Chapter 4
Loads and Loading Conditions

4-1. General

Previously, stability criteria was provided in separate manuals for each type of structure. Those manuals listed all of the load cases (loading conditions), which had to be investigated as part of the stability analysis. Those loading conditions are now summarized in tables provided in Appendix B of this manual. The tables list the loading condition and give a classification as usual, unusual, or extreme, as defined in Table 3-1. Following each table are brief descriptions of the loading conditions. The loading conditions have been revised in some cases for general consistency with the provisions of this manual, especially to comply with current practice for flood and seismic loadings. This chapter defines most of the types of loads that are combined to form each loading condition. However, soil loads are defined in Chapter 2 for multiple wedge sliding analyses and in Chapter 5 for single wedge sliding analyses.

4-2. Construction

Based on past practice, construction loading conditions shall be classified as *unusual*, regardless of duration.

4-3. Water Loading Conditions

a. General. All water loading conditions should be based on hydrologic information, which gives median water elevations in terms of return periods. A typical flood hazard curve is illustrated in Figure 4-1. Curves for both headwater and coincident tailwater will be necessary to determine the water loads for dams and navigation locks. Hydraulic engineers commonly use the 90-percent confidence level hazard curve when determining flood protection requirements. However, for stability analysis, structural engineers require median flood hazard curves, which can also be provided by the hydraulic engineers. Based on the information presented in Figure 4-1 a flood pool elevation equal to 21 meters (68.9 feet) would be used to determine the maximum *unusual* loading.

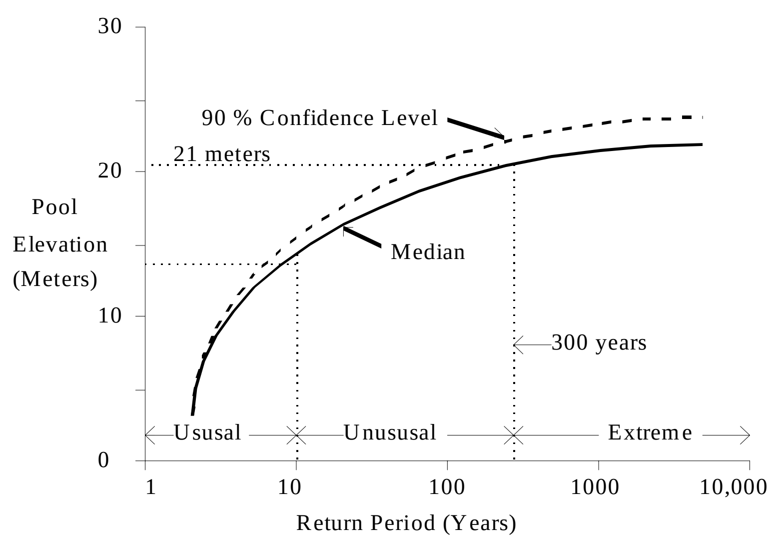

Figure 4-1 Flood Hazard Curve

b. Coincident pool. Coincident pool represents the water elevation that should be used for combination with seismic events. It is the elevation that the water is expected to be at or below for half of the time during each year.

c. Normal operation. In the past, a normal operation loading condition has been used to describe loadings with various probabilities of occurring, including rare events with long return periods. To be consistent with Table 3-1, normal operating conditions are now defined as maximum loading conditions with a return period of no more than 10 years (annual probability of 10%). For certain floodwalls, this means that there might be no water loads on the structure for normal operation. For hydropower dams, the pool will be fairly high for normal operation, while for some flood-control dams, the pool will be low for normal operation. For navigation projects, the maximum loading for normal operation might correspond to the usual navigation pool, combined with the lowest tailwater expected with a 10-year return period. Water loads defined by the normal operation loading condition are sometimes combined with other types of events (such as barge impacts).

