Chapter 9
Loads

9-1. General

This chapter contains discussion and guidance on loads that can normally be expected to be imparted to a dam or appurtenant structure. Loads described in this section are those resulting from construction activity: lateral earth pressures; hydrostatic, including uplift and hydrostatic pressures; water above or contained within a structure; dynamic forces from impact or seismic activity; line pull forces; wind, wave, ice, and debris forces; gate, bridge, crane, and bulkhead forces; pressures on sheet pile cutoff walls; forces in monolith joints; superstructure weight; and thermal stresses.

9-2. Construction Loads

a. Cofferdam tie-in loads. Cofferdam tie-in loads are encountered on dam projects that are constructed either within the river in phases or where additional dam structures are provided at an existing project. These loads are caused by either cellular or embankment cofferdams constructed for a phase subsequent to the dam construction which uses a portion of the newly completed or existing structure as a portion of the dewatering cofferdam. Loads not to be overlooked in the design of a cofferdam tie-in include loads that are applied eccentrically and impart a twisting force to the foundation, loads that cause an unsymmetrical load to the dam pier, and concentrated loads due to tie-in details to the dam pier. These loads must be accounted for in the design, but they are normally considered an unusual condition.

b. Construction equipment. Loads due to construction equipment are normally small in comparison to other loads but should be accounted for when they are expected to be present during construction. The loads may result from both moving and stationary equipment.

c. Loads on partially cured concrete. Loads should not be placed on concrete until the concrete has achieved sufficient strength. Intermediate strength requirements should consider proposed loads, including forming and shoring systems, and concrete strength data.

d. Surcharge. In some instances, the design of vertical walls below grade will be affected by wheel loads or other surcharge loads on the ground surface. These loads should be considered in the structural stability calculations and in the detailed design as appropriate. They should be based on the heaviest piece of equipment likely to be placed on the fill during construction.

e. Seepage forces resulting from adjacent dewatering. When the newly completed or existing structure is used as a portion of the dewatering cofferdam, the adjacent dewatering causes seepage forces to act on these structures and the foundation materials below the structures. Research is currently underway at WES to evaluate the magnitude of these forces at the Mel Price Locks and Dam project.

f. Construction shoring. Shoring, or false work, is considered to be any temporary structure which supports structural elements of concrete or other material during their construction or erection. The loads due to this false work and the supported elements must be accounted for in the dam design.

9-3. Lateral Earth Loads

Lateral earth loads can be due to backfill or silt deposition. Careful investigation of available backfill materials and methods of backfilling, as well as the potential for silt accumulation, is of primary importance.

a. Backfill loads. The dam design should include the forces due to dry, naturally drained, saturated, and submerged soil conditions corresponding to the applicable loading condition, i.e., construction operation, maintenance, and so forth. The corresponding angle of internal friction of the proposed backfill material for the conditions expected to result from the proposed field placement method should also be included. Generally, “at rest” pressures should be used for gravity sections on rock and piling foundations. Values for these pressures should be determined for the various conditions of the backfill (drained, saturated, or submerged) by soil analysis methods. The lateral pressure coefficient K_o varies from about 0.3 for loose, granular soil to perhaps 1.0 for compacted clay and fine grained material. The lateral earth pressure coefficient should be determined through consultation with the geotechnical engineer. Factors that affect the lateral earth pressure coefficient are wall and foundation flexibility. See EM 1110-2-2502.

b. Silt and sediment. Horizontal pressures produced by possible silt deposits must be considered in the dam design. Model studies can only indicate tendencies for locations of silt accumulations. If the model studies indicate tendencies for silt build-up, conservative assumptions should be used. Caution should be exercised in assuming that silt and sediment will be removed, because of the possibility that removal may not continue throughout the life of the project.

c. Vertical shear. Vertical shear, or downdrag, is the change in the state of stress in the soil backfill as vertical loading changes. These changes occur during initial construction (backfill is consolidating) and possibly during dewatering (the dewatered monolith tends to move up with respect to the stationary backfill). The procedure for computing the vertical shear is to use a vertical shear coefficient K_v applied to the effective vertical stress of the soil. The vertical shear is then applied at a plane extended at the outermost extremity of the wall. A detailed description of the procedure for gravity walls founded on rock is included in EM 1110-2-2602.

9-4. Hydrostatic

a. Horizontal water pressure. The horizontal water pressure against the dam is variable and depends on the waterway stages that prevail at a particular time or on other conditions which may produce higher pressures. For most monoliths that are not required to resist lateral earth pressures in conjunction with water pressures, the maximum pressures are easily determined. Dam abutment monoliths often have backfill adjacent to one side.

(1) No definite rule can be followed in determining the level of the groundwater in the backfill adjacent to the dam. The saturation level varies between upper and lower pool elevations, depending on the physical characteristics of the backfill material. The location of the saturation line should be based on thorough laboratory tests of the dry and wet characteristics of the soil, the extent of compaction expected, and the effect of local climatic conditions.

