Chapter 3
Estimating Earthquake Ground Motion Demands

3-1. Specification of Earthquake Ground Motions

a. General. The earthquake ground motions for design and evaluation of CHS are generally characterized in terms of response spectra and acceleration time histories. Information on response spectra can be found in EM1110-2-6050, “Response Spectra and Seismic Analysis for Concrete Hydraulic Structures." Information on earthquake acceleration time histories and time history analysis can be found in EM 1110-2-6051, “Time History Analysis for Concrete Hydraulic Structures.” ER 1110-2-1806, “Earthquake Design and Evaluation for Civil Works Projects,” provides guidance and direction for the seismic design and evaluation of all civil works projects.

b. Using response spectra for earthquake design and analysis. Acceleration response spectra represent the peak acceleration response of a number of single-degree-of-freedom (SDOF) oscillators to particular earthquake ground motions. Information on response spectra can be found in Technical Report ITL-92-4 (Ebeling 1992) and EM1110-2-6050. Earthquake response spectra can be site specific or standard (non-site specific). Standard response spectra are based on spectral shapes developed for recorded ground motions with similar subsurface characteristics. The standard spectral shapes are defined with respect to effective peak ground accelerations or spectral accelerations taken from the national seismic hazard maps. Although a response spectrum represents the maximum response of SDOF systems, the response of multi-degree of freedom systems (MDOF) can also be obtained from the response spectrum by applying the mode-superposition technique. According to this technique the linear earthquake response of a MDOF system can be obtained by combining responses of several SDOF systems, each of which represents a mode of vibration of the MDOF system. The dynamics of MDOF systems are described in Technical Report ITL-94-4 (French et al. 1994) and EM 1110-2-6050.

c. Standard response spectra. Guidance is provided in Appendix B for constructing standard acceleration response spectra based on the most recent national seismic hazard data. Appendix B provides a procedure for developing standard acceleration response spectra and effective peak ground accelerations for use in the seismic design and evaluation of structural features of USACE projects as required by ER 1110-2-1806. Standard response spectra are based on a general characteristic shape that is defined in terms of effective peak accelerations or spectral accelerations. The effective peak ground acceleration is obtained from division of the spectral ordinate (for a 5%-damped spectrum) at period of 0.2 seconds by a standard value of 2.5. Examples of standard response spectra and determination of effective peak ground accelerations are given in Appendix C. The standard response spectra can be used as a starting point for developing conceptual designs and performing evaluations, determining if the earthquake loading controls the design, and establishing the need for more refined analysis and the impact the earthquake loading might have on construction cots.

d. Site-specific response spectra. Earthquake ground motions are dependent on source characteristics, source-to-site transmission path properties, and site conditions. The source characteristics include stress conditions, source depth, size of rupture area, amount of rupture displacement, rise time, style of faulting, and rupture directivity. The transmission path properties include the crustal structure and shear-wave velocity and damping characteristics of the crustal rock. The site conditions include the rock properties beneath the site to depths up to 2 km, the local soil conditions at the site up to a hundred meters or more in depth, and the topography of the site. All these factors are considered in detail in a site-specific ground motion study, rather than in a general fashion as occur in the standard response spectra methodology. Also, due to regional differences in some of the factors affecting earthquake ground motions, different attenuation relationships exist. There are two basic approaches to developing site-specific response spectra: deterministic and probabilistic. In the deterministic approach, typically one or more earthquakes are specified by magnitude and location with respect to a site. Usually, the earthquake is taken as the Maximum Credible Earthquake (MCE), and assumed to occur on the portion of the source closest to the site. The site ground motions are then estimated deterministically, given the magnitude and source-to-site distance. In the probabilistic approach, site ground motions are estimated for selected values of probability of ground motion exceedance in a design time period or for selected values of the annual frequency or return period of ground motion exceedance. A probabilistic ground motion assessment incorporates the frequency of occurrence of earthquakes of different magnitudes on the various seismic sources, the uncertainty of the earthquake locations on the various sources, and the ground motion attenuation including its uncertainty. Guidance for developing site-specific response spectra and for using both the deterministic approach and the probabilistic approach can be found in EM 1100-2-6050.

e. Acceleration Time Histories. EM 1110-2-6051 describes the procedures for developing site-specific acceleration time-histories of ground motion for dynamic analysis of hydraulic structures. The overall objective is to develop a set (or sets) of time-histories that are representative of site ground motions that may be expected for the design earthquake(s) and that are appropriate for the types of analyses planned for specific structures. The following steps are included in this process:

(1) Initially selecting recorded time-histories that are reasonably consistent with the tectonic environment of the site; design earthquake (magnitude, source-to-site distance, type of faulting); local site conditions; and design ground motion characteristics (response spectral content, duration of strong shaking, and special characteristics, e.g. near-source characteristics). If sufficient recorded motions are not available, simulated recorded time-histories can be developed using ground motion modeling methods.