d. Infrequent flood. The infrequent flood (IF) represents flood pool or water surface elevations associated with events with a return period of no greater than 300 years (annual probability of 0.33%), making the IF an unusual loading per Table 3-1. This loading condition replaces loadings such as water *to top of spillway gates* and *water to spillway crest* previously used for the design and evaluation of gated and ungated spillways. It also replaces the *design flood* (top of wall less freeboard) used for the design and evaluation of floodwalls. In limited cases, historical hydrologic data may be inadequate to determine the 300-year water elevations with reasonable certainty. In such cases, traditional loading conditions such as *water to top of spillway gates*, *water to spillway crest*, and *design flood* shall be considered unusual events and evaluated in addition to the IF event.

e. Probable maximum flood. The probable maximum flood (PMF) is one that has flood characteristics of peak discharge, volume, and hydrograph shape that are considered to be the most severe reasonably possible at a particular location, based on relatively comprehensive hydro-meteorological analyses of critical runoff-producing precipitation, snow melt, and hydrologic factors favorable for maximum flood runoff. The PMF load condition represents the most severe hydraulic condition, but because of possible overtopping and tailwater effects, it may not represent the most severe structural loading condition, which is represented by the maximum design flood described below. Therefore, the PMF condition will not necessarily be examined for structural stability.

f. Maximum design flood. The maximum design flood (MDF) is the designation used to represent the maximum structural loading condition (as judged by the minimum factor of safety) and must be determined for each structure or even for each structural element. MDF may be any event up to PMF. For floodwalls, MDF is usually when the water level is at or slightly above the top of the wall. Overtopping from higher water levels would result in rising water levels on the protected side, thus reducing net lateral forces. The same situation may be true for dams, but often significant overtopping can occur without significant increases in tailwater levels. The design engineer must consult with the hydraulics engineer to explore the possible combinations of headwater and tailwater and their effects on the structure. Some elements of dam outlet works (such as chute walls or stilling basins) are loaded differently from the main dam monoliths. For such elements, different flow conditions will produce maximum structural loading. When it is not obvious which loading will produce the lowest factor of safety, multiple loadings should each be investigated as a possible MDF. Since sliding is the most likely mode of failure for most gravity structures, MDF can usually be judged by determining maximum net shear forces. However, due to variable uplift conditions, a loading with smaller shears could result in the lowest factor of safety. Once the MDF is determined, it should be classified as usual, unusual, or extreme per Table 3-1, based on its return period.

4-4. Uplift Loads

Uplift loads have significant impact on stability. Sliding stability, resultant location, and flotation are all aspects of a stability analysis where safety can be improved by reducing uplift pressures. Since uplift pressures are directly related to flow paths beneath the structure, uplift pressure distribution may be determined from a seepage analysis. Such an analysis must consider the types of foundation and backfill materials, their possible range of horizontal and vertical permeabilities, and the effectiveness of cutoffs and drains. Techniques for seepage analysis are discussed in EM 1110-2-1901, EM 1110-2-2502, Casagrande (1937), Cedergren (1967), Harr (1962), and EPRI (1992). Seepage analysis techniques to determine uplift pressures on structures include flow nets, finite element methods, the line-of-creep method, and the method of fragments. Uplift pressures resulting from flow through fractures and jointed rock, however, are poorly understood and can only be accurately known by measurements taken at the point of interest. Joint asperities, changes in joint aperture, and the degree to which joints interconnect with tailwater influence uplift pressures and pressure distribution. Uplift pressures are site-specific and may vary at a given site due to changes in geology. Uplift pressures can be reduced through foundation drainage, or by various cutoff measures such as grout curtains, cutoff walls, and impervious blankets. Uplift pressures should be based on relatively long-term water elevations. Short duration fluctuations, such as from waves or from vibrations due to high velocity flows, may be safely assumed to have no effect on uplift pressures. Uplift pressures to be used for stability analysis of new structures are covered in Appendix C. The conservative uplift pressures used for the design of new structures may be significantly higher than those the actual structure may experience during its lifetime. For this reason, the use of actual uplift pressures for the evaluation of existing structures is permitted under the provisions discussed in Chapter 7. However, the engineer should be aware that in some instances the actual uplift may not be reflected by uplift cell readings. Since uplift measurement devices only capture a snapshot of a given part of the foundation, they should be used with caution, based on an overall evaluation of the foundation.

