> ***Comment:*** In general, each seismotectonic province, source area or structure considered will have an MCE associated with it. In the present context these multiple MCEs are used to define the controlling MCE for the site of interest.

Maximum Design Earthquake (MDE) - A postulated seismic event, specified in terms of specific bedrock motion parameters at a given site, which is used to evaluate the seismic resistance of manmade structures or other features at the site.

> ***Comment*:** The maximum design earthquake is the earthquake that is used to evaluate the seismic resistance of the structure and is usually equated with the controlling MCE. However, where the failure of the dam presents no hazard to life, a lesser earthquake may

be justified, provided there are cost benefits and the risk of property damage is acceptable.

Operating Basis Earthquake (OBE) - The earthquake(s) for which the structure is designed to resist and remain operations. It reflects the level of earthquake protection desired for operational or economic reasons and may be determined on a probabilistic basis considering the regional and local geology and seismology.

Predominant Period - The period(s) at which maximum spectral amplitudes are shown on response spectra. Normally, acceleration response spectra are used to determine the predominant period(s) of the earthquake ground motion.

Response Spectrum - A plot of the maximum values of acceleration, velocity, and/or displacement response of an infinite series of single-degree-of-freedom systems subjected to a time-history of earthquake ground motion. The maximum response values are expressed as a function of natural period for a given damping.

Comment: The response spectrum acceleration, velocity, and displacement values may be calculated from each other assuming a sinusoidal relationship between them. When calculated in this manner, these are sometimes referred to as pseudoacceleration, pseudovelocity, or pseudodisplacement response spectrum values.

Scaling - An adjustment to an earthquake time-history or response spectrum where the amplitude of acceleration, velocity and/or displacement is increased or decreased, usually without change to the frequency content of the ground motion.

Comment: There are other methods to scale earthquakes and, if used, they should be clearly defined. The earthquake time-history or response spectrum can be scaled based on ground motion parameters of peak acceleration, peak velocity, peak displacement, spectrum intensity, or other appropriate parameters.

Seismic Moment (M_0) - A measure of the earthquake size containing information on the rigidity of the elastic medium in the source region, average dislocation and area of faulting. It determines the amplitude of the long-period level of the spectrum of ground motion. It is calculated as:

M_O = Shear Modulus of Faulted Rock (Dynes/cm^2)
x Length of Fault Rupture Zone (cm)
x Width of Fault (cm)
x Displacement of Fault (cm)

Seismotectonic Province - A geologic area characterized by similarity of geologic structure and tectonic and seismic history.

Seismotectonic Source Area - An area of known or potential seismic activity that may lack a specific identifiable seismotectonic structure.

Comment: Several source areas may occur within a seismotectonic province.

Seismotectonic Structure - An identifiable dislocation or distortion within the earth's crust resulting form recent tectonic activity or revealed by seismologic or geologic evidence.

Smooth Response Spectrum - A response spectrum devoid of sharp peaks and valleys that specifies the amplitude of the spectral acceleration, velocity and/or displacement to be used in the analyses of the structure.

Spectrum Intensity - The integral of the pseudovelocity response spectrum taken over the range of structural vibration periods from 0.1 to 2.5 seconds.

Surface Waves - Waves that travel along or near the surface and include Rayleigh (S_v) and Love (S_H) Waves of an earthquake.

Synthetic Earthquake - Earthquake time history records developed from mathematical models that use white noise, filtered white noise and stationary and non-stationary filtered noise, or theoretical seismic source models of failure in the fault zone.

> ***Comment:*** White noise is random energy containing all frequency components in equal proportions. Stationary white noise is random energy with statistical characteristics that do not vary with time.