5F Floodwalls and Levees

A properly designed and constructed floodwall or levee can often be an effective device for repelling floodwaters. Both floodwalls and levees provide barriers against inundation, protect buildings from unequalized hydrostatic and hydrodynamic loading situations, and in some cases may deflect floodborne debris and ice away from buildings. However, floodwalls and levees differ in their design, construction, site characteristics, and application.

Floodwalls are structures constructed of manmade materials such as concrete or masonry. The selection of a floodwall design is primarily dependent on the type of flooding expected at the building's site. High water levels and velocities can exert hydrodynamic and hydrostatic forces and impact loads, which must be accounted for in the floodwall design. The composition of any type of floodwall must address three broad concerns:

- overall stability of the wall as related to the external loads;
- sufficient strength as related to the calculated internal stresses; and
- ability to provide effective enclosures to repel floodwaters.

These internal and external forces pose a significant safety hazard if floodwalls are not properly designed and constructed, or their design level of protection is overtopped. Additionally, a tall floodwall can become very expensive to construct and maintain, and can require additional land area for grading and drainage. Therefore, in most instances, residential floodwalls are practical only up to a height of 3 to 4 feet above

NOTE

Under NFIP regulations, floodwalls and levees cannot be used to bring non-compliant structures into compliance.

The NFIP requires that all new construction and substantial improvements of residential structures within Zones A1–30 and AE and AH zones on the community's FIRM have the lowest floor (including basement) elevated to or above the base flood level. (44 CFR 60.3(c)(2)) existing grade, although residential floodwalls can be and are engineered for greater heights.

Levees are embankments of compacted soil that keep shallow to moderate floodwaters from reaching a structure. A well designed and constructed levee should resist flooding up to the design storm flood elevation, eliminating exposure to potentially damaging hydrostatic and hydrodynamic forces.

This chapter outlines the fundamentals of levee and floodwall design and provides the designer with empirical designs suitable to a limited range of situations.

WARNING

Placement of floodwalls in the floodway is not allowed under NFIP regulations. Additionally, floodwalls constructed in Zone V may be subject to wave forces, wave runup, and wave overtopping, so their design and siting must be considered carefully to avoid wave reflection or flow diversion toward buildings. Refer to NFIP Technical Bulletin 5-08, Free-of-Obstruction Requirements (FEMA, 2008c) for additional details.

5F.1 Floodwalls

Floodwalls as a flood mitigation measure in residential areas are not as common as many of the other retrofitting methods detailed in this manual. However, for instances where this measure is appropriate, the following sections provide details on important design and construction considerations.

WARNING

Placement of levees in the floodway is not allowed under local floodplain regulations.

5F.1.1 Types of Floodwalls

Figures 5F-1 and 5F-2 illustrate the use of floodwalls in typical residential applications. Figures 5F-3 and 5F-4 illustrate several types of floodwalls, including gravity, cantilever, buttress, and counterfort. The gravity and cantilever floodwalls are the more commonly used types.

5F.1.1.1 Gravity Floodwall

As its name implies, a gravity floodwall depends upon its weight for stability. The gravity wall's structural stability is attained by effective positioning of the mass of the wall, rather than the weight of the retained materials. The gravity wall resists overturning primarily by the dead weight of the concrete and masonry construction. It is simply too heavy to be overturned by the lateral flood load.

Frictional forces between the concrete base and the soil foundation generally resist sliding of the gravity wall. Soil foundation stability is achieved by ensuring that the structure neither moves nor fails along possible failure surfaces. Figure 5F-5 illustrates the stability of gravity floodwalls. Gravity walls are appropriate for low walls or lightly loaded walls. They are relatively easy to design and construct. The primary disadvantage of a gravity floodwall is that a large volume of material is required. As the required height of a gravity floodwall increases, it becomes more cost-effective to use a cantilever wall.