4-5. Maintenance Conditions

The return periods for a maintenance condition loading may be greater or less than 10 years, but based on past experience maintenance has been designated as an *unusual* load condition. The classification as an *unusual* loading is based on the premise that maintenance loadings take place under controlled conditions and that the structure performance can be closely monitored during maintenance.

4-6. Surge and Wave Loads

a. General. Surge and wave loads are critical in analyzing the stability of coastal protection structures but usually have little effect on the stability of inland structures. Wave and water level predictions for the analysis of structures should be based on the criteria presented in the Shore Protection Manual 1984, EM 1110-2-1612, and EM 1110-2-1614. Design forces acting on the structure should be determined for the water levels and waves predicted for the most severe fetch and the effects of shoaling, refraction, and diffraction. The methods recommended for calculation of wave forces are for vertical surfaces. Wave forces on other types of surfaces (sloping, stepped, curved, etc.) are not sufficiently understood to recommend general analytical design criteria. In any event, the structural engineer should consult with a coastal engineer in establishing wave forces for the design of critical structures.

b. Wave heights. Wave heights for design are obtained from the statistical distribution of all waves in a wave train and are defined as follows:

H_S = average of the highest one-third of all waves

$H_1 = 1.67\ H_S$ = average of highest 1 percent of all waves

H_b = height of wave which breaks in water depth d_b

c. Non-breaking waves. When the water depth is greater than approximately 1.5 times the wave height , waves do not break. The H_1 wave shall be used for the non-breaking condition. Design pressures for non-breaking waves shall be computed using the Miche-Rudgren method. Whenever the maximum stillwater level results in a non-breaking condition, lower stillwater levels should be investigated for the possibility that shallow water may produce breaking wave forces, which are larger than the non-breaking forces.

d. Breaking waves. Waves break when the steepness of the wave and the bottom slope at the front of the structure have certain relationships to each other. It is commonly assumed that a wave will break if the water depth is not greater than 1.3 times the wave height. Study of the breaking process indicates that this assumption is not always valid. The height of the breaking wave and its breaking point are difficult to determine, but breaker height can equal the water depth at the structure, depending on bottom slope and wave period. Detailed determination of breaker heights and distances for a sloping approach grade in front of the structure are given in the Shore Protection Manual 1984. Design breaking wave pressure should be determined by the Minikin method presented in EM 1110-2-1614. Breaking-wave impact pressures occur at the instant the vertical face of the wave hits the structure and only when a plunging wave entraps a cushion of air against the structure. Because of this dependence on curve geometry, high impact pressures are infrequent against prototype structures; however, they must be recognized as possible and must be considered in design. Also, since the impact pressures caused by breaking waves are of very short duration, their importance in design against sliding and rotational instability may be questionable relative to longer lasting, smaller dynamic forces.

e. Broken waves. Broken waves are those that break before reaching the structure, but near enough to have retained some of the forward momentum of breaking. The design breaker height in this case (H_b) is the highest wave that will be broken in the breaker zone. Design wave forces for this height should be determined by the method presented in Chapter 7 of the Shore Protection Manual (1984).

4-7. Earthquake Loading Conditions

a. Seismic Load Conditions. Earthquake loads are used to represent the inertial effects attributable to the structure mass, the surrounding soil (dynamic earth pressures), and the surrounding water (hydrodynamic pressures). Design earthquakes shall comply with requirements of ER 1110-2-1806, based on the following seismic events.

- *Operational basis earthquake (OBE).* The OBE is considered to be an earthquake that has a 50 percent chance of being exceeded in 100 years (or a 144-year return period).

- *Maximum design earthquake (MDE).* The MDE is the maximum level of ground motion for which a structure is designed or evaluated. For *critical* structures the MDE is the same as the maximum credible earthquake (MCE). Generally, the probabilistically determined MDE for other structures is an earthquake that has a 10 percent chance of being exceeded in a 100-year period (or a 950-year return period).

