Federal Guidelines for Earthquake Analyses and Design of Dams

FEDERAL EMERGENCY MANAGEMENT AGENCY

Prepared By
Interagency Committee on Dam Safety

Design Earthquake Task Group Subcommittee

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FEDERAL GUIDELINES
FOR EARTHQUAKE ANALYSES
AND DESIGN OF DAMS

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Design Earthquake Task Group
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FEDERAL GUIDELINES

FOR EARTHQUAKE ANALYSES AND DESIGN OF DAMS

A. Background

An ad hoc interagency committee on dam safety of the Federal Coordinating Council for Science, Engineering, and Technology prepared "Federal Guidelines for Dam Safety" which was published June 25, 1979. Preparation of these guidelines was directed by a Presidential Memorandum, dated April 23, 1977, to several Federal departments and agencies. A second Presidential Memorandum, dated October 4, 1979, directed Federal agencies to adopt and implement the "Federal Guidelines for Dam Safety." Those guidelines were part of a national effort to enhance dam safety, and they outlined the technical activities that agency management needs to undertake to ensure safe design of dams. The Interagency Committee on Dam Safety (ICODS) has established the interagency task group on design earthquakes to provide more complete guidelines for earthquake analyses and design of dams.

B. Purpose

The purpose of these guidelines is to develop some consistency in handling the earthquake analyses and design among the various Federal agencies involved in the planning, design, construction, operation, maintenance, and regulation of dams. They are intended to be used as general guides and are not to be considered as standards. It is recognized that the various agencies have differences in mission and diversified location which make agency independence desirable. It is further recognized that earthquake engineering is in the developmental stage and flexibility is desirable. While the content of these guidelines generally reflects current practices, it will be necessary to make periodic revisions, additions, deletions, etc., to maintain currency with the state of the art in earthquake engineering.

The importance, extent, and method of earthquake analysis and design depend on the seismicity of a region or site-specific considerations, the type of structures involved, and the consequences of failure. In some cases, for example, low hazard dams or dams in zones of low seismicity, extensive investigations and seismicity evaluations or detailed analyses may not be required. Previous studies, performance, and experience of existing structures suggest that concrete gravity dams located in regions of low-to-moderate seismicity and adequately designed to withstand the appropriate static forces are competent to also withstand the earthquake forces. The same may be true of certain well-constructed fill-type dams placed on good foundations, for example, well-compacted rolled fill dams on rock foundations. In other cases where the hazard is high or the seismic conditions are severe, extensive investigations and design analyses may be required.

The intent of these guidelines is to outline certain appropriate procedures and methods of analyses which can be used; each agency is expected to supplement these general guidelines with detailed procedures, as necessary, to accomplish their specific mission.
C. Scope

This document contains: (a) general guidelines for selecting design earthquakes and (b) methods of seismic analyses which ensure the safety of existing dams and provide for the safe design of proposed dams. In general, guidelines for proposed dams are to be applied to existing dams; however, it is recognized that special considerations may sometimes be required for existing dams. The following subjects are discussed in the guidelines:

1. Factors to Consider in the Selection of Design Earthquakes
2. Selection of Design Earthquakes
3. Characterization of Ground Motion Expected at the Site
4. Earthquake Analyses
5. Evaluation of Structural Adequacy for Earthquake Loading

A flowchart depicting the general process for selecting design earthquakes and methods of seismic analyses is shown in figure 1. The detail of the steps shown in the chart will depend on the needs discussed above under purpose.

Terms used in the guidelines are defined in Appendix A.

The use of references has been minimized except in a few cases where they are essential for clarity. Appendix B is a bibliography of selected publications, reference to which should assist the use of these guidelines. The references used in the text are contained in the bibliography.

FACTORS TO CONSIDER IN
SELECTION OF DESIGN EARTHQUAKES
- REGIONAL TECTONIC SETTING
- SEISMIC HISTORY
- SEISMOTECTONIC STRUCTURES
- LOCAL OR SITE GEOLOGY
- SEismic ATTENUATION
- RESERVOIR INDUCED SEISMICITY

SELECTION OF DESIGN EARTHQUAKES
- MAXIMUM CREDIBLE EARTHQUAKE
- MAXIMUM DESIGN EARTHQUAKE
- OPERATING BASIS EARTHQUAKE

DETERMINATION OF GROUND MOTION
FOR THE DESIGN EARTHQUAKES
- PEAK ACCELERATION, VELOCITY, AND DISPLACEMENT
- DURATION
- ACCELERATION TIME-HISTORIES
- RESPONSE SPECTRA

REQUIREMENT FOR EARTHQUAKE
ANALYSIS
- SEISMICITY AND GROUND MOTIONS
- FOUNDATION CONDITIONS
- TYPE OF DAM
- CONSTRUCTION METHODS
- MATERIAL PROPERTIES
- PAST EXPERIENCE

NO
- DOCUMENTATION
- EVALUATION

YES
- METHODS OF ANALYSES

CONCRETE DAMS
- PSEUDOSTATIC METHOD
- DYNAMIC ANALYSES METHODS
- RESPONSE SPECTRUM
- TIME-HISTORY

EMBANKMENT DAMS
- LIQUEFACTION EVALUATION
- PSEUDOSTATIC METHOD
- NEWMARK METHOD
- FINITE ELEMENT METHOD

EVALUATION OF STRUCTURAL
ADEQUACY
- EVALUATION OF ANALYSES RESULTS
- PAST EXPERIENCE OF DAMS
- CONSEQUENCES OF FAILURE

FIGURE 1
FLOWCHART DEPICTING STEPS
FOR EARTHQUAKE ANALYSES
AND DESIGN OF DAMS
D. Factors to Consider in the Selection of Design Earthquakes

The selection of design earthquakes should take into consideration the geologic and seismic conditions in the region of the intended site, the consequences of failure, the hazards associated with the facility, and the physical requirements of the proposed engineering structure. The following factors, ranging from regional to site specific, should be considered in the selection of design earthquakes. These factors are evaluated in the context of the current tectonic framework. The U.S. Geological Survey Professional Paper 1118, entitled "Procedures for Estimating Earthquake Ground Motions," 1980, by Walter W. Hays, is an excellent source document for more detailed information.