(2) A majority of the navigation dams in the United States are located in natural waterways where the backfill material used has granular characteristics. This material has a tendency to drain and become saturated with an approximately straight-line variation between pool elevations. For projects with fairly stable levels, these assumptions should be sufficiently accurate to give satisfactory results. However, varying pool levels and use of impervious backfill material will probably cause considerable departure from straight-line variation.

(3) For dam installations with a lower pool subject to greater fluctuations than the upper pool, a lower pool stage exceeded no more than a small part of the time should be selected from the stage duration curves. In this case, the saturation line can be constructed between this lower pool level and the normal upper pool level, and the height of the ground water table can be determined accordingly for that portion of the dam under consideration. The extent of saturated earth must be established with a reasonable degree of accuracy in order to accurately represent horizontal force due to earth, water, and uplift pressures.

(4) In addition to the usual stabilized groundwater levels caused by normal discharges, extreme loading conditions due to raised saturation levels must also be investigated. These conditions include the effect of locally heavy rains without an accompanying rise in the pool stages and of flood discharges which cause the earth to become saturated. Following flood discharges, the pool levels often approach their normal levels more rapidly than the fill material can drain. Although these increased loads are serious and should be investigated, they are normally of short duration and infrequent occurrence, and the stability requirements are usually relaxed for these increased loads. For a free-draining backfill in this condition, consider the backfill drainage as partially effective and assume, for design purposes, the saturation line to be halfway between its normal location and the top of the fill. The uplift pressure adjacent to the backfill should correspond to the assumed saturation level.

b. Uplift. Uplift due to hydrostatic pressure at the junction between the dam and foundation must be considered. Effective downstream drainage will generally limit the uplift at the toe of the dam to tailwater pressure. The uplift pressure at any point under the structure will be dependent on the presence, location, and effectiveness of foundation drains. Any existing artesian pressures should also be considered. Determination of uplift can be made using the guidance in EM 1110-2-2200. Since uplift may have a relieving effect on foundation loads, the stability of dam monoliths should be investigated for both the maximum and minimum probable uplift pressures. Dewatering systems used for construction can significantly reduce and possibly eliminate uplift and should be thoroughly investigated. A free body diagram of uplift pressures acting on the base of a dam is shown on Plate 18.

(1) Flow net and creep theory. The fundamental design principles and guidance concerning seepage considerations are detailed in EM 1110-2-1901. Additional theory is given in the documentation for the CSLIDE computer program.

(2) Geotechnical investigations. The permeability of the foundation soils greatly affects the uplift pressure;

therefore, close coordination with geotechnical engineers is needed in determining uplift pressures.

c. Vertical water. The downward force of water above the overflow section, stilling basin, flip or roller bucket, and apron should be considered to take the shape of the hydraulic profile. The shape should include the hydraulic jump of the water flowing over the section. Variations in the shape of the hydraulic profile should be investigated based on the loading condition being considered. Any contained water should also be included as a force acting downward.

d. Pulsating pressures. Pulsating pressures against sidewalls and stilling basin slabs are known to exist. The location and magnitude of these pressures can be investigated with a hydraulic model as was done for Baldhill Dam (Fletcher 1993). EM 1110-2-2400 recommends increasing the static load by 1.5 to account for pulsating pressures. These pressures should be accounted for in the design of stilling basin slabs and sidewalls.

9-5. Earthquake or Seismic

Two general approaches to determining seismic forces include the seismic coefficient method and a dynamic analysis procedure.

a. Seismic coefficient method. The seismic coefficient method (also known as the pseudo-static method) of analysis should be used only as a preliminary means of determining the location of resultant and sliding stability of monoliths, or to provide an initial pile layout. If the seismic loads computed by the seismic coefficient method indicate a critical load case, a more rigorous dynamic analysis should be performed as described in Chapter 11. The coefficients used are considered to be the same for the foundation and are uniform for the total height of the monolith wall. Seismic coefficients used in design are based on the seismic zones provided in ER 1110-2-1806. Details of these procedures are contained in EM 1110-2-2200.

b. Dynamic analysis procedure. Procedures for performing a dynamic analysis are contained in ETL 1110-2-365.

9-6. Tow Impact

Tows operating on inland waterways on occasion lose control and collide with dam piers and appurtenant structures. (This load is not applied concurrently with ice load.) The magnitude of the impact forces generated by a particular collision depends on numerous factors such as size, speed, and angle of tow; stiffness of object being struck; and stiffness of barge. An analytical approach which can be used to approximate the maximum impact forces on structures located on the inland waterway system is presented in ETL 1110-2-338.

9-7. Line Loads

When check posts or line hooks are provided on dam structures, a hawser pull of 160 kips should be used for the design of the posts or hooks and their anchorages.