(2) Modifying time-histories selected in (1) above to develop the final set(s) to be used in dynamic analysis. Two approaches that can be used in this process are simple scaling of time-histories (by constant factors) so that a set of time-histories has spectral values that, on average, are at the approximate level of the design response spectrum; and spectrum matching, which involves modifying the frequency content of a given time-history so that its response spectrum is a close match to the design response spectrum.

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

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

f. Selection of records for deterministically defined and probabilistically defined earthquakes. Application of the above guidelines is straightforward when design earthquakes are expressed deterministically, i.e., in terms of magnitude, faulting type, and source-to-site distance. However, the application of the guidelines is less straightforward when the design earthquake ground motions (typically the response spectrum) are derived from a probabilistic ground motion analysis (often termed a probabilistic seismic hazard analysis or PSHA). From this type of analysis, which is described in detail in EM 1110-2-6050, the design response spectrum for a certain selected probability of exceedance in a design time period (or, equivalently, for a design return period) reflects the contribution of different earthquake magnitudes and distances to the probabilities of exceedance. Therefore, when the design response spectrum is probabilistically based, the PSHA should be deaggregated to define the relative contributions of different magnitudes and distances to the ground motion hazard. Furthermore, the de-aggregation should be done for probability values or return periods that correspond to those of the design earthquake and for response spectral periods of vibration of significance for seismic structural response because the relative contributions of different magnitudes and distances may vary significantly with return period and period of vibration. The dominant magnitude and distance is then considered as representative in selecting time histories and defining strong motion duration.

3-2. Multi-Directional Effects

a. General. Two-dimensional structures are generally analyzed for one or two components of the earthquake ground motion (horizontal only, or horizontal + vertical). The ground motions may be defined as multi-component response spectra or acceleration time histories. Some three-dimensional structures such as navigation locks and straight gravity dams can be idealized as two-dimensional structures. While others such as arch dams must be evaluated using three-dimensional models requiring three components of ground motion (two horizontal + vertical). In certain cases, such as freestanding intake/outlet towers, the vertical component of earthquake ground motion may be ignored if it contributes very little to the total response. Structures must be capable of resisting maximum earthquake ground motions occurring in any direction. In response spectrum analysis, the secondary component of horizontal ground motion is usually set equal to the primary component. This is somewhat conservative but eliminates the need to determine the ground motion direction of attack that produces the greatest demand to capacity response (DCR). However, in time history analysis it may be necessary to apply horizontal components in either horizontal direction in order to obtain the largest response in accordance with Paragraph *e* below. The orthogonal components of earthquake ground motion are commonly applied along the principal axes of the structure. In time history analysis the total response due to all components of the ground motion are obtained by algebraic summation of responses due to each individual component. In response-spectrum analysis the total response due to multiple earthquake components are estimated as described in the following paragraphs.

b. Percentage combination method. In a response-spectrum analysis, the percentage combination method can be used to account approximately for the simultaneous occurrence of earthquake forces in two perpendicular horizontal directions. For rectangular structures with clearly defined principal directions, this method yields approximately the same results as the SRSS method described in Paragraph 3-2*c* below. However, for non-rectangular and complex three-dimensional structures the percentage method can under estimate structural responses. The percentage combination is accomplished by considering two separate load cases for both the OBE and MDE. Illustrating for the MDE loading combination, Equation 2-1, the two load cases would be as follows:

Load Case 1:

$$Q_{DC} = Q_D + Q_L \pm Q_{MDE(X1)} \pm \alpha Q_{MDE(X2)} \qquad (3\text{-}1)$$

Load Case 2:

$$Q_{DC} = Q_D + Q_L \pm \alpha Q_{MDE(X1)} \pm Q_{MDE(X2)} \qquad (3\text{-}2)$$

Generally α is assumed to be 0.30 for rectangular structures, and 0.40 for circular structures.