Locating the project area on published information of source zones and seismic risk areas will provide an initial indication of the level of anticipated seismicity and a starting point for a more comprehensive evaluation of site seismicity.

1. Regional Tectonic Setting

Information on regional geology and late Cenozoic tectonic history is needed to assess the long-term seismic potential of the site as part of the design earthquake selection process. This should include:

a. Identification of the physiographic province(s) within which the project is located, e.g., Basin and Range, Lower Mississippi Alluvial Plain, Appalachian Highlands, etc.

b. Location of the project area within its particular seismotectonic province and relating its province with adjacent provinces.

c. Description and identification of the principle tectonic events and elements which characterize the most recent tectonic activity.

d. Description of the regional geologic rock units and structure for consideration when evaluating seismic energy transmission, effects, and duration.

2. Seismic History

Historical seismological information exists primarily within governmental and academic references and the media record. The National Earthquake Information Service of the U.S. Geological Survey in Golden, Colorado, compiles source and magnitude sources will provide information for a seismicity evaluation of the site. Steps to follow and data to collect are:

a. Define the limits of the historical data search area considering the boundaries of seismotectonic sources and provinces of potential significance to the site.

b. Conduct a search of the historic seismological record to determine the location, time of occurrence, and characteristics (e.g., earthquake source parameters and wave propagation effects) of previous seismic events within the area of interest. Consideration should be given to the completeness of the record and the location accuracy of individual events.

c. Conduct a search of the instrument record to determine event locations, frequency of occurrence of various event levels, and characteristics of measured seismic activity within the area of interest. Additional analysis of field monitoring may be useful in areas where historic data are lacking.

d. Array the data for statistical analysis to develop frequency, magnitude, and recurrence relationships. Consider the relationships between regional province-wide recurrence and recurrence for individual faults or sources.

e. Display the developed seismic information by appropriate tabulations and map presentations.

f. Review the historical records to develop information relating to the surface effects of past earthquake events.

3. Seismotectonic Structures

Of primary importance to the earthquake analysis is the identification and characterization of faults and other seismotectonic structures which may be the sources of the design earthquakes. This effort may require literature searches in the fields of tectonics, structure, seismology, and quaternary geology, the documented seismic record, analyses of remote sensing imagery, geophysical data, and field mapping and exploration programs to verify and further define documented or suspected structures. These activities should:

a. Document the type, location, areal extent, displacement, and age of displacement of faults in the area of interest.

b. Characterize located faults by sense of movement, and degree and age of activity. Potential for surface faulting in the immediate vicinity of the dam should be addressed.

c. Characterize other seismotectonic structures which are not correlated with surface features via the geophysical record, regional geologic data, and seismicity characteristics.

d. Analyze recent historic earthquakes or data from micro-earthquake monitoring to gain information on current stress conditions.

Relationships may be established between identified faults and other seismotectonic features and the frequency of occurrence and magnitude recorded from previous earthquake events by
a. Integrating the data for identified faults and other seismotectonic structures with the project area seismicity data to develop event/source correlations.

b. Developing and evaluating conceptual models to explain significant earthquake events which do not correlate with identified faults or other seismotectonic structures.

4. Local or Site Geology

Site-specific geologic information should be developed to determine the hazards from potential fault surface ruptures and to evaluate material and structural responses to ground motions from the expected range of seismic events. This information may be obtained through the evaluation of published geologic reports, field observations, and specific site investigations including geologic mapping, trenching, drilling, material sampling and testing, which should:

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

b. Define the character, lateral extent, and thickness of rock units at the site area.

c. Define the structural geology of the site including rock unit attitudes, faults and joint systems, folding and intrusive bodies.

d. Define the age and activity of faults in the dam and reservoir area.

e. Define the geohydrology of the site area including water table conditions, soil and rock transmissivity coefficients, and recharge areas.

f. Identify existing and potential ground failure and subsidence, including rock and soil stability, dispersive soil conditions, and soil units with characteristics for potential liquefaction.

g. Evaluate flooding potential due to seiche action by examination of reservoir shape, topography, and slope stability conditions.

5. Seismic Attenuation

To complete the analysis of seismic potential, the energy attenuation factors should be developed, so the energy transfer potential between event sources and the proposed structure can be evaluated. Because of the frequent inhomogeneity of areal and cross-sectional geology, the attenuation assessment is difficult, and to a large extent is based upon historical, empirical, and theoretical data. The following procedures are most frequently used:

a. Strong ground motion data, where available, should be evaluated to develop acceleration-attenuation relationships. Families of curves have been developed for areas of the U.S. where strong motion data is available.

b. Isoseismal maps constructed from historical earthquake data provide regional intensity attenuation data when strong motion information is unavailable. These maps have been produced from many of the high magnitude events in the U.S. The maps must be used with concurrent consideration for the geology of the site to be most effective. The maps have illustrated significant differences in intensity attenuation in the U.S. between areas east of the Rocky Mountains versus those west of the mountains. Intensity attenuation is much slower in the area east of the mountains.

6. Reservoir-Induced Seismicity

Reservoir-induced seismicity should be considered in establishing seismic loadings for high dams if the proposed reservoir contains active faults within its hydraulic regime and if the regional and local geology and seismic record within the reservoir's hydraulic regime may be judged to indicate potential for reservoir-induced seismicity. If all the faults within a reservoir are considered tectonically inactive, the possibility of reservoir-induced seismicity should not be totally ruled out. The potential should still be considered to exist if the local and regional geology and seismicity suggest that the area is subject to reservoir-induced seismicity. The magnitude of reservoir-induced earthquakes is established using the guidelines for likelihood of occurrence similar to those used for the design earthquakes.

