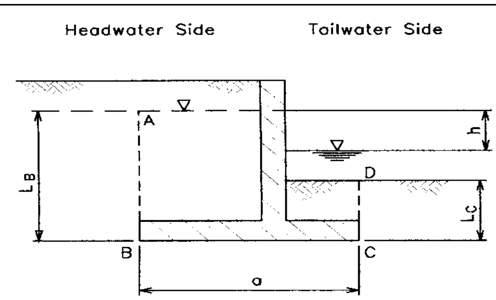
Appendix C
Uplift

C-1. General

This appendix provides a summary of design uplift requirements as they apply to gravity dams, spillway chutes, navigation locks, and other miscellaneous structures. These design uplift requirements will produce conservative designs under most geologic site conditions. The permeability of the foundation soils, or for rock foundations, the permeability of joints, faults, and shear zones in the foundation, greatly affect uplift pressures. Therefore, close coordination with geotechnical engineers is needed in determining uplift pressures. Drainage can be used effectively to reduce uplift pressures. The uplift pressure at any point under the structure will be dependent on the presence, location, and effectiveness of foundation drains. Cutoffs such as grout curtains, impervious blankets, sheet-pile walls, and keys also affect uplift pressures and should be considered in determining design uplift pressures and drainage requirements. Seepage flow net and creep theory can be used to determine uplift pressures for structures on soil foundations. The fundamental design principles and guidance concerning seepage are detailed in EM 1110-2-1901. Uplift pressure is an applied force that must be included in the stability and stress analysis. The uplift pressure will be considered as acting over 100 percent of the base. Uplift pressures are assumed to be unchanged by earthquake loads. Uplift assumptions are valid only if there is adequate resistance to piping. If there is a concern about piping, geotechnical engineers should be consulted.

C-2. Uplift Pressures Due To Seepage

a.. *General.* Where seepage occurs, the pressure heads at points of interest must be obtained from a seepage analysis. Where soil conditions adjacent to and below a structure can be assumed homogeneous (or can be mathematically transformed into equivalent homogeneous conditions), simplified methods such as the line-of-seepage method may be used. However, designers should ensure that water pressures are based on appropriate consideration of actual soil conditions. The line-of-seepage method is illustrated in Figure C-1. The uplift pressures at the ends of the base (points B and C) are estimated by assuming that the head varies linearly along the shortest possible seepage path (ABCD). Where a key is present (Figure C-2), point B is at the bottom of the key, and line BC is drawn diagonally. Permeabilities that are different in the horizontal and vertical directions can be handled by adjusting the length of the different segments along the total seepage path in accordance with the relationship between these different permeabilities.