- *Maximum Credible Earthquake.* The MCE is defined as the greatest earthquake that can reasonably be expected to be generated on a specific source, on the basis of seismological and geological evidence. The MCE is based on a deterministic site hazard analysis.

Earthquake-generated inertial forces associated with the OBE are unusual loads. Those associated with the MDE are extreme loads. Earthquake loads are to be combined with other loads that are expected during routine operations, and should not be combined with other infrequent events such as flood loads. Seismic loads should be combined with *coincident pool*, which is defined as the elevation that the water is expected to be at or below for half of the time during each year.

b. Analytical Methods. Several analytical methods are available to evaluate the dynamic response of structures during earthquakes: seismic coefficient, response spectrum, and time-history. These methods are discussed in reference ER 1110-2-1806. The current state-of-the-art method uses linear-elastic and nonlinear finite element time history analysis procedures, which account for the dynamic interaction between the structure, foundation, soil, and water. The seismic coefficient method, although it fails to account for the true dynamic characteristics of the structure-water-soil system, is accepted as a semiempirical method for determining if seismic forces control the design or evaluation, and to decide if dynamic analyses should be undertaken. The information in the following paragraphs describes the differences between the seismic coefficient method and dynamic analysis methods. Figure 4-2 illustrates the differences in the inertial and hydrodynamic earthquake loads obtained by the two different methods. The seismic coefficient used for the preliminary seismic stability evaluation of concrete hydraulic structures should be equal to 2/3 the effective peak ground acceleration (EPGA) expressed as a decimal fraction of the acceleration of gravity. The EPGA can be obtained by dividing the 0.30 second spectral acceleration, for the return period representing the design earthquake, by a factor of 2.5 The 0.30 second spectral acceleration is obtained from the spectral acceleration maps in Appendix D of ER 1110-2-1806.

c. Inertia force due to structure mass. In the seismic coefficient approach, the inertial force is computed as the product of the mass of the structural wedge (including the soil above the heel or toe and any water contained within the structure) and the seismic acceleration. This may also be expressed as the weight of the structural wedge times the seismic coefficient, expressed as a fraction of gravity.

$$F_h = m\,a = k_h\,W \qquad (4\text{-}1)$$

where: F_h = horizontal component of the inertial force (a similar equation can be used for vertical component)
m = mass of structural wedge
a = seismic acceleration
W = gross weight of structural wedge (including soil above the heel and toe, and water contained within the structure)
k_h = seismic coefficient = a / g
g = acceleration of gravity