E. Selection of Design Earthquakes

When the evaluation of the earthquake factors described in section D is completed, the maximum design earthquake (MDE) and the operating basis earthquake (OBE) are selected on the basis of an integrated evaluation of the earthquake factors. The MDE is the largest earthquake used in the seismic analysis of the dam and is generally equated to the controlling maximum credible earthquake (MCE) for the site. The OBE, usually smaller than the MDE, represents the maximum level of ground shaking that can be expected to occur at the site during the economic life of the dam. It may not be possible to show that all possible tectonic features have been discovered. Based on investigations, gaps of information may exist. If so, conservatism may be desirable dependent upon the potential hazards associated with the dam.

1. Maximum Credible Earthquakes

The first part of the investigation for selecting the MDE is to estimate the hypothetical MCE for each potential earthquake source, judged to have a significant influence on the site, from the information developed in section D. The hypothetical MCE for each
seismotectonic structure or source area within the region examined is defined by magnitude and or intensity, epicentral distance and focal depth. These MCEs are candidates for the controlling MCE.

2. Controlling Maximum Credible Earthquake

The second part of the investigation is to select the controlling MCE for the site as follows:

a. Select the most conservative distance from each seismic source to the site.

b. For each candidate MCE select strong motion records of earthquakes which have similar source and propagation path properties and were recorded on a foundation similar to that of the structure or, if these site-matched records are not available, attenuate the epicentral ground motion parameters or MM intensity to the site using one or more applicable attenuation relationships.

c. Select the controlling MCE based on the most severe ground motion parameters estimated for the site. There may be more than one controlling MCE because of the frequency characteristics on the dam and its components.

3. Maximum Design Earthquake

The final selection of the MDE considers whether or not the dam must be capable of resisting the controlling MCE, which is a "worst case" situation. Usually, the MDE is equated with the controlling MCE. However, where the failure of the dam presents no hazard to life, a lesser earthquake for the MDE may be justified providing there are cost benefits and the risk of property damage is acceptable.

4. Operating Basis Earthquake

The second level of design earthquake, the OBE, represents the maximum level of ground shaking that can be expected to occur at the site during the economic life of the project, usually 100 years for dams. It reflects the desired level of protection for the project from earthquake-induced structural and mechanical damage and loss of service during the project's economic life, or remaining economic life for existing dams. The OBE should be based on a probabilistic analysis which accounts for the time element involved in the definition of the OBE. A probabilistic analysis involves developing a magnitude-frequency or epicentral intensity-frequency (recurrence) relationship for each seismic source; projecting the recurrence information from regional information and past data into forecasts concerning future occurrence; attenuating the severity parameter, usually either peak ground acceleration or MM intensity, to the site; determining the controlling recurrence relationship for the site; and finally, selecting the design level of earthquake based upon an acceptable probability of exceedence and the project's exposure period selected for the design.

7. Characterization of Ground Motion Expected at the Site

The characteristics of the ground motion expected at the site are functions of the earthquake source mechanism, the epicentral distance, and the geometry and physical properties of the geologic structures traversed by the body and surface waves as they propagate from the source to the site. Based on this myriad of geologic and tectonic influences, seismograms recorded at all distances from the epicentral are very variable and difficult to interpret. They are especially complex in the near field where the ground motions are strongly influenced by the dynamics of the fault rupture, and where the source properties become more important in defining the ground motion than the site properties. At distances of greater than a few fault rupture widths from the source, site conditions begin to have a greater influence on the ground motion. Seismic waves traveling from rock to the ground surface through surficial deposits will be modified by a factor which is a function of the strength of the seismic wave motion and the depth, geologic structure, and dynamic properties of the soils. The inclination of the soil layers and bedrock can influence reflection and refraction processes; consequently, the complexity of the waves transmitted to the ground surface. Topography also affects the ground motion in that peaks or butting outcrops may significantly amplify the seismic parameters. The site parameters affecting ground motion at a site are not fully understood. These parameters deal with the physics of the earthquake source and the propagation of the released energy to the site. All current procedures for characterising ground motion are primarily empirical, an awareness of the state of the art must be maintained and incorporated in the characterisations.

Ground motion can be described in a number of ways for use in engineering calculations for evaluating the effects of earthquake ground motion on structures. Data used to develop ground motion are site dependent or site independent. Site-dependent data are obtained by using site-matched ground motion records or analytically determined ground response of the site. Site-independent data are standardised characterisation parameters obtained from statistical analyses of existing earthquake records chosen without the site-matched requirement. Although site-dependent data are preferable, sufficient site data may not be available. Therefore, it may be necessary to combine site-dependent data with site-independent data. Formats used to characterise ground motion are described below.

1. Peak Ground Motion Parameters

Ground motion can be characterized by peak values of ground acceleration, velocity, and displacement. The values are used
3. Graphically represented by plotting peak spectral acceleration, the concept of response spectra has evolved from attempts to describe the complicated nature of earthquake-induced ground motion and the complex transient response of structures. A response spectrum represents the peak response to a ground motion of an ensemble of single-degree-of-freedom systems having viscous damping. The response spectra can be graphically represented by plotting peak spectral acceleration, duration, and velocity against period or frequency. The response spectra of the ground motion at the site can be defined by using site-independent or site-dependent procedures.

a. Site-Independent Response Spectra

Site-independent response spectra are based on the use of standard spectrum shapes. The standard spectrum shapes are site-independent because they are based on earthquake ground motion records from a wide range of geologic, seismological, and local site conditions. For example, the ground motion records represent different earthquake magnitudes, soil (shallow and deep) sites, rock (various geological types) sites, different distances from the epicenter, etc.

There have been several studies published defining site-independent response spectra. The basic differences in these studies have been in the number of earthquake records used and the statistical treatment of the records.

Before using published site-independent response spectra, a determination should be made that they are applicable to the site. In order to do this, thorough evaluations should be made on how they were developed to ensure proper and consistent use. For example, some published site-independent response spectra shapes correspond to the average of the earthquake ground motion records used to develop the shape, while some are based on the 84th percentile shape. Use of these spectra without understanding their development can lead to improper application.

Additional studies can be performed to determine site-independent response spectra, but published studies should be evaluated to avoid unnecessary work. The published studies cover a broad range of conditions. If the published site-independent response spectra were not applicable, then development of site-dependent response spectra would be the best approach to pursue.

b. Site-Dependent Response Spectra

Site-dependent response spectra can be developed for site specific conditions using earthquake ground motion records which were recorded under similar site-matched conditions. The ability to develop site-dependent spectra is based on finding relevant ground motion records which permit a meaningful statistical evaluation.