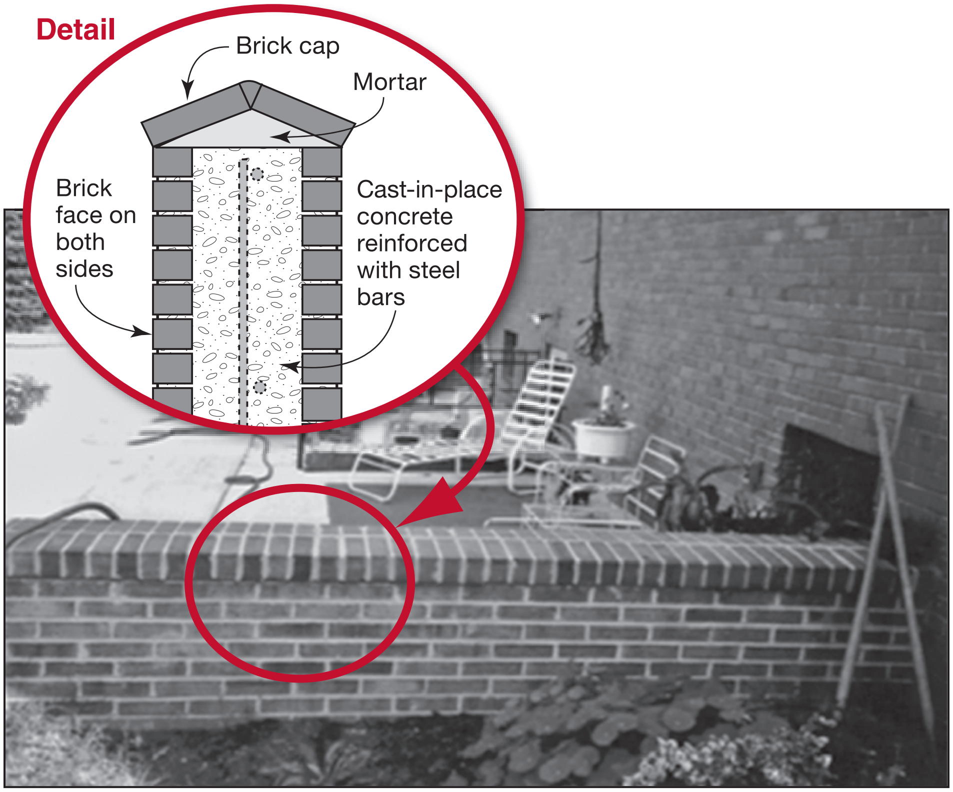


Figure 5F-1.
Typical residential floodwall

Figure 5F-2.
Typical residential floodwall

Figure 5F-3.
Gravity and cantilever floodwalls

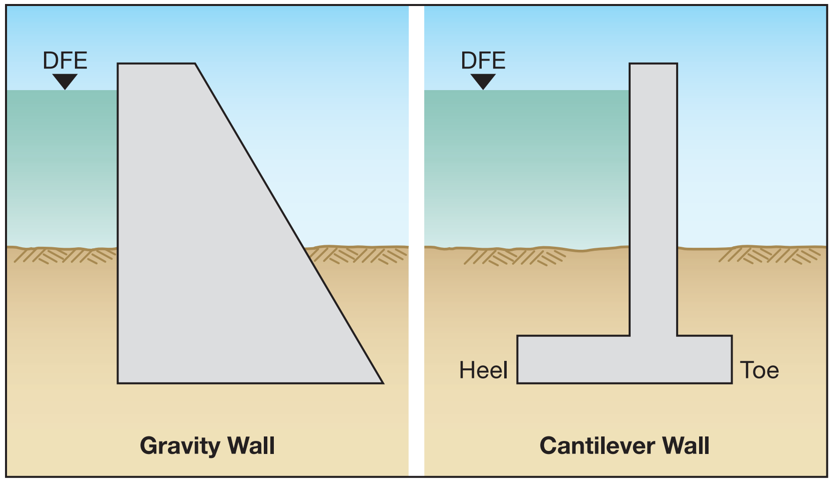
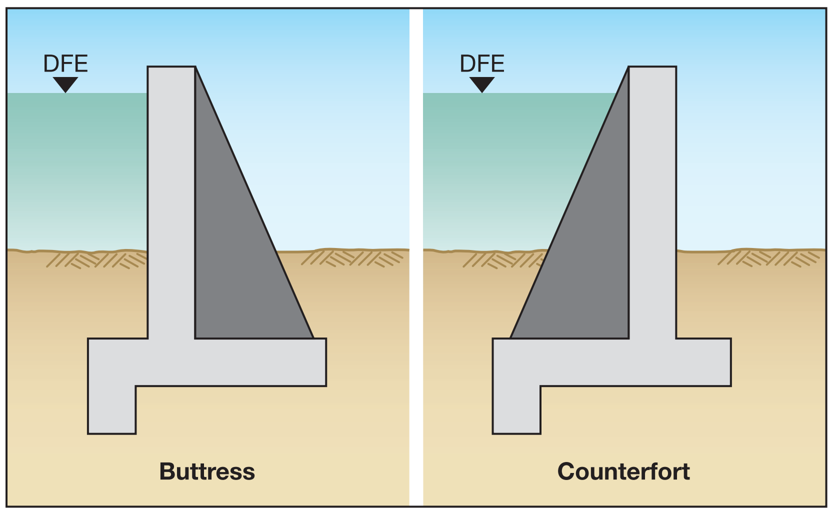


Figure 5F-4.
Buttress and counterfort floodwalls



5F.1.1.2 Cantilever Floodwall

A cantilever wall is a reinforced-concrete wall (cast-in-place or built with concrete block) that utilizes cantilever action to retain the mass behind the wall. Reinforcement of the wall is attained by steel bars embedded within the concrete or block core of the wall (illustrated by Figure 5F-6). Stability of this type of wall is partially achieved from the weight of the soil on the heel portion of the base, as illustrated in Figure 5F-7. The cantilever floodwall is the one most commonly encountered in residential applications.

NOTE

Reinforced concrete provides an excellent barrier in resisting water seepage, since it is monolithic in nature. The reinforcement not only gives the wall its strength, but limits cracking as well.

The floodwall is designed as a cantilever retaining wall, which takes into account buoyancy effects and reduced soil bearing capacity. However, other elements of a floodproofing project (i.e., bracing effects of any slab-on-grade, the crosswalks, and possible concrete stairs) may help in its stability. This results in a slightly conservative design for the floodwall, but provides a comfortable safety factor when considering the