L = Total length of seepage path = $L_B + a + L_C$

h = Head difference across wall

u_B = Water pressure at B = $\left[L_B - \frac{h\,L_B}{L}\right]\gamma_w$

u_C = Water pressure at C = $\left[L_B - \frac{h(L_B + a)}{L}\right]\gamma_w$

Where γ_w = The unit weight of water

Figure C-1 Line-of-seepage method for water pressures

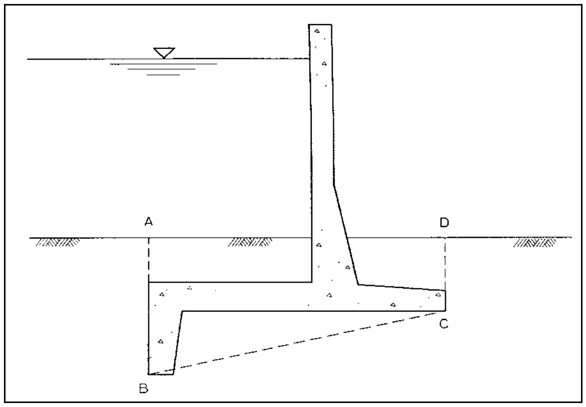
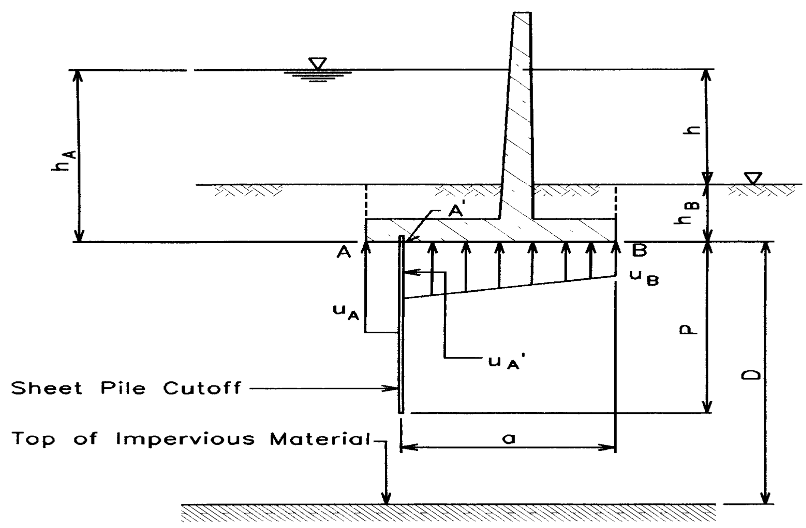


Figure C- 2 Seepage path for wall with key

b. *Uplift calculation for rock foundations.* Seepage beneath structures founded on rock typically occurs in joints and fractures, not uniformly through pores as assumed for soils. Consequently, the assumptions of isotropy and homogeneity and the use of two-dimensional analysis models employed for soil foundations will generally be invalid. Total head, uplift pressure, and seepage quantities may be highly dependent on the type, size, orientation, and continuity of joints and fractures in the rock and the type and degree of treatment afforded the rock foundation during construction. For structures on rock, the total seepage path can be assumed to be length of base that is in compression.

c. Effect of gallery drains on uplift pressures. Drainage is an effective means of reducing uplift pressures for structures founded on rock. It also is effective for structures founded on soils, provided the loss of soil materials through piping can be prevented. For a dam with a line of drains connected to a drainage gallery, uplift downstream of the line of drains can be at or near tailwater, provided the drainage gallery floor is at or below tailwater, the drains are adequately spaced, and they penetrate the pervious strata. For design, it would be unconservative to assume that the aforementioned ideal drainage conditions exist. Normally for design purposes, the drain efficiency is assumed to be 50 percent. Since drainage is such an important factor in reducing uplift pressures, the best policy is to regularly inspect, maintain, and clean the drains to prevent clogging. In cases where it is impossible to clean the drains, a drains-clogged condition should be included as part of the stability analysis for an existing structure. This load condition should be treated as a usual, unusual, or extreme load condition depending on the return period for the load condition being evaluated.

d. Effect of cutoffs on uplift pressures. Cutoffs can also contribute to reducing uplift below structures. Cut-offs can be either grout curtains, concrete trenches, steel sheet piling, or impervious blankets. The effectiveness of cutoffs, however, can be jeopardized by leakage through joints, cracks, and fractures. Therefore, drains are considered to be the most reliable and cost-effective way of reducing foundation-uplift pressures, especially for structures founded on rock. Although grout curtain cutoffs are commonly used in combination with drainage systems for dams founded on rock, the grout-curtain cutoff helps more to reduce drain flows in the drainage gallery than to reduce uplift pressures. Steel sheet-pile cutoffs are not entirely watertight due to leakage at the interlocks, but can significantly reduce the possibility of piping of coarse-grained foundation material. The efficiency of a steel sheet- pile cutoff through a coarse-grained stratum in reducing uplift depends upon conditions at the interlocks, the penetration distance (P) of the cutoff into the pervious stratum, and the depth (D) below the base of the structure to the top of impervious material. When $P \geq 0.95D$, and the pile interlocks are in good condition, an efficiency (E) of 0.50 may be assumed. It has been observed that some steel sheet piling driven into certain types of foundation material (such as gravel) can experience a complete loss of interlock. Before relying on a steel sheet-pile cutoff the designer must be certain that the assumed reduction in seepage will actually occur. A design uplift diagram and method for computing pressures at pertinent points are shown in Figure C-3. In this figure, uplift pressure for that part of the base on the heel side of the cutoff is that due to the full head (no seepage). Seepage is assumed to occur between the cutoff and the toe of the structure, and soil permeabilities are assumed equal in the horizontal and vertical directions.