The site-dependent characteristics which should be considered are the foundation conditions (soil and rock, deep or shallow soil, rock type, etc.), magnitude of the
design earthquake, distance of the site from epicenter of the earthquake, attenuation characteristics of the earthquake to the site, and the source mechanisms of the earthquake (type of fault, depth, etc.). The current library of strong-motion records, in most cases, is not sufficient to completely consider all of the above factors, particularly the attenuation characteristics and the source mechanisms. Site-dependent response spectra have been developed for various conditions including foundation conditions, earthquake size, and attenuation effects.

Site-dependent spectra will generally produce ground-motion parameters that correspond better with those expected on the basis of the seismological and geologic conditions at the site. The limiting factor is that source mechanisms and the attenuation characteristics to the site may not be fully represented by the available data. Also, the number of relevant ground motion records may not be sufficient to perform a statistical evaluation. Therefore, care should be exercised in developing a site-dependent response spectra.

4. Acceleration Time-Histories

The acceleration time-histories used in the analysis are usually selected in conjunction with the site-independent or site-dependent response spectra. The time-histories may be real earthquake records or synthetic (mathematical models) records whose response spectra closely represent the site-independent or site-dependent response spectra. The response spectra from the time-histories may envelope site response spectra or they may represent an average; this depends on how the site response spectra were developed and the amount of conservatism desired in defining the time-histories.

The synthetic earthquake records are developed from mathematical models that use white noise, filtered white noise, and stationary and nonstationary filtered white noise or theoretical seismic source models of failure in the fault zone to generate time-histories.

Selection of real earthquake records to represent the site acceleration time-histories is usually preferable to using synthetic earthquakes. Sometimes, this is not possible due to lack of recorded data.

Actual and synthetic earthquake time-histories may be modified and/or combined to form time-histories which contain desired ground motion characteristics of peak acceleration, duration, and frequency content. Accelerations can be increased or decreased by scaling the amplitudes, i.e., by multiplying by the ratio of the peak acceleration desired to the peak unmodified acceleration. Frequency content can be modified by extending or compressing the time scale or by adding portions of other records that contain the desired frequencies. The predominant period can be changed by adjusting the time scale by the ratio of the predominant period desired to the predominant period of the unmodified record. The duration of the record can be extended by repeating significant portions or it can be reduced by truncating. The modified time-histories should be critically reviewed to ensure that the desired characteristics are incorporated and reasonably represent an expected earthquake. Modified ground motion records are the most commonly used sources of ground motion time-histories for dynamic analyses.

5. Uncertainties in Ground Motion Design Values

Definition of the uncertainty in ground-motion design values is a complex task depending upon the seismotectonic province where the earthquake occurs and the physical parameters controlling the source, path, and local ground-response effects. The complexity of the problem arises from the unknown or poorly known statistical distribution of many of the physical parameters and not knowing the precise way to combine the uncertainties of the individual physical parameters of the system, even if the statistical distribution for each parameter is well known. The difficulty is a consequence of the short and incomplete world seismicity record and the lack of adequate geologic data to define earthquake potential at a site. The quality of the data base is one of the most important factors leading to the capability for precise specifications of earthquake ground motions. Empirical procedures currently used will be refined and extended in the future. Meanwhile, parametric studies can provide the bounds within which the evaluation of structural adequacy can be made.

G. Earthquake Analyses

1. Need for and Extent of Analyses

The extent and type of analysis required for the seismic design of a dam depends on the following factors: the seismicity of the region, foundation conditions at the site of the dam and impounded reservoir, type and height of dam, construction methods and as-built material properties, and sound engineering judgment based on past experiences.

Experience has shown that well-engineered dams which are designed to resist the static forces with generally accepted factors of safety and which are well constructed have the ability to resist strong earthquake ground motions. Consequently, all dams or their appurtenant structures will not necessarily require additional earthquake design studies; however, some analyses to document that additional studies are not needed should be performed.
If analyses are judged to be required, the analyses should begin with the simplest appropriate methods and conservative assumptions. If, from this first analysis, the structure is judged able to resist earthquake forces, then that analysis should be sufficient. If not, progressively more detailed analyses should be performed and the structure designed accordingly. Regardless of the methods of analyses, the final evaluation of the seismic safety of the dam should be based on engineering judgment and past experience and not just on the numerical results of the analyses.

2. Concrete Dams

a. Safety Concerns

(1) Types of Instability

Safety concerns for concrete dams subjected to earthquakes involve two types of instability: sliding on a plane of weakness and excessive cracking of the concrete. For gravity 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 can occur through the failure of the abutment support. Sliding instability for a buttress dam can occur at or below the foundation-dam interface. Instability due to excessive cracking of the concrete can occur in the upper half of a gravity and arch dam and at any location of a buttress dam, especially at points of abrupt changes in the cross-section.

(2) Foundation Importance

Of the two types of possible instability discussed above, foundation and/or abutment 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 has to be estimated from exploratory borings and tests on only a small fraction of the material present. Key zones of weakness are difficult to detect.

(3) Field Performance

No major concrete dam is known to have failed due to earthquake-induced ground motion, although some have experienced strong ground motion. The field performance of concrete dams during earthquakes is discussed in section 4.

b. Defensive Design Measures

The application of defensive design measures is the most dependable approach to alleviate safety concerns. Defensive design measures for concrete dams include:

- Sufficient foundation and abutment exploration, material testing, and strengthening, if necessary, to ensure 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.
- The best geometrical design, such as curved transitions, minimal mass at the crest, gradual changes in arch and cantilever stiffness in the top half of arch dams, and a downstream face slab for buttress dams. The objective is to limit the concentration, magnitude, and extent of excessive compressive and tensile stresses from earthquake-induced ground motion.
- Sufficient strength of the concrete and lift joints, especially in the upper half of the dam where the tensile stresses can be the greatest.
- Effective quality control during construction to ensure adequate foundation preparation, strengths of the concrete and lift joints, and placement of the reinforcement when used.

c. Methods of Analyses

(1) General

Three methods currently being applied to various levels of studies and types of dams, in the order of complexity, are: the pseudostatic, response spectrum, and time-history methods. All commonly used methods assume the dam is made of linearly elastic, homogeneous, and isotropic material.