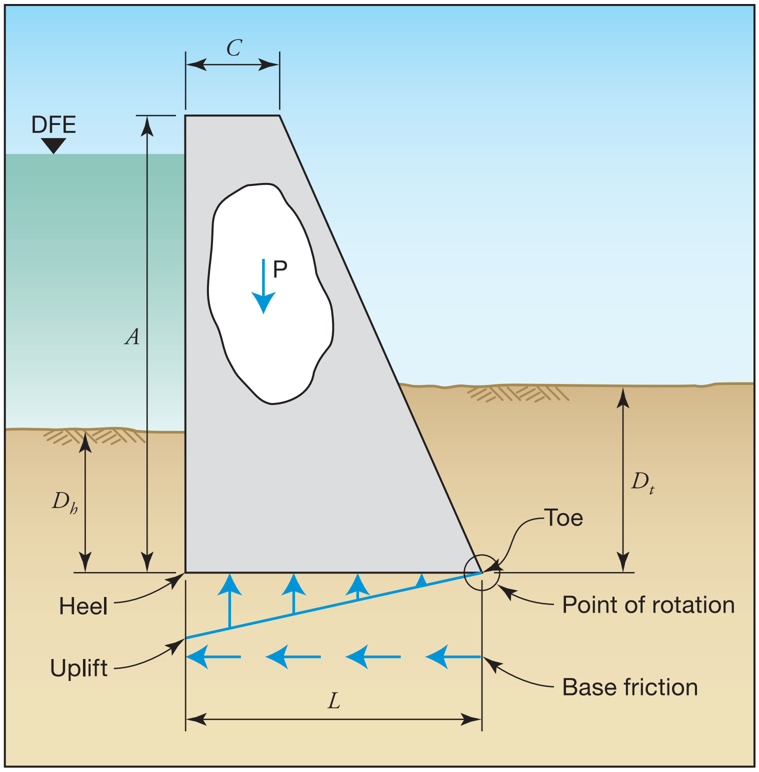


Figure 5F-5. Stability of gravity floodwalls

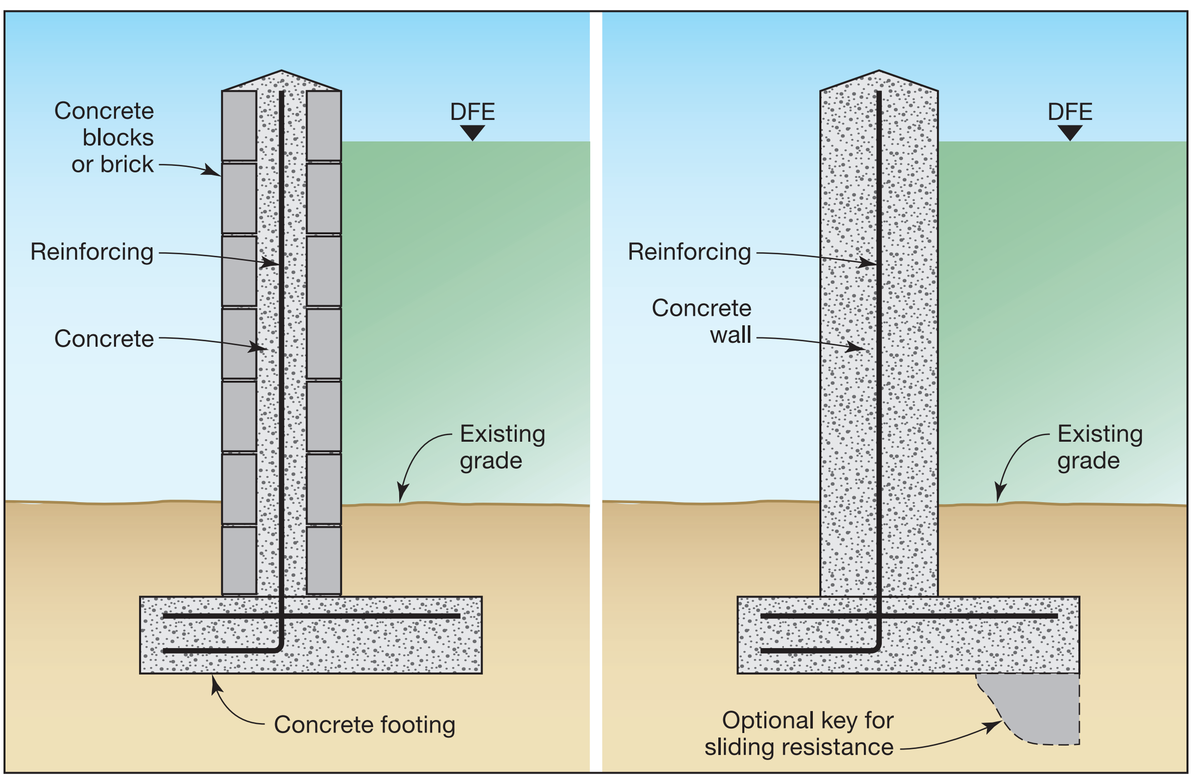
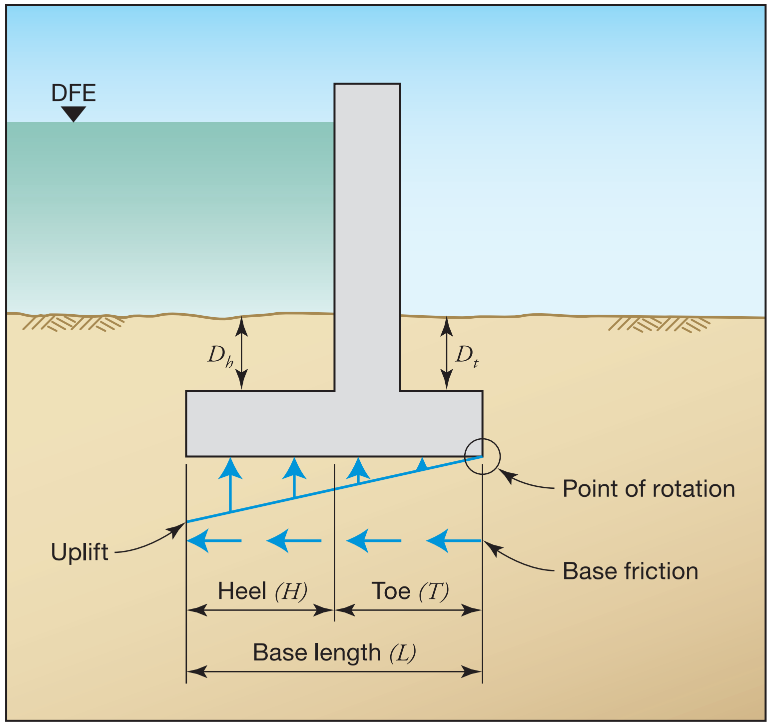


Figure 5F-6. Concrete cantilever floodwall reinforcement

Figure 5F-7.
Stability of cantilever floodwalls



unpredictability of the flood. Backfill can be placed along the outside face of the wall to keep water away from the wall during flooding conditions.

NOTE

Information and details for a standard reinforced concrete floodwall are provided in Case Study 1 in Chapter 6.

The concrete floodwall may be aesthetically altered with a double-faced brick application on either side of the monolithic cast-in-place reinforced concrete center (illustrated in Figure 5F-1). This reinforced concrete core is the principal structural element of the wall that resists the lateral hydrostatic pressures and transfers the overturning moment to the footing. The brick-faced wall (illustrated in Figure 5F-8) is typically used on homes with brick facades. Thus, the floodwall becomes an attractive modification to the home. In terms of the structure, the brick is considered in the overall weight and stability of the wall and in the computation of the soil pressure at the base of the footing, but is not considered to add flexural strength to the floodwall.

When the flood protection elevation requirements of a gravity or cantilever wall become excessive in terms of material and cost, alternative types of floodwalls can be examined. The use of these floodwall alternatives is generally determined by the relative costs of construction and materials, and amount of reinforcement required.

5F.1.1.3 Buttressed Floodwall

A buttressed wall is very similar to a counterfort wall. The only difference between the two is that the transverse support walls are located on the side of the stem, opposite the retained materials.

