Chapter 10
Design Criteria

10-1. Applicability and Deviations

The design criteria set forth in this chapter apply in a general sense to the design and analysis of dam structures. Conditions that are site-specific may necessitate variations which must be substantiated by study and testing of both the structure and the foundation.

10-2. Load Cases

a. Usual. Dam structures are designed for usual load conditions, those that occur most commonly during the life of a project, including both normal operating and frequent flood conditions. Basic allowable stresses and safety factors apply in these cases.

b. Unusual. Higher allowable stresses and lower safety factors may be used in accounting for unusual loading conditions such as maintenance, infrequent floods, barge impact, construction, hurricanes, or earthquakes with nonspecific ground motions for OBE (operating basis earthquake). For these conditions, allowable stresses may be increased up to 33 percent. Lower safety factors for pile or foundation capacity may also be used.

c. Extreme. High allowable stresses and low safety factors are used for extreme loading conditions such as accidental or natural disasters that have a remote probability of occurrence and that involve emergency maintenance conditions such as earthquakes with nonsite-specific ground motion for MCE (maximum credible earthquakes). For these conditions, allowable stresses may be increased up to 75 percent. Low safety factors for pile or foundation capacity may be used as described for unusual loads. Special provisions (such as field instrumentation, frequent or continuous field monitoring of performance, engineering studies and analyses, and constraints on operational or rehabilitation activities) are required to prevent catastrophic structure failure during or after extreme loading conditions. Deviations from these criteria for extreme loading conditions should be formulated in consultation with and approved by CECW-ED.

10-3. Earth and Rock Foundations

Generally, an earth- or rock-founded structure is the most cost-effective foundation alternative. A prime consideration in selecting a foundation system is differential settlement. Deflection and differential settlements must be within acceptable limits for the serviceability of the gates and other operating equipment, and adequate stability must be provided. Adequate stability is attained by specific limitations on the magnitude of the foundation pressure (bearing capacity) and the resistance to sliding, and on the location of the resultant of the applied forces within the base of the structure.

a. Foundation pressure.

(1) In general, allowable foundation pressures should not be exceeded for any loading condition; however, the allowable values may be different for usual and extreme load cases. For comparison, only one allowable foundation pressure per material should be used, and an increase of one-third should be allowed for unusual and extreme load case categories. For bearing capacity, EM 1110-1-1905 allows a safety factor of 2.0; however, current practice in the Corps is to use 3.0 for usual load cases and 2.0 for unusual or extreme load cases.

(2) Base pressure computation should be made by uniformly distributing the normal component of the resultant of all forces on the structure (including uplift) as a reaction on the base (or plane of investigation) by means of the general flexure formula or by equations of equilibrium. Uplift should be adjusted in areas of non-compression. Foundation pressure is equal to base pressure plus uplift pressure. Therefore, the stability design should be checked using full uplift forces for overturning and without uplift forces for maximum foundation pressures.

b. Sliding.

(1) Purpose. The purpose of a sliding stability analysis is to assess the safety of a structure against a potential failure due to horizontal movement. The potential for sliding failure may be assessed by comparing the applied shear forces to the available resisting shear forces along an assumed failure surface. A sliding failure is imminent when the ratio of the applied shear forces to the available resisting shear forces is equal to one.

(2) Soil rock shear strength. The shear strength of the soil and/or rock that comprises the foundation (failure surface) is sensitive to the duration of the load, the soil’s ability to drain, the saturation elevation, the number of layers, and many other conditions. Due to these sensitivities, a fully coordinated team of structural, hydraulic, and geotechnical engineers and geologists should be formed to ensure that all pertinent engineering considerations are adequately integrated into the analysis and the correct shear strengths are used.

(3) Analysis model.

(a) The shape of the failure surface may be irregular, depending on the homogeneity of the backfill and the foundation material. The failure surface may be composed of any combination of plane and curved surfaces. However, for simplicity, all failure surfaces are assumed to be planes which form the bases of wedges, as shown in Figure 10-1.

(b) Except for very simple cases, most sliding stability problems encountered in engineering practice are statically indeterminate. To reduce an indeterminate problem to a statically determinate one, the problem must be simplified by dividing the system into a number of rigid body wedges. This division arbitrarily assumes the direction of the equilibrium forces which act between the wedges and neglects any frictional forces between adjacent wedges.