(2) Pseudostatic Method

This method consists of determining a set of loads acting on the dam to prescribed average horizontal and vertical ground accelerations based on a seismic zone map and applying these loads statically to the dam. The acceleration is assumed either constant or varying over the height of the dam, depending on the flexibility of the dam. The earthquake loadings consist of those resulting from hydrodynamic pressures from the reservoir and inertia forces of the dam. The hydrodynamic forces are determined using Westergaard's approximation for an equivalent mass of water to move with the dam. Westergaard's method is more applicable to stiff gravity-type dams. The pseudostatic method is commonly used for checking sliding and overturning stability for gravity and buttress dams.
(3) Dynamic Analyses Methods

These methods include any analyses which determine the structural response based on the dynamic characteristics of the structures and of the earthquake ground motions established for the site. The dynamic method currently used most often is the modal 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 modal analysis technique. Dynamic methods should be used whenever a stress analysis of a concrete dam is required.

(a) Response Spectrum Method

In this method the maximum response in each mode, in the form of equivalent lateral loads, is directly computed from the earthquake response spectrum and the structure's dynamic characteristics. These modal responses are then combined to obtain estimates (but not the exact values) of the maximum total response. Adding the absolute values of each mode gives an upper bound for the total response, while using the square root of the sum of squares yields a more probable total response. The internal stresses are then computed by a static analysis of the structure subjected to the equivalent lateral loads and added algebraically to the static load stresses. Either a simple beam analysis (P/A ± Mc/I) or finite element analysis can be used to compute the stresses.

(b) Time-History Method

In this method the response of each mode, in the form of equivalent lateral loads, is calculated for the entire duration of an earthquake acceleration-time record or synthetic accelerogram, starting with initial conditions, taking a small time interval, and computing the response at the end of each time interval. The responses of each mode are added together for each time interval to yield the total time-history response. The internal stresses are then computed by a static analysis, normally using the finite element method of analysis, and added algebraically to the static load stresses. Stress histories of horizontal and vertical stresses on each face show the maximum, the number and duration of excursions beyond the tension or compression limits, and the area of the dam in distress.

(4) Sequence of Analyses

The stress analysis should begin with the simplest appropriate methods, using conservative assumptions, and progress to more refined methods as needed.

(5) Evaluating Results

Because all methods assume linearly elastic material, considerable judgment must be exercised in evaluating the dam's stability from earthquakes producing excessive tensile stresses over large areas of the concrete portion of the dam. At this time, the state of the art has advanced sufficiently to include the nonlinearity of the structure and materials and to account for the reservoir-dam-foundation interaction, however, it is not yet practical to use in production.

(6) Foundation Stability

The depth of study for determining foundation stability should be at least equivalent to that of the dam analysis. Foundation stability is often determined by pseudostatic assessments of sliding along foundation discontinuities using limit equilibrium techniques. Changes in material behavior, structural interaction, and water pressure response should also be considered for critical structures. Time-history analyses of foundation blocks are practical to do, but are not routine.

d. Dam Properties for Dynamic Analysis

The ability to analyze a dam often exceeds the ability to define material properties. This should be kept in mind when performing investigations. Sensitivity studies of material properties should be performed where appropriate.

(1) Concrete

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 the rate of earthquake loading.

The concrete properties needed to evaluate the results of the dynamic analysis are the compressive and tensile strengths. The standard unconfined compression test, excluding creep effects, is 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 compression does not normally control. The modulus of rupture test is the tensile test commonly used to
determines the tensile strength. For this test, specimens should be loaded at a rate which will simulate the expected seismic loading rate. For existing dams where testing of concrete cores is desired, the splitting tensile test can also be used to determine the tensile strength.

(2) Rock

When the foundation is included in the seismic analysis, elastic moduli and Poisson's ratios for the foundation materials are required for the analysis. If the foundation is not considered as massless, the rock densities and damping characteristics are also required.

Determining the elastic moduli for a rock foundation may include several different methods or approaches, but the effects of rock inhomogeneity (due partially to rock discontinuities) on foundation behavior must be taken into account. The shear strength of weak discontinuities may be the critical element in the analyses. Thus, the determination of foundation compressibility should consider both elastic and inelastic (plastic) deformations. The resulting "modulus of deformation" is a lower value than the elastic modulus of intact rock.

The rate of loading effect on the foundation moduli is considered to be insignificant relative to the other uncertainties involved in determining rock foundation properties and is not measured; to account for the uncertainties, a lower and upper bound for the foundation moduli should be used for the structural analysis.

(3) Damping Ratio

The damping ratio is usually taken to be 5 percent for concrete gravity dams on competent rock in which cracking of the concrete does not occur. If stresses indicate cracking occurs, a value from 7 to 10 percent (based on severity of cracking expected) should be used and the analysis performed again.

3. Embankment Dams

a. Safety Concerns

(1) Types of Instability

Safety concerns for embankment dams subjected to earthquakes involve two types of instability: sliding of slopes and/or foundation and erosion of the dam or foundation materials. The main cause of sliding instability due to earthquake-induced ground motion is liquefaction of the embankment or foundation material. Other causes of sliding instability are the reduction of the shear strength of the material caused by the infiltration of water through cracks in the embankment or foundation, the increased pore pressure due to earthquake-induced ground motion, and the exceedence of the shear strength of the material due to the increased loading induced by the earthquake. Internal erosion, or piping, can be caused by the impounded water infiltrating cracks due to differential settlement; fault movement; exceedence of the tensile strength of the embankment or foundation material and overtopping due to the settlement or slope failure of the embankment; seiches in the reservoir; waves caused by slides or rockfalls into the reservoir; or failure of the spillway or outlet works.

b. Defensive Design Measures

Many problems which may develop into potential causes for failure do not require extensive analytical treatment; rather, simply the application of defensive measures will prevent deleterious effects. Such defensive measures include the following:

(1) Ample freeboard to allow for settlement, slumping, or fault movements.