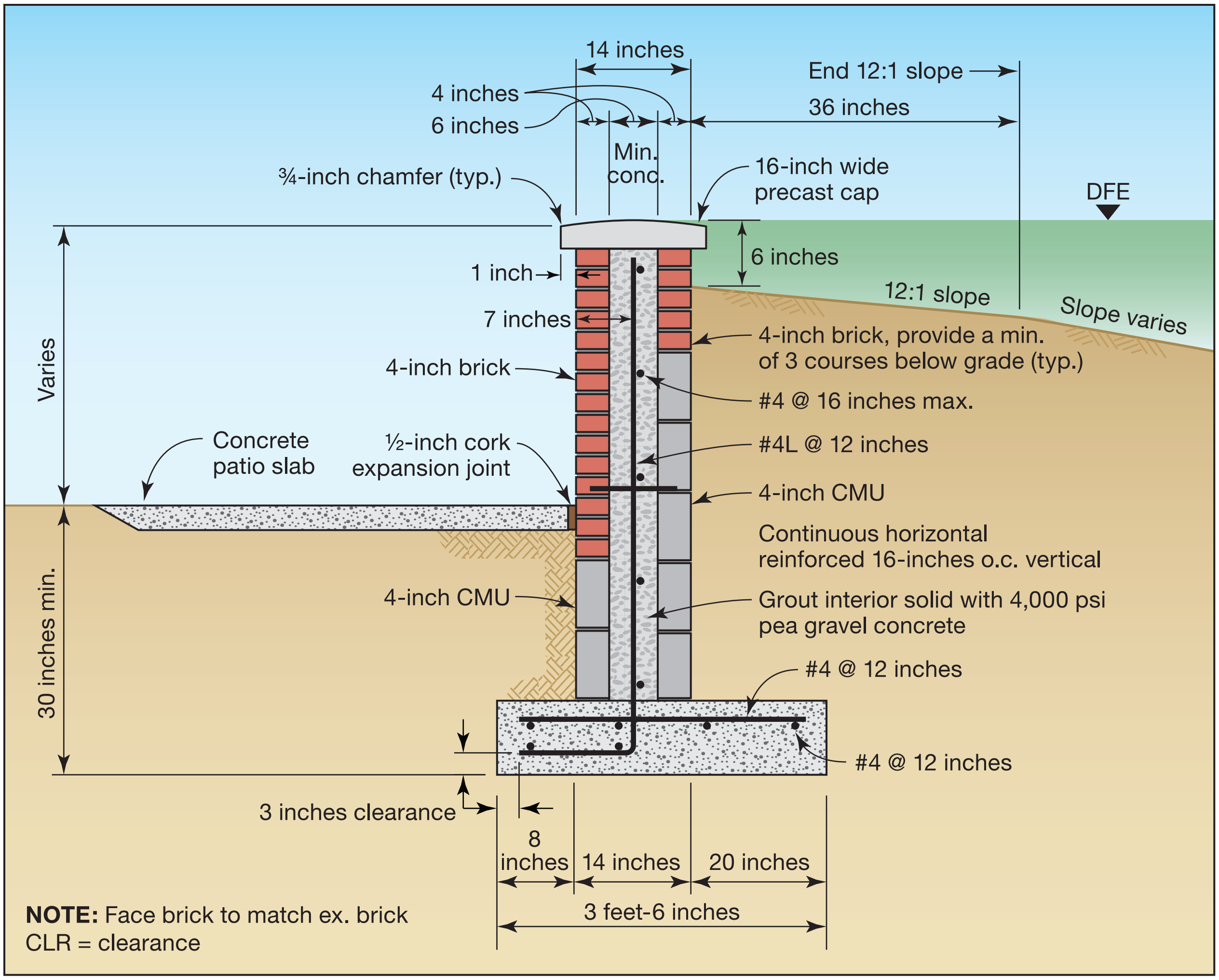


Figure 5F-8. Typical section of a brick-faced concrete floodwall

The counterfort wall is more widely used than the buttress because the support stem is hidden beneath the retained material (soil or water), whereas the buttress occupies what may otherwise be usable space in front of the wall.

WARNING

While the double-faced brick floodwall application is used on either side of concrete block with reinforced and grouted cores, experience has indicated it is not as strong or leak-proof as monolithic cast-in-place applications.

5F.1.1.4 Counterfort Floodwall

A counterfort wall is similar to a cantilever retaining wall except that it can be used where the cantilever is long or when very high pressures are exerted behind the wall. Counterforts, or intermediate traverse support bracing, are designed and built at intervals along the wall and reduce the design forces.

Generally, counterfort walls are economical for wall heights in excess of 20 feet, but are rarely used in residential applications.

5F.1.2 Field Investigation for Floodwalls

Detailed information must be obtained about the site and existing structure to make decisions and calculations concerning the design of a floodwall. The designer should utilize the guidance presented in this chapter where detailed information and checklists for field investigation are presented. Key information to collect includes the low point of elevation survey, topographic and utilities surveys, hazard determinations, local building requirements, and homeowner preferences. Once the designer has developed the above-mentioned low point of entry and site and utility survey information, a conceptual design of the proposed floodwall can be discussed with the homeowner. This discussion should cover the following items:

- previous floods and which areas were flooded or affected by floods;
- a plan of action as to which opening(s) and walls of the structure can be protected by a floodwall and floodwall closures;
- evidence of seepage/cracking in foundation walls, which would indicate the need to relieve hydrostatic pressure on the foundation;
- a plan of action to use a floodwall to relieve hydrostatic pressure on the foundation and other exterior walls;
- the various floodwall options and conceptual designs that would provide the necessary flood protection (obtain consensus on the favored type, size, location, and features of the floodwalls);
- a plan of action as to which utilities need to be adjusted or floodproofed as a result of the floodwall; and
- a plan of action for construction activity and access/egress to convey to the owner the level of disruption to be expected.

The designer of a floodwall should be aware that the construction of these measures may not reduce the hydrostatic pressures against the below-grade foundation of the structure in question. Seepage beneath the floodwall and the natural capillarity of the soil layer may result in a water level inside the floodwall that is equal to or above grade. This condition is worsened by increased depth of flooding outside the floodwall and the increased flooding duration. Unless this condition is relieved, the effectiveness of the floodwall may be compromised. This condition is illustrated in Figure 5F-9.

It is important that the designer check the ability of the existing foundation to withstand the saturated soil pressures that would develop under this condition. The computations necessary for this determination are provided in Chapter 4.

The condition can be relieved by installation of foundation drainage (drainage tile and sump pump) at the footing level and/or by extending the distance from the foundation to the floodwall. The seepage pressures can also be decreased by placing backfill against the floodwall to extend the point where

NOTE

Determination of an appropriate distance from the structure for the floodwall is a function of the depth of the foundation and the soil type. The deeper the lowest level of the structure, the farther away the floodwall should be placed.

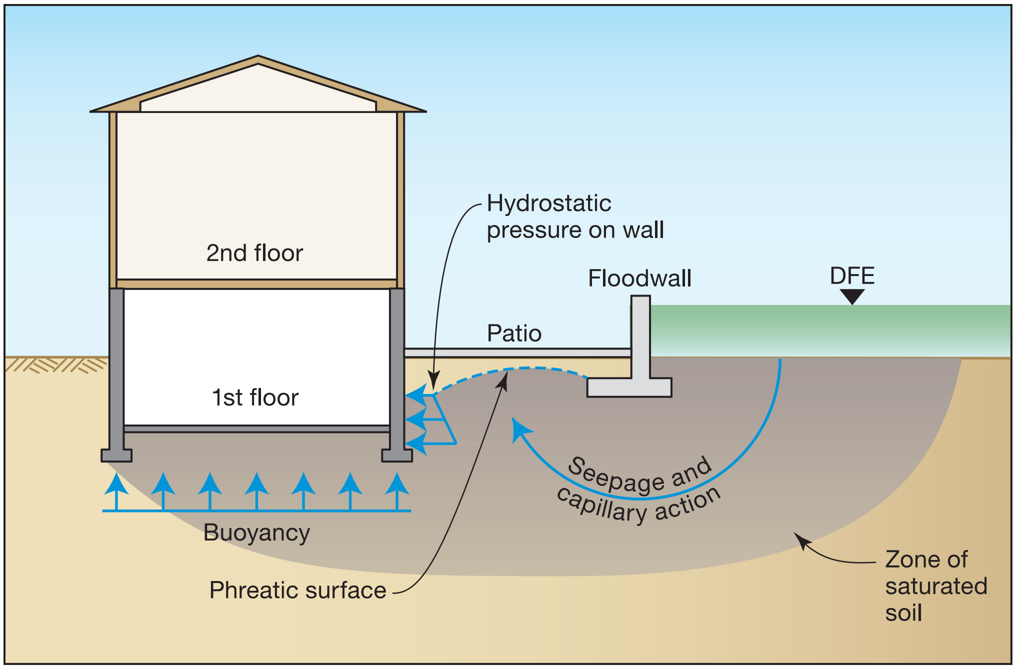


Figure 5F-9. Seepage underneath a floodwall

floodwaters submerge the soil, but the effectiveness of this measure depends on the relative characteristics of the soils in the foundation and the backfill. The design of foundation drains and sump pumps is presented in Section 5D.