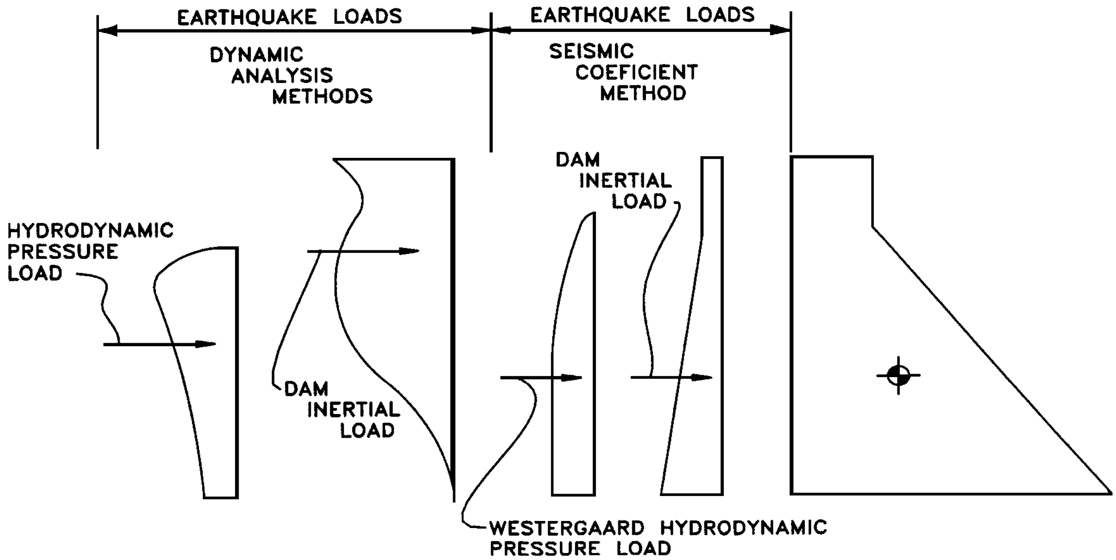

Figure 4-2 Idealized earthquake loads

The horizontal component of the inertial force is assumed to act at the center of mass of the structure, based on the assumption that the structure is a rigid body. In actuality, almost all structures have some flexibility, and the use of the rigid body concept often under estimates the magnitude of the inertial force. The location of the horizontal inertial force is also related to the flexibility of the structure, and usually acts at a location higher than the center of mass. However, because of the cyclic nature of earthquake loads, there is little probability of a rotational-stability related failure.

d. Inertial effects of soil. Backfill material adjacent to a structure will induce inertial forces on the structure during an earthquake. See Chapter 5 and Appendix G for information on soil loads due to earthquakes.

e. Effects of water. Water that is above the ground surface and adjacent to, or surrounding a structure will increase the inertial forces acting on the structure during an earthquake. The displaced structure moves through the surrounding water thereby causing hydrodynamic forces to act on the structure. The water inside and surrounding the structure alters the dynamic characteristics of the structural system, increasing the periods of the fundamental modes of vibration and modifying the mode shapes. In seismic coefficient methods, the hydrodynamic effects are approximated by using the Westergaard method (equation 4-2) (Westergaard 1933). The hydrodynamic force can either increase or decrease the water force, depending on direction of seismic acceleration. Figure 4-3 illustrates hydrodynamic pressures based on the Westergaard method.

$$P_E = (7/12)\, kh\ \gamma_w\, h^2 \tag{4-2}$$

where: P_E = hydrodynamic force per unit length
k_h = horizontal seismic coefficient
γ_w = unit weight of water
h = water depth.
The hydrodynynamic force is added algebraically to the static water pressure force to get the total water force on the structure. The pressure distribution is parabolic and the line of action for the force P_E is 0.4 h above the ground surface. The Westergaard

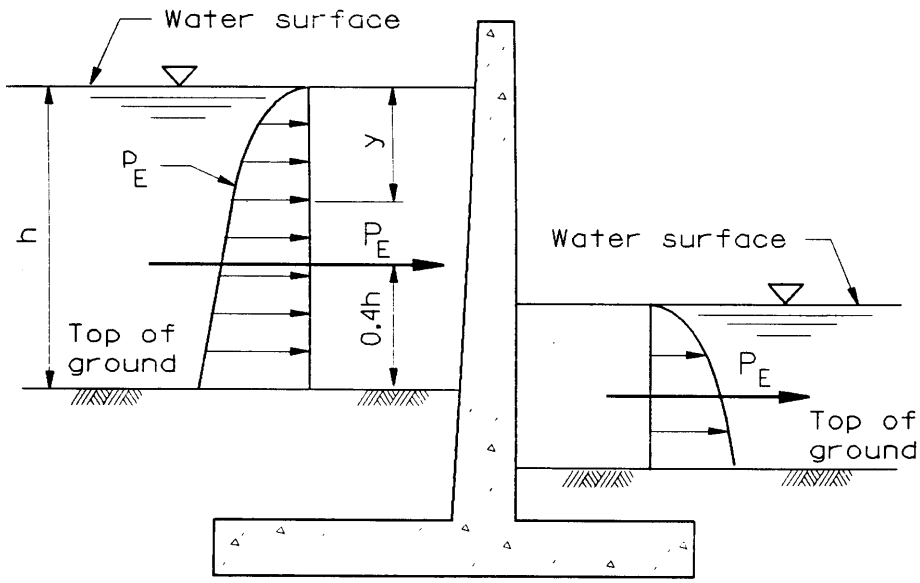

Figure 4-3 Hydrodynamic Forces for Freestanding Water

method assumes the structure is rigid and the water is incompressible. Since most structures are flexible, this method can lead to significant error. For free-standing intake towers, the hydrodynamic effects are approximated by adding mass to the structure to represent the influence of the water inside and surrounding the tower. Engineers using the seismic coefficient approach for stability analyses should be aware of the limitations and the simplifying assumptions made with respect to hydrodynamic pressures and their distribution on the structure.