(2) Wide transition zones of material not vulnerable to cracking.

(3) Chimney drains near the central portion of the embankment.

(4) Ample drainage zones to intercept possible flow of water through cracks.

(5) Wide core zones of plastic materials not vulnerable to cracking.

(6) Well-grade filter zone upstream and downstream of the core to choke cracks that may open.

(7) Crest details which will prevent erosion in the event of overtopping.

(8) Flaring the embankment core at abutment contacts.

(9) Locating the core to minimize the degree of saturation of materials.

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-23-
(10) Stabilizing slopes around the reservoir rim to prevent slides into the reservoir.

(11) Providing special details for treating the embankment-foundation interface if there is a potential for fault movement in the foundation.

(12) Providing good quality, free draining rockfill shells.

(13) Remove foundation materials that may present potential problems.

Defensive measures should be the first consideration in arriving at a solution to problems suggested by the possibility of earthquake effects. Since the occurrence of liquefaction and slope instability may invalidate the beneficial effects of defensive measures, these two concerns warrant analytical effort of sufficient sophistication to define their possibility.

c. Methods of Analysis

(1) General

For a dam and foundation not subject to liquefaction, deformation should not be a problem if all of the following conditions are satisfied:

(a) The dam is a well-built (densely compacted) dam and the peak bedrock accelerations are 0.2g or less;

(b) The slopes of the dam are 3 horizontal to 1 vertical or flatter (These slopes are applicable to earthfill dams. Slopes could be steeper on rockfill dams with central or inclined core);

(c) The static factor of safety of the critical failure surfaces involving the crest (other than the infinite slope case and rapid drawdown) are greater than 1.5 under loading conditions expected prior to an earthquake; and

(d) The freeboard is 2 to 3 percent of the embankment height.

If these conditions are not satisfied, a deformational analysis should be made using the Newmark approach; or alternatively, a deformational analysis using a finite element-strain potential approach may be used.

(2) Liquefaction Analysis

The evaluation of the potential for liquefaction of silts, sands, and gravels at a site requires specification of the earthquakes affecting the site by magnitude and epicentral distance. A rapid evaluation of the possibility of liquefaction can be made by examining the considerable body of information existing which documents known cases of liquefaction and no liquefaction during earthquake-induced ground motion. This information should be used to help determine liquefaction possibilities.

A procedure widely used in determining the liquefaction potential of natural deposits is to estimate resisting capacity from standard penetration tests and compare this to the required capacity as determined by empirical correlations or analytical methods. For embankment or foundation materials that can be effectively sampled and tested, the cyclic and/or steady state strengths of material may be determined by laboratory testing and compared to the required strengths determined by analytical or empirical procedures.

(3) Pseudostatic Analysis

A pseudostatic analysis (sometimes called seismic coefficient analysis) should only be considered as an indication of the seismic resistance available in a structure, and then only for structures not subject to significant dynamic response or build-up of pore pressure due to shaking.

Studies of earthquake-induced slides show that the method does not always predict failure where failures have been found to occur in embankments consisting of sandy soil or constructed on sandy foundations which show a marked loss of strength due to earthquake shaking. Pseudostatic analysis is not a reliable procedure for evaluating the possible performance of dams and foundations of these types of soils. However, it has been shown that when applied to soils which show no significant loss of strength or pore pressure development due to earthquake shaking (usually clayey soils, dry sands, and some dense saturated sands), the procedure will generally provide an acceptable method of ensuring adequate performance for embankments if minimum safety factors for corresponding levels of acceleration are achieved.

(4) Newmark-Deformational Analysis

Several procedures are available for carrying out the basic Newmark-type deformational analysis. Two steps in this analysis are required. The first is to obtain the response of the structure to the earthquake ground motion and the second is to make the displacement calculations on one or more potential sliding masses. Various methods are available for calculating the response and
displacement of the structure. The deformation analysis can be made by a simplified or a more rigorous site-specific procedure.

(5) Finite Element Strain Potential Analysis

The details of this analysis have undergone many improvements through development and application of finite element procedures and through the development of improved testing procedures. The basic principles of the method involve:

(a) Determining the cross-section of the dam to be used for analysis.
(b) Determining, with the cooperation of geologists and seismologists, the maximum time-history of base excitation to which the dam and its foundation might be subjected.
(c) Determining, as accurately as possible, the stresses existing in the embankment before the earthquake; this is probably done most effectively at the present time using finite element analysis procedures.
(d) Determining the dynamic properties of the soils comprising the dam, such as shear modulus, damping characteristics, bulk modulus, or Poisson's ratio, which determine its response to dynamic excitation. Since the material characteristics are nonlinear, it is also necessary to determine how the properties vary with strain.
(e) Computing, using an appropriate dynamic finite element analysis procedure, the stresses induced in the embankment by the selected base excitation.
(f) Subjecting representative samples of the embankment materials to the combined effects of the initial static stresses and the superimposed dynamic stresses; determining their effects in terms of the generation of pore water pressures and the development of strains; and performing a sufficient number of these tests to permit similar evaluations to be made, by interpolation, for all elements comprising the embankment.
(g) Evaluating the factor of safety against failure of the embankment either during or following the earthquake from the knowledge of the pore pressures generated by the earthquake, the soil deformation characteristics, and the strength characteristics.

(b) If the embankment is found to be safe against failure, assessing the overall deformations of the embankment using the strains induced by the combined effects of static and dynamic loads.

It is important to incorporate the requisite amount of judgment in each of steps (a) to (h) as well as in the final assessment of probable performance, being guided by a thorough knowledge of typical soil characteristics, the essential details of finite element analysis procedures, and a detailed knowledge of the past performance of embankments in other earthquakes. While the procedure may seem rather long and cumbersome, it does lend itself to simplified versions.

d. Cracking Potential and Internal Erosion

In addition to analyses of embankment deformation and liquefaction, it is necessary to assess the potential for internal erosion should cracking of the embankment or its foundation be caused by deformation during earthquake shaking or fault rupture in the dam foundation. There are no analysis techniques that can be directly applied to this problem. Judgment must be used to decide whether or not erosion in zones of cracking would tend to be self-healing as the result of filtering. In some cases, concern about poor filtering properties of embankment and foundation materials has led to construction of protective filters for the specific purpose of preventing internal erosion following cracking.