$$u_A = \text{uplift at } A = \gamma_w h_A$$

$$u_B = \text{uplift at } B = \gamma_w h_B + \frac{h_B}{a + h_B}\left(u_{A'} - \gamma_w h_B\right)$$

$$u_{A'} = \text{uplift at } A' = u_A - E\left(u_A - \gamma_w h_B\right)$$

Where:

γ_w = The unit weight of water

E = Efficiency of sheet pile cutoff

Figure C-3 Uplift pressures with sheet pile cutoff

C-3. Design Uplift for Gravity Dams

a. *General.* A hydraulic gradient between the upper and lower pool is developed between the heel and toe of the dam. The pressure distribution along the base and in the foundation is dependent on the effectiveness of drains and cutoffs, where applicable, and geologic features such as rock permeability, seams, jointing, and faulting.

b. *Without drains.* Where there are no provisions for uplift reduction, the hydraulic gradient will be assumed to vary, as a straight line, from headwater at the heel to zero or tailwater at the toe, assuming the entire base remains in compression between the concrete and the foundation.

c. *With drains.* Uplift pressure at the base, or below the foundation, can be reduced by installing foundation drains. The effectiveness of the drainage system will depend on the depth, size, and spacing of the drains; the character of the foundation; and the facility with which the drains can be maintained. The assumed effectiveness will be limited to no greater than 50 percent, and the design documentation should contain supporting data to justify this assumption. If foundation testing and flow analysis provide supporting justification, the drain effectiveness can be increased beyond 50 percent. Use of a higher assumed drain effectiveness will depend on the pool-level operation plan, instrumentation to verify and evaluate uplift assumptions, and an adequate drain maintenance program. Along the base, the uplift pressure will be assumed to vary linearly from the undrained pressure head at the heel, to the reduced pressure head at the line of drains, to the undrained pressure head at the toe, as shown in Figure C-4. This figure also provides formulas for uplift calculation where the drainage gallery is above or below tailwater.

d. *Grout curtain.* For drainage to be controlled economically, retarding flow to the drains from the upstream head is mandatory. This may be accomplished by a zone of grouting (curtain) or by the natural imperviousness of