Computation of the spacing required to obviate the problem is a complicated process that should be done by an experienced geotechnical engineer. Figure 5F-10 illustrates the change in phreatic surface as a result of increasing the distance between the foundation and the floodwall and/or the installation of a foundation

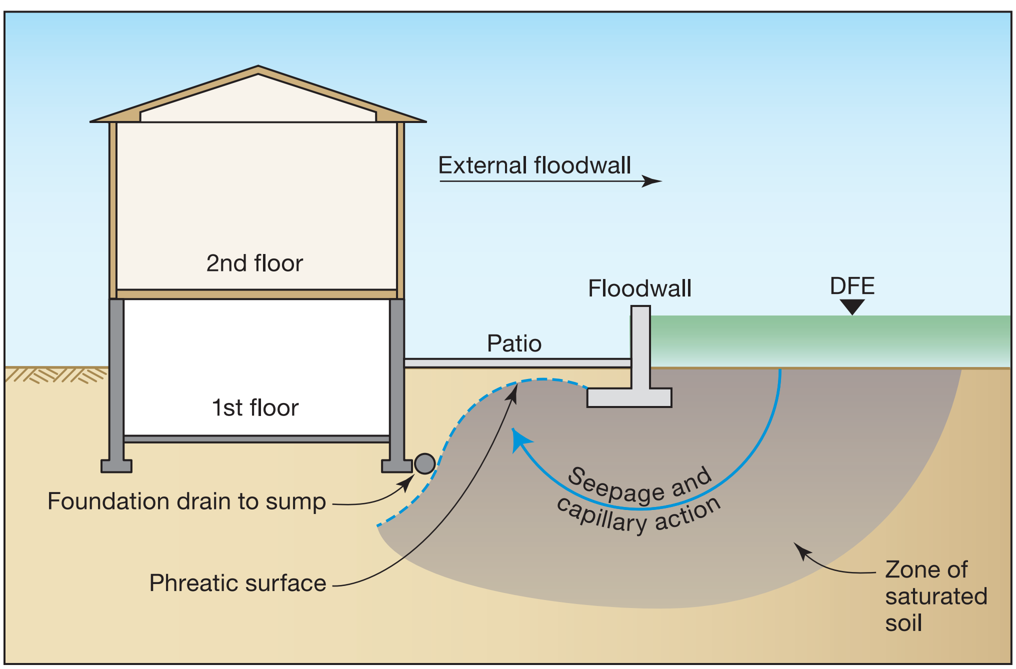


Figure 5F-10. Reducing phreatic surface influence by increasing distance from foundation to floodwall and adding foundation drain

drain and sump pump system. The phreatic surface represents the surface of the water table at or below the ground level.

5F.1.3 Floodwall Design

The design of floodwalls consists of the proper selection and sizing of the actual floodwall and the specification of appurtenances such as drainage systems; waterproof materials to stop seepage and leakage; and miscellaneous details to meet site and homeowner preferences for patios, steps, wall facings, and support of other overhead structures (posts and columns). The following sections describe both a detailed design and a simplified design approach.

WARNING

The permeability of concrete block may necessitate the use of a monolithic core or the application of sealants to eliminate seepage through the wall.

5F.1.3.1 Floodwall Design (Selection and Sizing)

The structural design of a floodwall to resist anticipated flood and flood-related forces follows the eight-step process outlined in Figure 5F-11.

The stability of the floodwall should be investigated for different modes of failure.

Sliding: A wall, including its footing, may fail by sliding if the sum of the lateral forces acting upon it is greater than the total forces resisting the displacement. The resisting forces should always be greater than the sliding forces by a factor of safety (see Figure 5F-12).

Overturning: Another mode of failure is overturning about the foundation toe. This type of failure may occur if the sum of the overturning moments is greater than the sum of the resisting moments about the toe. The sum of resisting moments should be greater than the sum of the overturning moments by a factor of safety (see Figure 5F-13).

Excessive Soil Pressure: Finally, a wall may fail if the pressure under its footing exceeds the allowable soil bearing capacity (see Figure 5F-14).