Q_{DC} = peak value of any response quantity (forces, shears, moments, stresses, or displacements) due to the effects of dead load, live load, and the MDE

$Q_{MDE(X1)}$ = effects resulting from the X_1 component of the MDE ground motion occurring in the direction of the 1st principal axis of the structure.

$Q_{MDE(X2)}$ = effects resulting from the X_2 component of the MDE ground motion occurring in the direction of the 2nd principal axis of the structure.

c. Square Root of the Sum of the Squares Method (SRSS). A better way to combine structural responses for the multi-component earthquake response spectra is the use of the SRSS method. This method is applicable to rectangular and complex three-dimensional structures. For any response quantity of interest, e.g. moment or shear at a particular location, the results from the separate application of each component of ground motion are combined by the square root of the sum of the squares to obtain the total response. Note that since response-spectrum stresses, forces, or moments have no sign, the combination should consider response-spectrum quantities to be either positive or negative. Illustrating for the MDE loading combination, Equation 2-1, the SRSS demand would be as follows:

$$Q_{DC} = Q_D + Q_L \pm \left[(Q_{MDE(X1)})^2 + (Q_{MDE(X2)})^2 + (Q_{MDE(Z)})^2 \right]^{1/2} \qquad (3\text{-}3)$$

d. Critical Direction of Ground Motion. The directions of ground motion incidence are usually assumed along the fixed structural reference axes. Considering that the ground motion can act along any horizontal direction, there could be a different direction of seismic incidence that would lead to an increase of structural dynamic response. The maximum structural response associated to the most critical directions of ground motions has been examined in several publications (Wilson and Button 1982, Smedy and Der Kiureghian 1985, Wilson et al. 1995, Lopes and Torres 1997, and Lopez et al. 2000). These investigations indicate that the critical direction of the horizontal ground motion components yielding the maximum structural response depends on the two horizontal spectra and also on the structural response parameters but not on the vertical spectrum. For the special case of identical spectra along the two horizontal directions, the structural response does not vary with the angle of incidence, i.e. any direction is a critical direction. The response value for this special case is the upper bound to all possible structural responses due to any combination of spectral ratios and angle of incidence. A conservative approach is therefore to analyze the structure with the same horizontal spectrum applied simultaneously along the two horizontal structural reference directions, and the vertical spectrum.

e. Load combination cases for time history analysis. 3D time-history analysis of CHS with response in the damage control range should be evaluated for three or more sets of three-component earthquake ground motions plus the effects of usual static loads. For each set of three-component earthquake ground motions the static loads and earthquake ground motion components should be combined in accordance with Table 3-1. In general, a complete permutation of all three components with positive and negative signs may be required to obtain the most critical directions that would cause the largest structural response (EM 1110-2-6051). 2D time-history analysis of CHS with response in the damage control range is conducted in a similar fashion requiring three or more sets of two-component acceleration time-history records, except that for two-component excitation a total of 4 permutations will be required.

Table 3-1
Load Combination Cases for Combining Static and Dynamic Stresses for Three-component Excitation

	Seismic Loads			
Case	**Horizontal (H1)**	**Vertical (V)**	**Horizontal (H2)**	**Static Loads**
1[1]	+	+	+	+
2	+	+	-	+
3	+	-	+	+
4	+	-	-	+
5	-	+	+	+
6	-	+	-	+
7	-	-	+	+
8	-	-	-	+

Note: The (+) and (-) signs indicate the loads are multiplied by +1 (zero phase) or -1 (180 phase) to account for the most unfavorable earthquake direction.
[1] Case-1: Static + H1 + V + H2

3-3. Earthquake Demands on Inelastic Systems

a. General. The inelastic response of a structure to earthquake ground motion is different than the elastic response. The difference occurs because the vibration characteristics of the structure change as the structure yields. The predominant change is a shift in the fundamental period of vibration. In most cases, a reduction in earthquake demand occurs as the period of the structure lengthens. In Figure 3-1, a capacity spectrum is used to illustrate the inertia force reduction (or spectral acceleration reduction) that occurs when a structure yields. In Figure 3-1, earthquake demands for an elastic system, as represented by a response spectrum, are reconciled with the elasto-plastic load/displacement characteristics for a ductile structure. Point "*A*" represents the earthquake demand assuming the structure remains elastic with the line "*O-A*" representing the linear elastic response. Point "*B*" represents the earthquake demands for elasto-plastic behavior with the line "*O-B-C*" representing the load/displacement characteristics of the elasto-plastic system. As can be seen, the earthquake force demand on the elasto-plastic system is substantially less than that of the elastic system. It should also be recognized that as a structure yields, damping increases significantly, thus leading to further reduction of earthquake demands. However, if the fundamental period of the structure falls in the ascending portion of the response spectrum, a shift in period may actually increase earthquake demands. This condition is likely to occur for stiff structures founded on soft soils, but can also occur for stiff structures founded on rock. In such cases, the structure should be designed to remain elastic.