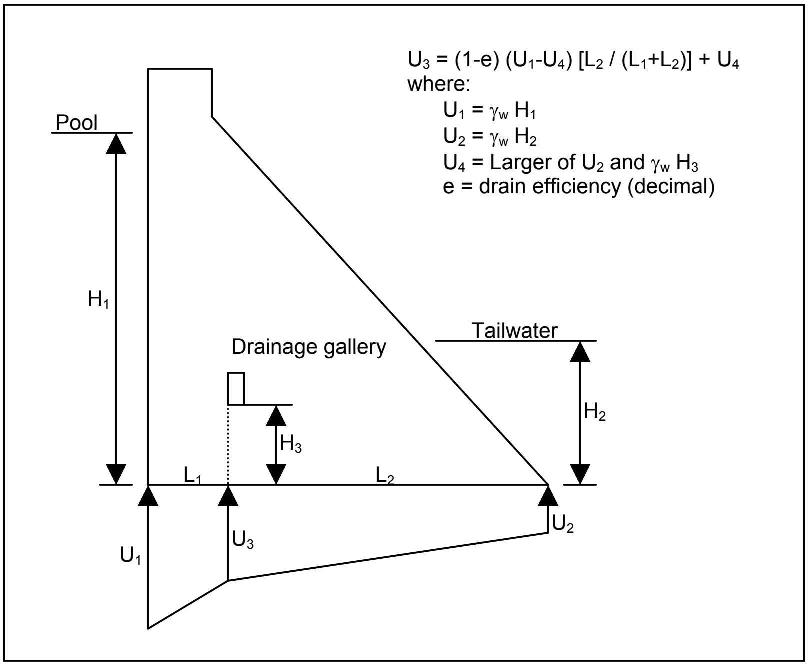


Figure C-4 Uplift distribution with foundation drains

the foundation. A grout curtain should be used wherever the foundation is amenable to grouting. Grout holes should be oriented to intercept the maximum number of rock fractures to maximize its effectiveness. Under average conditions, the depth of the grout zone should be two-thirds to three-fourths of the headwater-tailwater differential and should be supplemented by foundation drain holes with a depth of at least two-thirds that of the grout zone. Where the foundation is sufficiently impervious to retard the flow and where grouting would be impractical, an artificial cutoff is usually unnecessary. Drains, however, should be provided to relieve the uplift pressures that would build up over a period of time in a relatively impervious medium. In a relatively impervious foundation, drain spacing will be closer than in a relatively permeable foundation.

e. Zero compression zones. Any portion of the foundation plane not in compression shall be assumed to behave as an open crack, except where loss of compression is the result of instantaneous loading resulting from earthquake forces. For the length of this crack, uplift pressures shall be assumed as 100 percent of the hydrostatic head of the adjacent face. When the zero compression zone does not extend beyond the location of the drains, the uplift will be calculated using the formula in Figure C-4, but substituting the length of the compression zone as if it was the full length of the base. When the zero compression zone extends beyond the drains, drain effectiveness shall be assumed to be negligible. Uplift shall vary linearly from headwater at the end of the crack, to tailwater at the toe of the dam. When an existing dam is being evaluated, and expensive remedial measures are required to satisfy this loading assumption, the design office should consult with CECW-E to determine if a waiver would be appropriate.

f. Overflow sections. For overflow sections, tailwater pressure must be adjusted for retrogression when flow conditions result in a significant hydraulic jump in the downstream channel, i.e., spillway flow plunging deep into tailwater. The forces acting on the downstream face of overflow sections due to tailwater may fluctuate significantly as energy is dissipated in the stilling basin. Therefore, these forces must be conservatively estimated when used as a stabilizing force in a stability analysis. Studies have shown that the influence of tailwater retrogression can reduce the effective tailwater depth used to calculate pressures and forces to as little as 60 percent of the full tailwater depth. The amount of reduction in the effective depth used to determine tailwater forces is a function of the degree of submergence of the crest of the structure and the backwater conditions in the downstream channel. For new designs, EM 1110-2-1603 provides guidance in the calculation of hydraulic pressure distributions in spillway flip buckets due to tailwater conditions. When tailwater conditions significantly reduce or eliminate the hydraulic jump in the stilling basin, tailwater retrogression can be neglected and 100 percent of the tailwater depth can be used to determine tailwater forces. Full tailwater depth will be used to calculate uplift pressures at the toe of the structure in all cases, regardless of overflow conditions. Figure C-5 illustrates the forces and uplift pressures to be used in stability analysis for an overflow and stilling basin section operating under hydraulic jump conditions.