(c) The failure surface can be divided into wedges, as shown in Figure 10-1. In this example, the base of a wedge is formed from a section of the failure surface that lies in a single soil material or along the base of the structure. The interface between any two adjacent wedges is assumed to be a vertical plane which extends from the intersection of the corners of the two adjacent wedges upward to the top soil surface. The base of a wedge, the vertical interface on each side of the wedge, and the top soil surface between the vertical interfaces define the boundaries of an individual wedge.

(d) In the sliding analysis, the dam monolith and the surrounding soil are assumed to act as a system of wedges, as shown in Figure 10-1. The soil-structure system is divided into one or more driving wedges, one structural wedge, and one or more resisting wedges.

(e) Depending on the geologic conditions of the foundation material, the location of the total failure surface or parts of the failure surface may be predetermined. Natural constraints at the site may also predetermine the inclination of some of the failure planes or the starting elevation of the failure planes adjacent to the structure. Conditions which warrant the predetermination of parts of the failure surface include bedding planes and cracks in a rock foundation.

(4) Analysis procedure of the soil-structure system. An iterative procedure can be used to find the critical failure surface. For an assumed factor of safety (FS), the inclination of the base of each wedge is varied to produce a maximum driving force for a driving wedge or a minimum resisting force for a resisting wedge. The assumed FS is varied until a failure surface is produced that satisfies equilibrium. The failure surface which results from this procedure will be the one with the lowest FS. Finite element analysis procedures may also be used.

(5) Sliding factor of safety (FS). Limit equilibrium analysis is used to assess stability against sliding. An FS is applied to the factors which affect the sliding stability and are known with the least degree of certainty. These factors are the material strength properties. An FS is applied to the material strength properties in a manner that places the forces acting on the structure and soil wedges in equilibrium. Because the in-situ strength parameters of rock and soil are never known exactly, one role of the FS is to compensate for the uncertainty that exists in assigning single values to these important parameters. In other words, the FS compensates for the difference between what may be the real shear strength and the shear strength assumed for the analysis. Sliding stability criteria for navigation dams are listed below:

- Usual 2.0
- Unusual 1.7
- Extreme 1.3

(6) Detailed design. Detailed design procedures and multiple wedge derivations can be found in ETL 1110-2-256.

(7) Computer programs. The computer program CSLIDE can assist in performing a multiple wedge sliding analysis.

c. Location of resultant. The location of the resultant of all forces within the base (or limits of the plane of investigation) determines what percentage of the base is in compression. See Figure 10-2. If the resultant lies within the kern (middle third), then the entire base is in compression. This requirement applies to usual load cases. If the resultant lies outside the kern but within the base, then only a portion of the base area is in compression. This portion can be expressed as a percentage of the base computed from the general flexure formula or from the equilibrium equations. At least 75 percent of the base should be in compression for unusual load cases. This measurement is consistent with an eccentricity not exceeding one-fourth of the base length, and it is a suitable approximation for all base shapes. Because the resultant