b. Inelastic displacement demands. Knowing displacements in yielding structures, the level of damage or yielding can be controlled by limiting displacements to predetermined values for a specified level of earthquake shaking. Simple techniques have been developed for estimating inelastic displacements from elastic displacements. Structures with a period of vibration between zero and 0.75 T_0, where T_0 is characteristic ground motion period (period corresponding to the peak acceleration response), exhibit an equal acceleration response which offers no benefit from yielding. Structures with periods of vibration greater than 0.75 T_0, however, will benefit from structure displacement ductility. The inelastic response of structures with fundamental periods of vibrations between 0.75 T_0 and 1.5 T_0, can be estimated using equal energy principles. The inelastic response of structures with fundamental periods of vibration greater than 1.5 T_0, can be estimated using equal displacement principles.

(1) Equal acceleration response. Rigid structures, with a period of vibration (T) equal or less than 0.04 sec (or between 0 and 0.75 T_0), will exhibit an equal acceleration response. In this case, force or acceleration is conserved regardless of any ductile properties attributed to the structure. Structures exhibiting an equal force or acceleration response should therefore be designed to remain elastic.

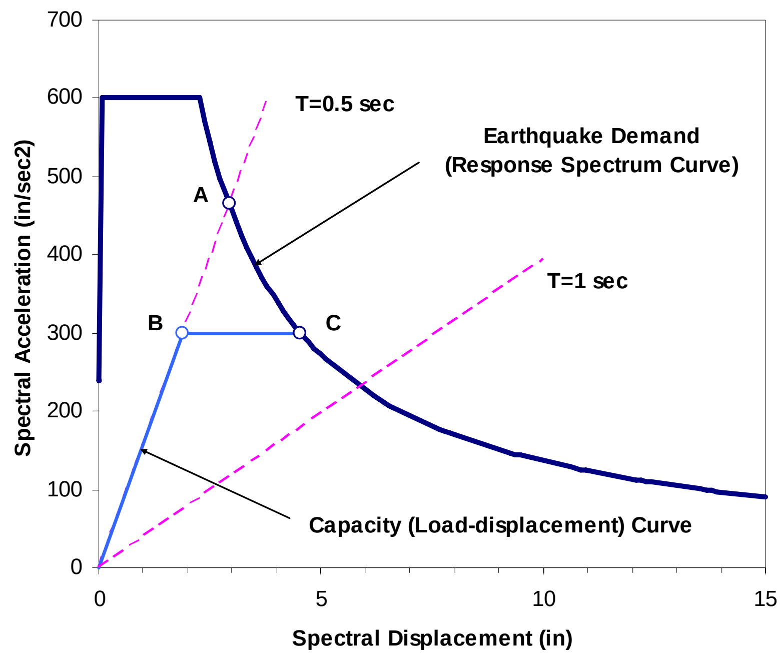

Figure 3-1. Earthquake Demands on Inelastic Structures

(2) Equal energy response. Structures with fundamental periods of vibration between 0.75 T_0 and 1.5 T_0 will exhibit an equal energy response. The characteristic ground motion period (T_0) generally varies between 0.2 seconds and 0.7 seconds depending on site conditions, with firm sites having shorter characteristic periods than soft sites. The structure must have sufficient displacement ductility to provide the reserve inelastic energy capacity needed to resist earthquake ground motion demands. The equal energy response concept is presented in Figure 3-2. For a given displacement ductility (μ_δ), the inelastic (yield) capacity (F_Y) must be sufficient to produce an equal energy response. Equating the energy for a linear elastic response to that for an inelastic response (hatched area under the nonlinear portion of the load displacement curve equal to the hatched area under the linear elastic curve), it can be determined that the yield capacity of the structure must be equal to or greater than the capacity required of the structure if it were to remain elastic (F_E) divided by $\sqrt{2\mu_\delta - 1}$, or:

$$F_Y \geq \frac{F_E}{\sqrt{2\mu_\delta - 1}} \qquad (3\text{-}4)$$

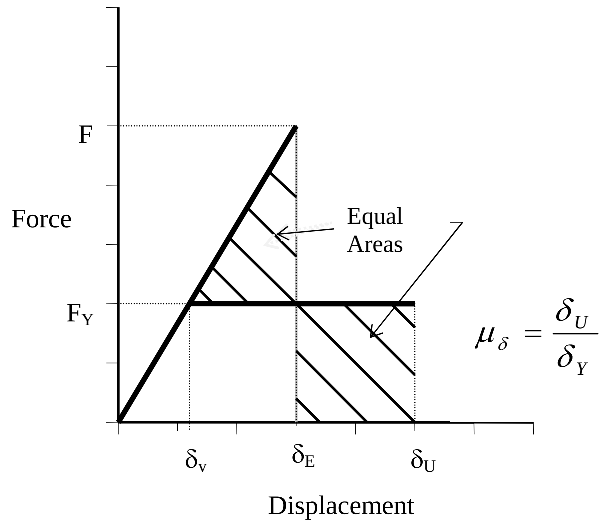

Figure 3-2. Equal Energy Response

(3) Equal displacement response. Structures with fundamental periods of vibration greater than 1.5 T_0 will exhibit an equal displacement response. An equal displacement response means that to perform as intended, the displacement ductility capacity must be sufficient to provide a structure displacement capacity equal to, or greater than, the peak displacement the structure will experience during the design earthquake. The equal displacement response concept is presented in Figure 3-3. From Figure 3-3 it can be determined that the yield capacity of the structure must be equal to or greater than the capacity required of the structure if it were to remain elastic (F_E) divided by μ_δ, or:

$$F_Y \geq \frac{F_E}{\mu_\delta} \tag{3-5}$$

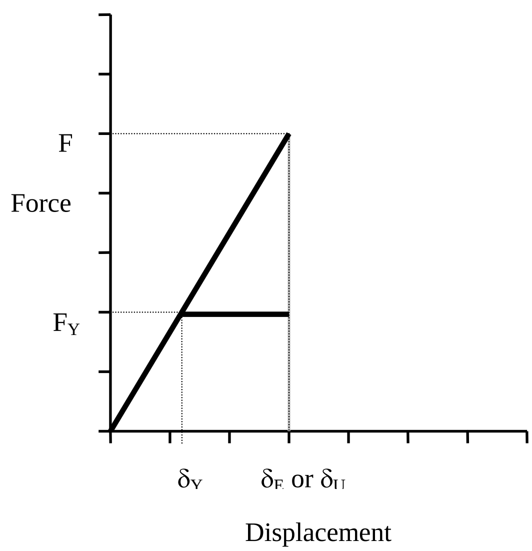

Figure 3-3. Equal Displacement Response

(4) General relationship between required yield strength (F_Y) and elastic demand (F_E). A general relationship has been developed (Paulay and Priestley, 1992) for relating required yield strength to elastic demand. This relationship provides a smooth transition from an equal acceleration response ($\frac{F_E}{F_Y} = 1$ regardless of μ_δ) at T = 0, through the equal energy approximation ($\frac{F_E}{F_Y} = \sqrt{2\mu_\delta - 1}$) at about T = 0.75 T_0, to the equal displacement approximation ($\frac{F_E}{F_Y} = \mu_\delta$) for $T \geq 1.5\ T_0$. The relationship for the smooth transition is:

$$\frac{F_E}{F_Y} = 1 + \frac{(\mu_\delta - 1)T}{1.5T_0} \geq \mu_\delta \qquad (3\text{-}6)$$

Using Equation 3-6; for a known level of displacement ductility (μ_δ) and a given elastic earthquake demand (F_E), the required yield capacity of a structural component (F_Y) can be determined.

3-4. Mandatory Requirements

a. Standard spectra. Standard spectra used in the preliminary design and evaluation of Corps hydraulic structures shall be developed in accordance with the procedures described in Appendix B.

b. Site-specific spectra. Site-specific spectra used in the preliminary design and evaluation of Corps hydraulic structures shall be developed in accordance with the procedures described in EM 1110-2-6050.

c. Acceleration time histories. Acceleration time histories for dynamic analysis of CHS shall be selected and developed in accordance with EM 1110-2-6051.

d. Multi-directional effects. Multi-directional effects shall be considered when designing or evaluating concrete hydraulic structures for earthquake ground motions. General information on multi-directional effects can be found in Paragraph 3-2.