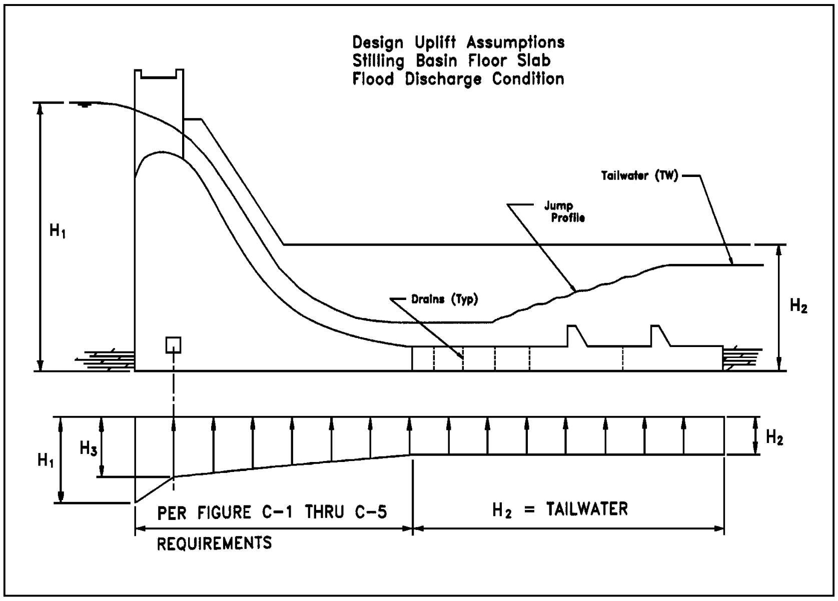


Figure C-5 Stilling basin with discharge condition

C-4. Design Uplift for Navigation Locks

a. General. The problem of uplift for lock walls is complicated by fluctuating water levels within a lock chamber. The rate of change of uplift as the chamber is filled or emptied is not known. The design uplift assumptions used for the stability analysis of lock structures is similar to gravity dams and is described in the following paragraphs.

b. Rock foundations. During construction, uplift acting on the base of any monolith within the cofferdam is assumed to be zero. For walls without drains, uplift will vary linearly from the chamber face to the opposite face; uplift at each face will be 100 percent of the water elevation adjacent to that face (either the current chamber water elevation, the river water elevation or the saturation line in the backfill). In cases where adequate drainage (relieving to tailwater) is provided near the chamber face, total uplift may be reduced for the condition of upper pool in the lock chamber. For river walls, uplift will vary from 100 percent of tailwater plus 50 percent of the difference between headwater and tailwater at the chamber face to 100 percent of tailwater at the river face. For land walls, use the saturation line instead of tailwater. Probably the most effective land wall drainage is that provided in the backfill to reduce the saturation level.

c. Soil and pile foundations. Monoliths on soil or pile foundations usually have cutoff walls and sometimes have drainage systems. At one face of the monolith, uplift should be the full headwater pressure from the face of the wall to the cutoff. At the other face, uplift equals the full tailwater pressure (or the saturation head in the backfill). Uplift pressures between these points should be determined by evaluations of cutoff and drain effectiveness and soil permeability. Cutoffs and drains will normally be designed for 50 percent reduction in uplift, similar to rock foundations. Under excellent conditions, cutoffs and drains can be considered beyond 50 percent effective in reducing uplift pressures, subject to approval from CECW-E. Except for earthquake loading, any portion of the base not in compression will be assumed to sustain a uniform uplift equivalent to 100 percent of the adjacent pool or saturation level. Uplift for loadings which include earthquake forces will be assumed to be equal to that for the same loading without earthquake forces. Because minor movements of gate sills affect the gate operations, all sill blocks should be analyzed for stability resulting from maximum differential heads. Uplift on the sills should be determined similar to the lock walls. The uplift under U-frame locks is complicated by alternative seepage paths along and perpendicular to the lock axis. The permeability of the foundation soils, as well as the existence of sheet-pile cutoff walls and foundation drains, affect the magnitude and distribution of the uplift pressure. Close coordination with geotechnical engineers is needed to determine the uplift pressure for each lock monolith. All combinations of operating and maintenance conditions should be analyzed to determine the most critical condition.

C-5. Design Uplift for Other Structures

The influence that drains and cutoffs have on uplift pressures for other structures such as retaining walls, intake towers, and lined flood-control channels are similar to those described above for gravity dams and navigation locks. A drainage system should be considered for all retaining walls. The benefits of a drainage system are a reduction in hydrostatic pressures in the backfill, lower lateral water pressures on the structure, and a reduction of uplift pressures.