H. Evaluation of Structural Adequacy for Earthquake Loading

1. Performance Criteria

Dams should be capable of withstanding the MDE without failure that would result in a catastrophic loss of the reservoir. Inelastic behavior with associated damage is permissible as long as the reservoir is retained during and after the MDE.

In addition, dams should be capable of resisting an OBE without serious damage, remain operational, and not require extensive repair work. All systems and components necessary to operate the project should be designed to remain operable during and after the OBE.

2. Evaluation of Analyses Results for Concrete Dams

a. Pseudostatic Analysis

The analysis results are evaluated in terms of sliding and overturning stability criteria. The safety factor against
sliding should be based on the degree of uncertainty in
determining the sliding resistance of the dam and the frequency
of the loading. Generally, for concrete dams, the minimum
sliding safety factor for the earthquake loading determined by
the pseudostatic method is taken as two-thirds of the minimum
safety factor for the usual static loading conditions. For the
overturning stability criteria, the location of the resultant
of all the forces at the plane of investigation should be
within the boundaries of the structure such that the allowable
pressure for the material at the plane of investigation is not
exceeded.

The pseudostatic method is the traditional method for
checking the stability of dams for earthquake loading
conditions. The average horizontal and vertical ground
accelerations used in this method are usually considerably less
than those prescribed for the MDE, but some conservatism is
introduced by treating the loads as static loads and by
determining the sliding resistance based on material strength
parameters from static tests. Sliding and overturning


cracking is acceptable for the MDE loading, the significance of
the tensile stresses is not as easily evaluated.

b. Dynamic Analyses

The dynamic analyses results are evaluated in terms of
compressive and tensile strengths of the concrete. The
compressive stresses resulting from the combination of static
and earthquake loads should be substantially less than the
dynamic strength capacity of the concrete. But since tensile
cracking is acceptable for the MDE loading, the significance of
the tensile stresses is not as easily evaluated.

To evaluate tensile stresses which exceed the tensile strength,
sound engineering judgment based upon the expected effects of
nonlinear behavior and the past performance of dams under
similar earthquake loadings is required. To estimate the
extent of cracking, nonlinear behavior resulting in stiffness
degradation and energy absorption should be considered.
Nonlinear behavior reduces the peak values of tensile stress
and the extent of tensile zones. Thus, large tensile stresses
given by a linear elastic analysis do not necessarily
indicate an unsafe condition.

The past performance of concrete dams under large earthquake
loading has been excellent. No failures of concrete dams which
have resulted in a release of the reservoir have been reported.

3. Evaluation of Analyses Results for Embankment Dams

a. Liquefaction Analysis

The evaluation of the results of a liquefaction analysis should
be based on cases of embankment and foundation failures due to
liquefaction and soil tests to determine if the embankment or
foundation material is subject to loss of strength due to
earthquake-induced shaking.

b. Pseudostatic Analysis

The pseudostatic analysis results should be evaluated with
respect to a minimum safety factor against a slope failure,
involving a significant portion of the embankment.

c. Deformational Analysis

The deformational analysis results should be evaluated in terms
of allowable permanent displacement and in comparison with the
field performance of dams shaken by earthquakes producing
similar levels of seismic loading on embankment dams. If a
finite element analysis is performed, the adequacy of the
embankment dam should additionally be based on an overall
evaluation of computed tension zones, strain failure zones, and
ratios of calculated-to-required failure shear stresses.

4. Past Experience of Dams Shaken by Earthquakes

a. 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 this experience.

(1) Gravity Dams

(a) 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 1.3 magnitude (estimated)
1906 San Francisco earthquake with no apparent
damage.

(b) Several 200-foot-high gravity dams in Japan
experienced earthquakes producing a NH intensity of
VIII in the area of the dams and were not damaged.
However, there are some examples 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 1963 a gravity dam for deposition of muck from a mine was cut at a horizontal section at the elevation approximately two-thirds up the dam height. However, the cross section of this dam was smaller than that of a water storage dam, probably due to evaluating the pressure exerted by deposited muck as being smaller than water pressure.

(c) In 1967 the Koyna Dam, a 340-foot-high gravity dam in India, survived a near-field magnitude 6.4 earthquake, with a MM intensity at the site of VIII or IX. A peak horizontal acceleration of 0.5g perpendicular to the dam axis and a peak vertical acceleration of 0.36g were measured at the dam. The dam did experience major cracking on both upstream and downstream faces of the nonoverflow monoliths. The overflow monoliths were not damaged. Dynamic analysis of the Koyna Dam to the Koyna earthquake forces was performed using two-dimensional finite element methods which incorporate hydrodynamic interaction and assume linear elastic behavior of the concrete and foundation. The analysis indicated tensile stresses in the upper part of the nonoverflow monoliths up to three times the tensile strength of the concrete. Analysis of the overflow monolith indicated maximum tensile stresses approximately equal to the concrete tensile strength.

(2) Arch Dams

(a) In 1971 the Pacoima Dam, a 372-foot-high arch dam built in areas where large earthquakes are anticipated. In 1971 the Pacoima Dam, a 372-foot-high arch dam in California, survived a magnitude 6.5 earthquake but suffered major cracking at the top of the arch in the buttress-face slab transition zone. The analysis indicated tensile stresses in the upper part of the overflow monolith which could have caused cracking; but again there was no physical evidence of any structural damage to the dam from that earthquake.

(b) In 1976, several Italian arch dams (Ambiente, Maina di Sauris, and Barcis) were subjected to a series of earthquakes having reported intensities as high as MM IX in the vicinity of the dams. No structural damage was reported at any of these dams during inspection after the earthquakes.