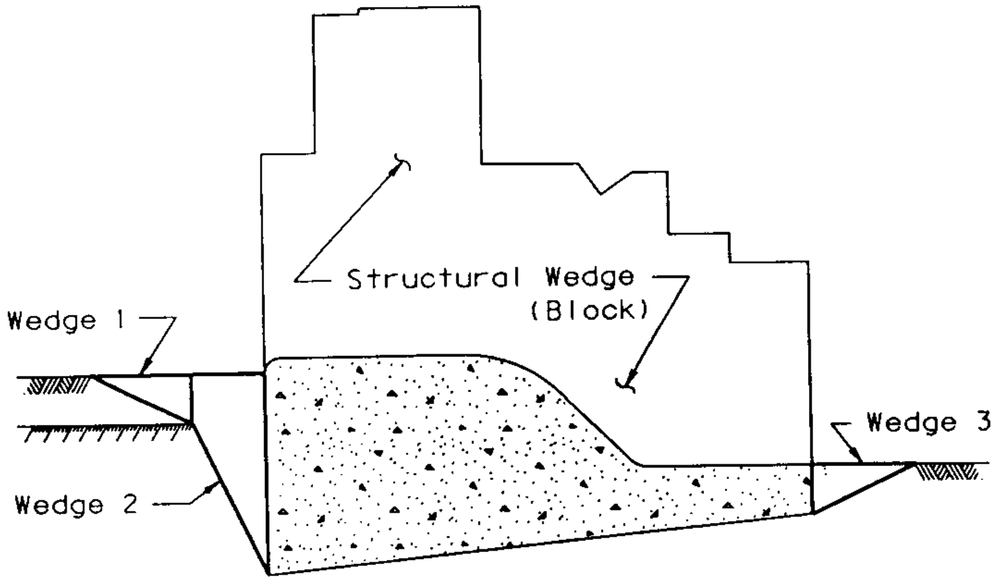

Figure 10-1. Wedge analysis model

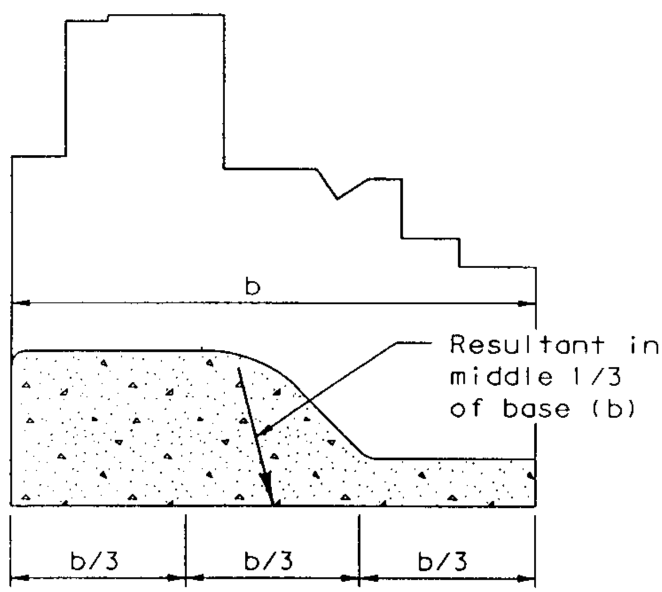

Figure 10-2. Location of resultant

is required to fall within the base only for extreme load cases, the corresponding percentage of the base in compression will be “greater than zero.” These specific requirements are true only for structures of unit width or with rectangular bases and subject to bending about one principal axis; however, these concepts are extended to three-dimensional (3-D) structures with irregular bases and subjected to biaxial bending loads.

d. Settlement. Foundation pressures should not produce total differential settlements that result in operational difficulties (e.g., improper operation of gates and rupture of water stops). In locations where detrimental settlement of dam foundations might occur, a settlement analysis should be made as presented in EM 1110-1-1904. If the settlement analysis indicates a possible concern, the settlement should be corrected by extending the foundation to reduce base pressures, designing alternative foundation (piles), using an alternative site, or using means to ensure that monoliths move together as discussed in Chapter 9.

10-4. Internal Stability

For gravity sections, structural adequacy within the body of a section is attained by limiting internal stresses to values which do not exceed the safe working stresses of the material under stress. Internal instability will be a result of overturning forces and excess pore pressure. Pore pressures may be estimated by methods presented in EM 1110-2-2602. In general, horizontal planes above the foundation are required to have the resultant force inside the kern. For usual load cases, this requirement limits stresses normal to the plane to the compressive range only. For unusual load cases, 75 percent of the base should be in compression when the normal stress component does not exceed the permissible tensile stress for the material (e.g., maximum fiber stress in plain concrete due to factored loads and moments shall not exceed a tensile stress of $0.05f'_c$).

10-5. Uplift and Flotation

These items are closely related in meaning, and both usually act to minimize the degree of structural stability with respect to sliding and overturning. Uplift is determined from seepage analysis by methods presented in Chapter 9. The stability analysis with regard to flotation should be done in accordance with EM 1110-2-2602.

10-6. Pile Criteria

If deflection and differential settlements are not within acceptable limits for the serviceability of the gates and other operating equipment, and adequate stability cannot be attained, a pile foundation should be considered. The pile cap (dam structure) should be modeled as either a rigid block or a flexible base consistent with flexural properties of the pile cap. The pile response is usually based on linear elastic behavior with limiting axial and lateral deflections of 1/4 and 1/2 in., respectively. In general, the FS for axial pile capacity varies between 1.15 and 3, depending on the method of predicting the capacity and the loading condition. Detailed guidance on pile foundation design is provided in EM 1110-2-2906.