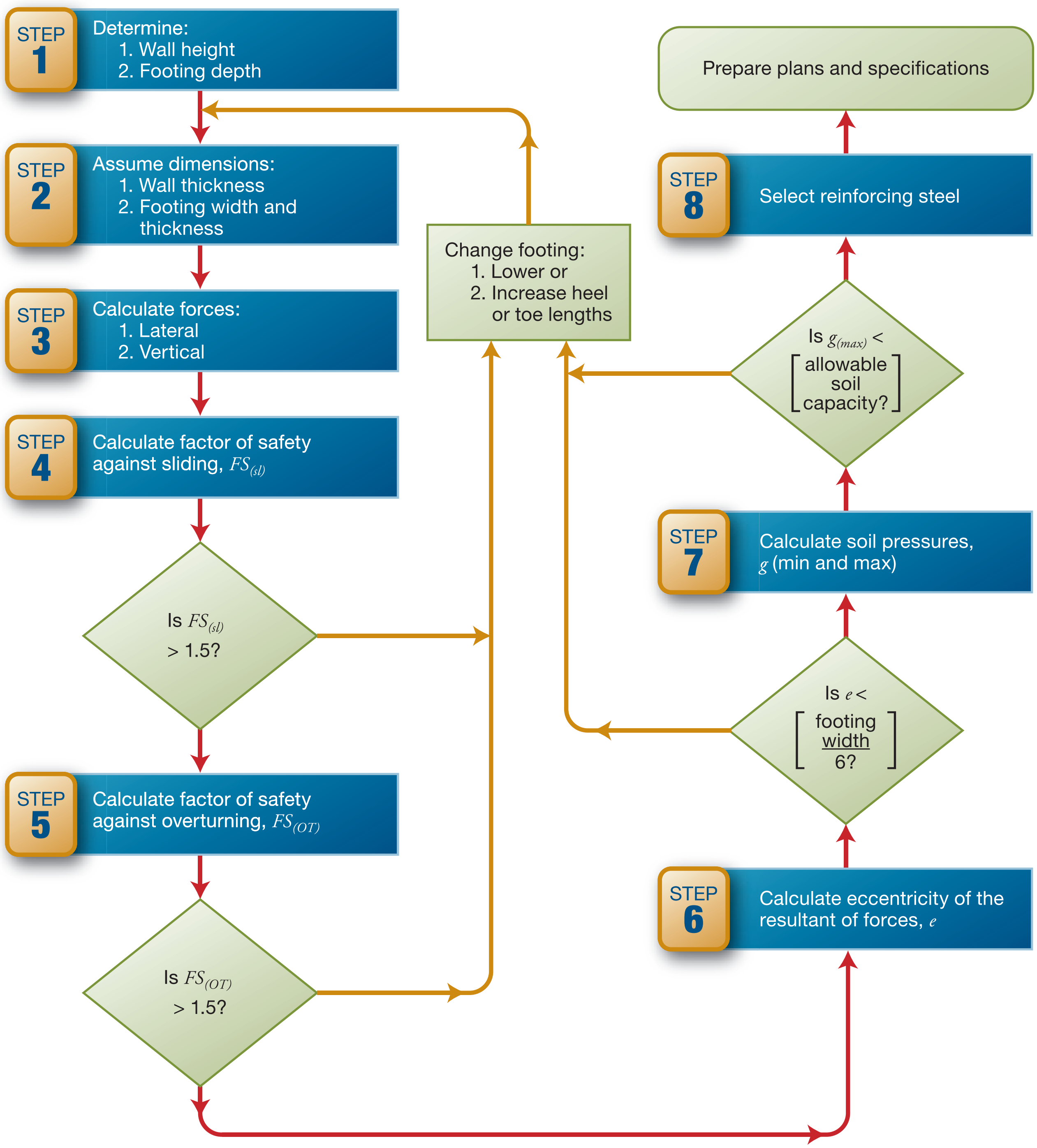


Figure 5F-11. Floodwall design process

Figure 5F-12.
Failure by sliding

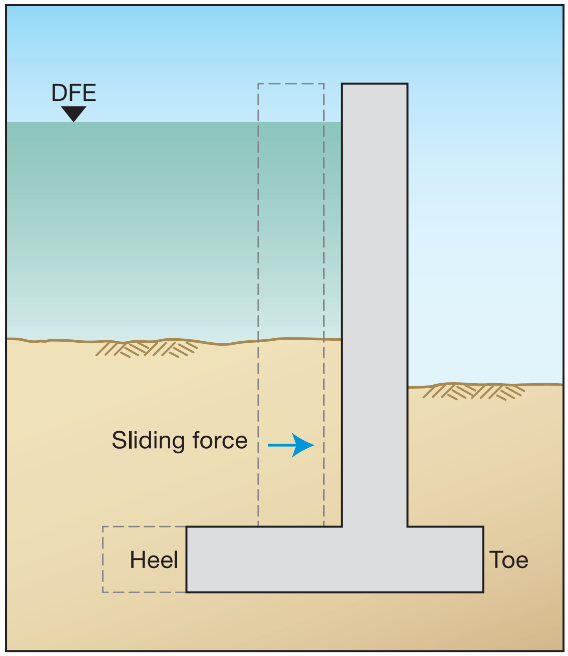
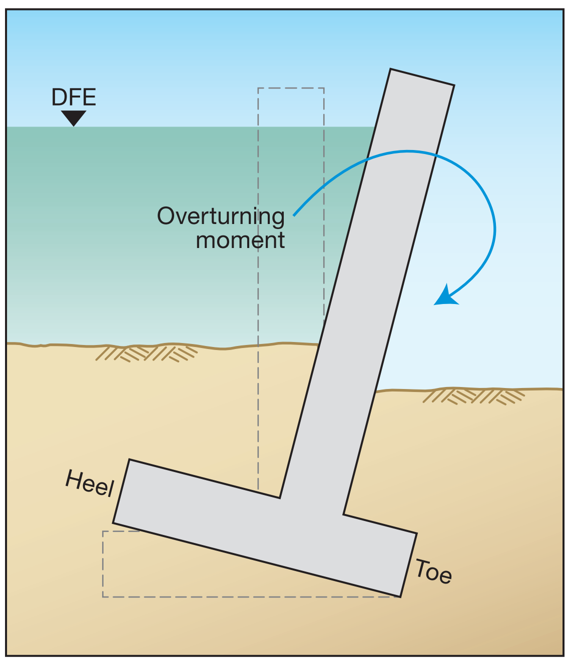


Figure 5F-13.
Failure by overturning



In the following paragraphs, the step-by-step process for completing the structural design of a floodwall is presented, followed by an example illustrating the use of the equations. Note that the floodwall design process is iterative: an initial design is assumed based on experience and past successful designs, checked against design loads and conditions, then revised as needed until all requirements are satisfied by the design.

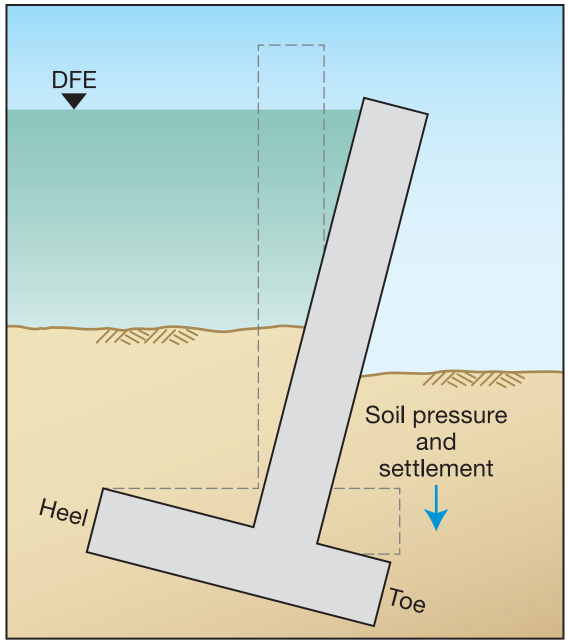


Figure 5F-14.
Failure due to excessive soil pressure

Table 5F-1 provides soil information that is necessary in the computations that follow.

Table 5F-1. Soil Factors for Floodwall Design

USCS Soil Type	Allowable Bearing Pressure, S_{bc} (lb/ft^2)	Coefficient of Friction, C_f
Clean, dense sand and gravel, GW, GP, SW, and SP	2,000	0.55
Dirty sand and gravel of restricted permeability, GM, GM-GP, SM, and SM-SP	2,000	0.45
Firm to stiff silts, clays, silty fine sands, clayey sands and gravel, CL, ML, CH, SM, SC, and GC	1,500	0.35
Soft clay, silty clay, and silt, CL, ML, and CH	600	0.30

USCS = United Soil Classification System

Step 1: Determine wall height and footing depth.

a. Determine wall height based on the DFE or BFE plus 1 foot of freeboard, whichever is greater.

b. Determine minimum footing depth based on the frost depth, local code requirements, and the soil conditions. The footing should rest on suitable natural soil or on controlled and engineered backfill material.

Step 2: Assume dimensions.

Based on the following guidelines or reference to engineering handbooks, assume dimensions for the wall thickness, footing width, and footing thickness.

a. The choice of wall thickness depends on the wall material, the strength of the material, and the height of the wall. Typical wall thicknesses are 8, 12, and 16 inches for masonry, concrete, or masonry/concrete walls.

b. The footing width depends on the magnitude of the lateral forces, allowable soil bearing capacity, dead load, and the wall height. The typical footing width is the proposed wall height. Typically, the footing is located under the wall in such a manner that 1/3 of its width forms the toe and 2/3 of the width forms the heel of the wall as shown in Figure 5F-15. Typical footing thicknesses are based upon strength requirements and include 8, 12, and 16 inches.