(c) 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 overstressed. Vertical cracking may be preempted by additional damping, due to nonlinearity of material properties or the computed but nonexistent formation of horizontal tensile stresses across contraction joints.

(3) Buttress Dams

In 1962 the Hsienfoukiang Dam, a 345-foot-high buttress dam in China, survived a near-field magnitude 6.1 earthquake but suffered major cracking at the top of the buttress-face slab transition zone.

b. Embankment Dams

The past experience of embankment dams shaken by earthquakes has been comprehensively reviewed. This review has shown the importance of examining the response of embankments shaken by earthquakes as a guide in making earthquake safety evaluations of other embankments. The following are some of the major conclusions concerning embankment dam performance under earthquake loading:

(1) Hydraulic fill dams have been found to be vulnerable to failures under unfavorable conditions (in particular, shaking produced by strong earthquakes).

(2) Many hydraulic fill dams have performed well for many years and when they are built with reasonable slopes on good foundations they can survive moderately strong shaking (0.2g) with no detrimental effects.

(3) Virtually any well-built rolled fill dam can withstand moderate earthquake shaking, with peak accelerations of 0.2g and more, with no detrimental effects.
(6) 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.

(5) Two rockfill dams have withstood moderately strong shaking with no significant damage and, if the rockfill is kept dry by means of an impervious facing they should be able to withstand extremely strong shaking with only small deformations.

In summary, 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 probably true of well-drained, compacted rock fill dams or dumped rock fill dams with an impervious core, although some surface deformation can be expected. Rockfill dams with membrane facing, e.g. concrete, are susceptible to earthquake damage. Permanent displacement or cracking of the facing can be expected. Therefore, these dams should be considered as requiring an analysis.

Low density fill dams built of low plasticity granular soils, especially hydraulic or semihydraulic fill dams, are highly susceptible to earthquake damage because of liquefaction potential. Also, dams placed on low density, poorly drained foundations subject to liquefaction are suspect. Consequently, an analysis of these dams should be performed.

5. Evaluation of Existing Dams

The criteria and analyses methods cited in these guidelines may differ significantly from those in use when many older dams were designed; and therefore, some existing structures may be found deficient using the procedures outlined. If the analyses indicate the dam is overstressed 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 excessive property damage, remedial action may not be necessary. However, this position should be fully documented.

APPENDIX A

DEFINITION OF TERMS

1. 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.

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

3. 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 3500 feet/second.

4. 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.)

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

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

   a. Movement at or near the ground surface at least once within the past 35,000 years.
   b. Microseismicity (3.5 magnitude or greater) instrumentally determined with records of sufficient precision to demonstrate a direct relationship with the fault.
   c. A structural relationship to a capable fault such that movement on one fault could be reasonably expected to cause movement on the other.
   d. Established patterns of microseismicity which define a fault, with historic macroseismicity that can reasonably be associated with the fault.
7. Critical Damping - The minimum amount of damping which prevents free oscillatory vibration.

8. 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.

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

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

11. 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 which, if used, should be clearly defined.

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

13. 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.

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

15. 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.) has been used to designate the size of an earthquake due to the lack of instrumental data.

16. 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.

17. Limits of Near-Field Motion:

<table>
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<tr>
<th>Richter Magnitude, M</th>
<th>Modified Mercalli Intensity, I°</th>
<th>Near-Field, km</th>
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</thead>
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<tr>
<td>5.0</td>
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<td>5</td>
</tr>
<tr>
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<td>VII</td>
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</tr>
<tr>
<td>7.5</td>
<td>XI</td>
<td>45</td>
</tr>
</tbody>
</table>

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.

18. 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 (Mw) - 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 (Mw) or the 1-second period surface waves (M1).

Richter or Local Magnitude (ML) - 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.
APPENDIX A (Continued)

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 $M_g$ equal to 6.3. Above this $M_g$ is used to define magnitude.

Surface Wave Magnitude ($M_g$) — 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.

19. 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.

20. 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 which 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.

21. Operating Basis Earthquake (OBE) — The earthquake(s) for which the structure is designed to resist and remain operational. 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.

APPENDIX A (Continued)

22. 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.

23. 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 pseudoecceleration, pseudovelocity, or pseudodisplacement response spectrum values.

24. 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.

25. 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:

$$M_0 = \text{Shear Modulus of Faulted Rock (Dynes/cm}^2\text{)} \times \text{Length of Fault Rupture Zone (cm)} \times \text{Width of Fault (cm)} \times \text{Displacement of Fault (cm)}$$

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

27. Seismotectonic Source Area — An area of known or potential seismic activity which may lack a specific identifiable seismotectonic structure.

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

28. Seismotectonic Structure — An identifiable dislocation or distortion within the earth's crust resulting from recent tectonic activity or revealed by seismologic or geologic evidence.
29. Smooth Response Spectrum - A response spectrum devoid of sharp peaks and valleys which specifies the amplitude of the spectral acceleration, velocity, and/or displacement to be used in the analyses of the structure.

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

31. Surface Waves - Waves which travel along or near the surface and include Rayleigh (Sr) and Love (Lr) Waves of an earthquake.

32. Synthetic Earthquake - Earthquake time history records developed from mathematical models that use white noise, filtered white noise, and stationary and nonstationary filtered white 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.

APPENDIX B

BIBLIOGRAPHY

The reader is referred to the following two documents which include extensive reference lists on the topics of seismology and defining design earthquake ground motions.


A supplementary list of selected publications, particularly for the years 1979 to 1983, is also included to assist users of this guideline.


Reading and Interpreting Strong Motion Accelerograms, by Donald K. Hudson.

Dynamics of Structures - A Primer, by Anil K. Chopra.

Earthquake Spectra and Design, by Nathan M. Newmark and William J. Hall.

Earthquake Design Criteria, by George W. Housner and Paul C. Jennings.

Ground Motions and Soil Liquefaction During Earthquakes, by H. Bolton Seed and I. M. Idriss.


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APPENDIX B (Continued)


Creager, William F., Justin, J. D., and Hinds, Julian, 1945, Engineering for Dams: John Wiley and Sons, New York, 3 v.


APPENDIX B (Continued)


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