9-8. Ice and Debris

The magnitude of the ice load to be figured into the design of dams should be estimated for the particular structural element being designed, with consideration given to locale and available records of ice conditions. The effect of wedging ice flows between piers should be considered, and the most unfavorable direction of ice load chosen. Accepted practice has been to assume a load of 5,000 lb/ft of width of dam to account for impact of debris and ice loads. For more detailed methods of computing ice forces, see EM 1110-2-1612.

9-9. Wave Loads

Wave loads are usually more important in their effect on gates and appurtenances, but they may in some instances have an appreciable effect on the design of the dam structure. Wave loads are not calculated concurrently with the ice and debris loads. Wave dimensions and forces depend on the extent of water surface or fetch, the wind velocity and duration, and other factors. More information relating to waves and wave pressures is presented in CERC's "Shore Protection Manual" (SPM), Volume 111 (SPM 1984).

9-10. Wind Loads

Wind and subatmospheric pressures may ordinarily be neglected in analyses for low-navigation dams. Wind pressures on the exposed piers, service bridge, crane, etc., should be assumed to act in the most unfavorable direction, and also should be assumed as 30 lb/sq ft (corresponding to a wind velocity of 85 mph). The load should be assumed to act on the following surfaces:

a. Bridge girders - One and one-half times the vertical projection of the span.

b. Bridge trusses - The vertical projection of the span plus any portion of the leeward trusses not shielded by the floor system.

c. Crane - The vertical projection of the crane.

d. Pier - The vertical projection of the exposed parts of the pier and accessories.

For locations where the wind velocities exceed the previously stated velocity, correspondingly higher wind load allowances should be used. TM 5-809-1 contains additional guidance on wind loading.

9-11. Gate Loads

All reactions determined from analysis of the gates should be accounted for in the design of the dam. These include anchorage forces from tainter gates, wickets, hinged-crest gates, and other gates. Internal forces from post-tensioned tainter gate trunnion girders must also be considered in the design of dam piers. Gate piers are subject to eccentric lateral loading, such as that resulting from the raising of one gate while the adjacent gate remains closed. The monolith stability and the pier design should reflect these loading conditions. The torsional shear from this loading should be considered, and the pier should be reinforced appropriately.

9-12. Bridge Loads

The bridge should be designed for dead and live loads including crane and bulkhead loads, as well as wind and seismic loads. Bridge loads are imparted to the dam structure as a result of piers that support a service, access, pedestrian, or highway bridge. Bridge loads should be considered in the stability computations as well as in determining localized stresses in the dam pier.

9-13. Crane Loads

Loads due to cranes and other machinery can be significant and must be included in the analysis of the structure. The loads may result from both moving and stationary equipment, such as bulkhead handling equipment and tainter gate hoists. These loads should be applied as point loads at the appropriate locations.

9-14. Bulkhead Loads

Reactions from maintenance or emergency bulkheads should be accounted for in the design of the dam. Gate piers are subject to eccentric lateral loading, such as that resulting from the dewatering of one gatebay while the adjacent gatebay remains operational. The torsional shear from this loading should be considered, and the pier should be reinforced appropriately.

9-15. Sheet Pile Cutoff Loads

A sheet pile cutoff wall can impart a load to the structure if the sheet pile wall is subject to an unbalanced loading, as in the case where uplift pressures on each side of the sheet pile wall vary due to a pressure relief system or seepage losses. Past analyses showed this load to be negligible in comparison to other loads except in the case of sheet pile driven deep with a high section modulus. In this case, the load should be based on an analysis similar to that presented in EM 1110-2-2602, Chapter 8.

9-16. Monolith Joint Loads

a. Waterstop related. Pressures in monolith joints should be evaluated based on the critical condition of waterstops being either ruptured or intact to give the worst case.

b. Keying between structures. Dam structures are typically designed so each monolith acts as a separate structural unit independent of adjacent monoliths. However, in some cases it may be beneficial to key monoliths together. A typical case is where the upstream end of a stilling basin is keyed to the downstream end of a dam monolith to satisfy stability requirements for the stilling basin. Both monoliths should be designed to withstand all forces that may be transferred across the joint for both stability and local considerations. A typical keyed joint is shown in Plate 2. See paragraph 11-3*f* for additional information on dowels and reinforcing between joints.

c. Reinforcing bars. Dowels or reinforcing bars can be used to prevent differential movement between monoliths, in both the lateral and the longitudinal direction. The reinforcing must be designed to account for the complete load transfer between monoliths, and the monoliths designed to withstand all forces that may be transferred across the joint for both stability and local considerations.

9-17. Superstructure Loads

Superstructure loads are imparted to the dam structure as a result of appurtenant structures such as machinery rooms placed upon the top of the dam piers. Loads from these structures should be considered in the stability computations as well as in determining localized stresses in the dam pier.

9-18. Thermal Loads

Thermal loads may be significant in determining cracking potential for a concrete monolith and also in determining residual stresses which would be additive to seismic loads. This topic is covered in Chapter 12.