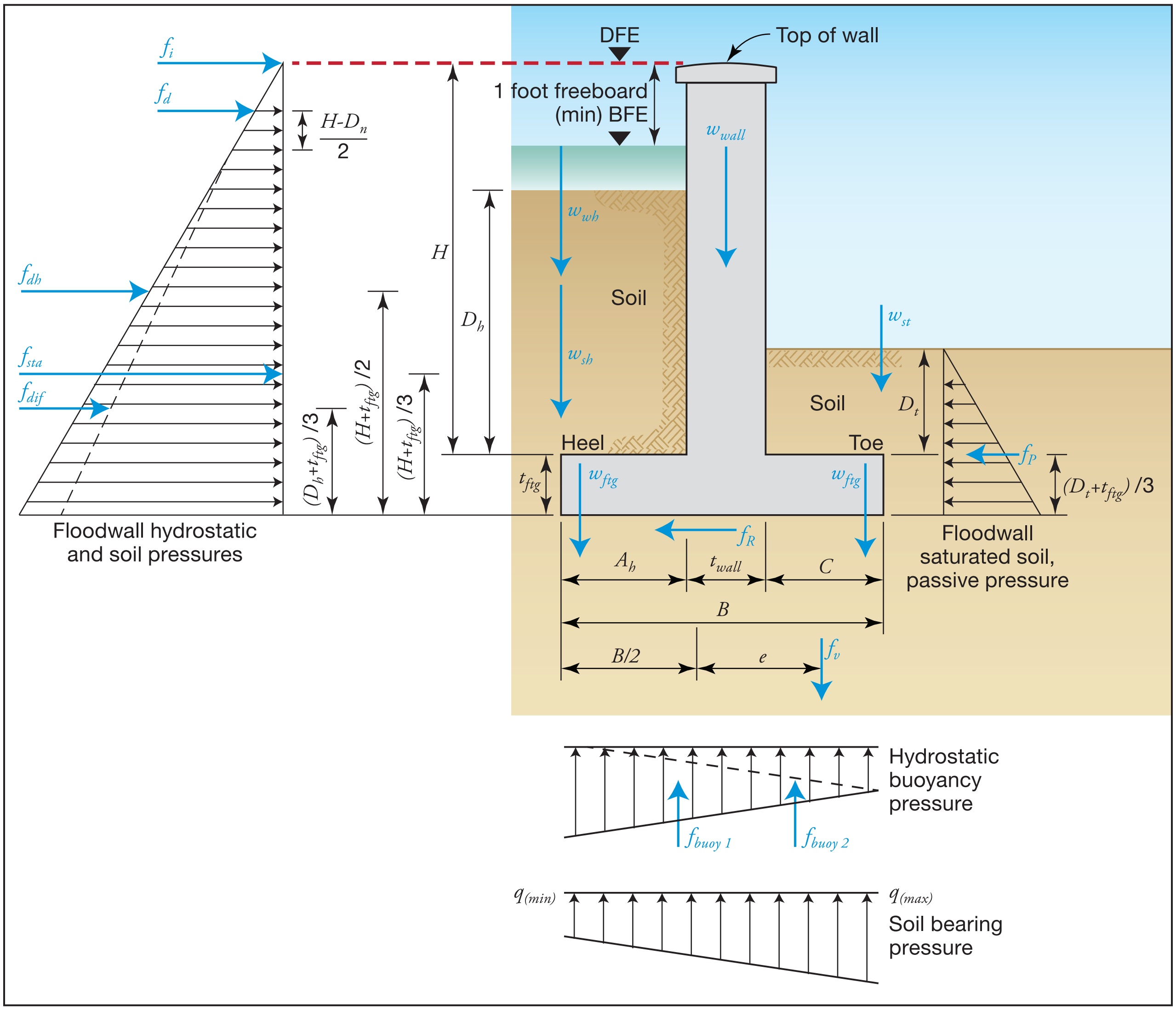


Figure 5F-15. Forces acting on a floodwall

Step 3: Calculate forces.

There are two types of forces acting on the wall and its footing: lateral and vertical. These forces were discussed in Chapter 4 and are illustrated in Figure 5F-15.

a. **Lateral forces:** These forces are mainly the hydrostatic and differential soil/water forces on the heel side of the wall, and the saturated soil force on the toe side of the wall. Hydrostatic and soil forces are as described in Chapter 4.

b. **Vertical forces:** The vertical forces are buoyancy and the various weights of the wall, footing, soil, and water acting upward and downward on the floodwall. The buoyancy force, f_{buoy}, acting at the bottom of the footing is computed as follows:

EQUATION 5F-1: BUOYANCY ON A FLOODWALL

$$f_{buoy} = f_{buoy1} + f_{buoy2} \qquad \text{(Eq. 5F-1)}$$

with f_{buoy1} and f_{buoy2} computed as follows:

$$f_{buoy1} = \gamma_w[(H)(\frac{1}{2}t_{wall}) + (A_h + \frac{1}{2}t_{twall})(t_{ftg})] \text{ (based on Equation 4-7)}$$

$$f_{buoy2} = \gamma_w[(D_t)(\frac{1}{2}t_{wall}) + (C + \frac{1}{2}t_{twall})(t_{ftg})] \text{ (based on Equation 4-7)}$$

where:

f_{buoy} = total force due to buoyancy (lb/lf)

f_{buoy1} = buoyancy force due to hydrostatic pressure at the floodwall heel acting at an approximate distance of B/3 from the heel (lb/lf)

f_{buoy2} = buoyancy force due to hydrostatic pressure at the floodwall toe, acting at an approximate distance of B/3 from the toe (lb/lf)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

A_h = width of the footing above the heel (ft)

C = width of the footing above the toe (ft)

H = floodproofing design depth (ft)

D_t = depth of soil above the floodwall toe (ft)

t_{ftg} = thickness of the floodwall footing (ft)

t_{wall} = thickness of the floodwall (ft)

(Refer to Figure 5F-15)

The gravity forces acting downward are:

- the unit weight of floodwall (w_{wall})

EQUATION 5F-2: FLOODWALL WEIGHT

$$w_{wall} = (H)t_{wall}S_g \qquad \text{(Eq. 5F-2)}$$

where:

w_{wall} = weight of the wall (lb/lf)

H = floodproofing design depth (ft)

t_{wall} = wall thickness (ft)

S_g = unit weight of wall material S (lb/ft^3)

(Refer to Figure 5F-15)

- the unit weight of the footing (w_{ftg})

EQUATION 5F-3: FOOTING WEIGHT

$$w_{ftg} = Bt_{ftg}S_g \qquad \text{(Eq. 5F-3)}$$

where:

w_{ftg} = weight of the footing (lb/lf)

B = width of the footing (ft)

t_{ftg} = footing thickness (ft)

S_g = unit weight of wall material (concrete is 150 lb/ft^3)

(Refer to Figure 5F-15)

- the unit weight of the soil over the toe (w_{st})

EQUATION 5F-4: WEIGHT OF SOIL OVER FLOODWALL TOE

$$w_{st} = C(D_t)(\gamma_{soil}) \quad \text{(Eq. 5F-4)}$$

where:

w_{st} = weight of the soil over the toe (lb/lf)

C = width of the footing toe (ft)

D_t = depth of the soil above the floodwall toe (ft)

γ_{soil} = unit weight of soil (lb/ft^3)

(Refer to Figure 5F-15)

NOTE

The unit weight of soil, γ_{soil} can be obtained from Chapter 4 or from the soil survey, engineering texts, or a geotechnical engineer.