4-8. Other Loads

a. Impact. Impact loads for locks and dams on navigation systems are due to the structures being struck by barges. These loads can be quite large and for some structures, such as lock guide walls, control the stability analyses. Where impact loads must be considered, refer to EM 1110-2-2602.

b. Ice. Loads due to ice are usually not critical factors in the stability analysis for hydraulic structures. They are more important in the design of gates and other appurtenances. Ice damage to gates is quite common, but there is no known case of a dam failure due to ice. Where ice loads must be considered, refer to EM 1110-2-1612.

c. Debris. Debris loads, like ice loads, are usually of no consequence in stability analyses. However, they may be critical for the design of gates and floodwalls.

d. Hawser pull. Hawser pulls from barges are significant in the stability analysis for lock guide walls, mooring facilities, and floodwalls. Where hawser pulls must be considered, refer to EM 1110-2-2602.

e. Wind. Wind loads are usually small in comparison to other forces, which act on civil works structures. Therefore, wind loads should usually be ignored. For structures such as coastal flood walls where wind might cause instability, or for structures under construction, wind pressures should be based on the requirements of ASCE 7

f. Silt. Silt accumulation can occur upstream of dams. Not all dams will be susceptible to silt accumulation and the structural engineer should consult with hydraulic engineers to determine if silt buildup is possible, and to what extent it may accumulate over time. Silt loads should be included in the loading conditions indicated in Appendix B. Horizontal silt pressure is assumed to be equivalent to that of a fluid weighing 1362 kg/m^3 (85 pcf). Vertical silt pressure is determined as if silt were a soil having a wet density of 1922 kg/m^3 (120 pcf). These values include the effects of water within the silt.

4-9. Mandatory Requirements.

For a general discussion on mandatory requirements, see Paragraph 1-5. As stated in that paragraph, certain requirements within this manual are mandatory. The following are mandatory for Chapter 4.

a. Load Conditions. Stability shall be satisfied for all load conditions listed in Appendix B.

b. Maintenance. Maintenance load conditions shall be classified as *unusual*.

c. Water loading conditions. Water loadings shall be based on hydrologic analyses giving median water elevations in terms of return periods. Water elevations for various load conditions shall be as follows:

- *Coincident pool* shall be the elevation that the water is expected to be at or below for half of the time during each year. This water loading shall be used in combination with seismic loads.
- *Normal operation* loading shall represent maximum loads with a 10-year return period.
- *Infrequent flood* shall represent maximum loads with a 300-year return period.
- *Maximum design flood* shall be the maximum structural loading up to PMF.

d. Uplift loads. Uplift loads shall be calculated per the requirements of Appendix C, as follows:

- Uplift pressures shall be calculated based on an approximate seepage analysis and shall be applied over the full area of the base of the structure, or the failure plane under investigation.
- When a loss of contact is calculated to occur at the heel of the structure, full uplift pressure due to headwater shall be assumed to exist in this area. This provision does not apply to earthquake loading conditions.
- The maximum assumed effectiveness of drainage systems, cutoff wall systems, and combined drain and cutoff wall systems shall be 50%.
- Where overflow results in significant velocities and causes hydraulic jump and retrogression, tailwater pressures used in uplift calculations shall be reduced as described in Appendix C.

e. Earthquake loads.

- Earthquake loads shall be based on design earthquakes specified in ER 1110-2-1806.
- Structural inertia loads shall be calculated using the seismic coefficient method.
- Hydrodynamic loads shall be calculated using Westergaard's formula.
- Soil loads for seismic events shall be calculated per the requirements of Chapter 5.