- the unit weight of the soil over the heel (w_{sh})

EQUATION 5F-5: WEIGHT OF SOIL OVER FLOODWALL HEEL

$$w_{sh} = A_h(D_h)(\gamma_{soil} - \gamma_w) \quad \text{(Eq. 5F-5)}$$

where:

w_{sh} = weight of the soil over the heel (lb/lf)

A_h = width of the footing heel (ft)

D_h = depth of the soil above the heel (ft)

γ_{soil} = unit weight of the soil (lb/ft^3)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

(Refer to Figure 5F-15)

- and the unit weight of the water above the heel (w_{wh})

EQUATION 5F-6: WEIGHT OF WATER ABOVE FLOODWALL HEEL

$$w_{wh} = A_h(H)(\gamma_w) \quad \text{(Eq. 5F-6)}$$

where:

w_{wh} = weight of the water above the heel (lb/lf)

A_h = width of the footing heel (ft)

H = floodproofing design depth (ft)

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

(Refer to Figure 5F-15)

The total gravity forces acting downward, w_G, in pounds per linear foot can be computed as the sum of the individual gravity forces computed in Equations 5F-2 through 5F-6:

EQUATION 5F-7: TOTAL GRAVITY FORCES PER LINEAR FOOT OF WALL

$$w_G = w_{wall} + w_{ftg} + w_{st} + w_{sh} + w_{wh} \quad \text{(Eq. 5F-7)}$$

where:

w_G = total gravity forces acting downward (lb/ft)

w_{wall} = weight of wall (lb/ft)

w_{ftg} = weight of footing (lb/ft)

w_{st} = weight of soil over the toe (lb/ft)

w_{sh} = weight of soil over the heel (lb/ft)

w_{wh} = weight of water above the heel (lb/lf)

(Refer to Figure 5F-15)

Therefore, the net vertical force, f_v, is then calculated as:

EQUATION 5F-8: NET VERTICAL FORCE

$$f_v = w_G - f_{buoy} \geq 0 \quad \text{(Eq. 5F-8)}$$

where:

f_v = net vertical force (lb/lf)

w_G = total gravity forces acting downward (lb/ft)

f_{buoy} = total force due to buoyancy (lb/lf)

(Refer to Figure 5F-15)

The net vertical forces in Equation 5F-14 must be greater than or equal to zero. If the value is determined to be less than zero, the designer should change the footing dimensions, then go back to Step 3 and try again (as illustrated in Figure 5F-11).

Step 4: Calculate factor of safety against sliding.

This step involves the computation of the sliding forces, the forces resisting sliding, and the factor of safety against sliding. For a stable condition, the sum of forces resisting sliding should be larger than the sum of the sliding forces.

a. **Sliding Forces:** The sum of the sliding (lateral hydrostatic, hydrodynamic, and impact) forces, f_{comb}, is computed as follows:

EQUATION 5F-9: SLIDING FORCES

$$f_{comb} = f_{sta} + f_{dif} + (f_{dh} \text{ or } f_d) \qquad \text{(Eq. 5F-9)}$$

where:

f_{comb} = cumulative lateral hydrostatic force acting at a distance $H/3$ from the point under consideration (lb/lf)

f_{sta} = lateral hydrostatic force due to standing water (lb/lf)

f_{dif} = differential soil/water force acting due to combined free-standing water and saturated soil conditions (lb/ft)

f_{dh} = equivalent hydrostatic pressure due to low velocity flood flows (lb/ft)

f_d = hydrodynamic force against the structure due to high velocity flood flows (lb/ft)

Note: The computations of f_{sta}, f_{dif}, f_{dh}, and f_d, and are presented in Equations 4-4, 4-5, 4-9, and 4-12. However, for floodwall design, H used in equations 4-4 and 4-9 is replaced with ($H + t_{ftg}$), and D used in equation 4-5 is replaced with ($D_h + t_{ftg}$), where:

H = floodproofing design depth (ft)

D_h = depth of soil above the floodwall heel (ft)

t_{ftg} = thickness of the floodwall footing (ft)

Additionally, the submerged area of the upstream face of the structure (A) in Equation 4-12 is replaced with H to allow the hydrodynamic force f_d to be expressed in lb/lf instead of lbs.

(Refer to Figure 5F-15)

b. **Resisting Forces:** The forces resistant to sliding are the frictional force, f_{fr}, between the bottom of the footing; the cohesion force, f_c, between the footing and the soil; and the soil and the saturated soil force, f_p, over the toe of the footing. These resisting forces are computed as follows:

c. **Frictional Force:** The frictional force, f_{fr}, between the bottom of the footing and the soil is a function of net vertical force, f_v, times coefficient of friction, C_f. The coefficient of friction, C_f, between the base and the soil depends on the soil properties (see Table 5F-1).

EQUATION 5F-10: FRICTIONAL FORCE

$$f_{fr} = C_f f_v \qquad \text{(Eq. 5F-10)}$$

where:

f_{fr} = friction force between the footing and the soil (lb/ft)

C_f = coefficient of friction between the footing and the soil

f_v = net vertical force acting on the footing as was previously presented in Equation 5F-8 (lb/ft)

d. **Cohesion Force:** The cohesion force between the base and the soil, f_c, is obtained by multiplying the width of the footing, B, by the allowable cohesion value of the soil. This allowable cohesion value is usually obtained from a geotechnical analysis of the soil. The cohesion between the footing and the soil may be destroyed or considerably reduced due to contact from water. Due to potentially high variations in the allowable cohesion value of a soil, the cohesion is usually neglected in the calculations; unless the value of cohesion is ascertained by soil tests or other means, it should be taken as zero in the calculations.

EQUATION 5F-11: COHESION FORCE

$$f_c = C_s B \qquad \text{(Eq. 5F-11)}$$

where:

f_c = cohesion force between the base and the soil (lb/ft)

C_s = allowable cohesion force between (lb/ft^2) (usually assumed to be zero)

B = width of the footing (ft)

(Refer to Figure 5F-15)

e. **Saturated Soil Force Over the Toe:** The saturated soil force over the toe, f_p, is calculated as:

EQUATION 5F-12: SATURATED SOIL FORCE OVER THE TOE

$$f_p = \frac{1}{2}[k_p(\gamma_{soil} - \gamma_w) + \gamma_w](D_t + t_{ftg})^2 \qquad \text{(Eq. 5F-12)}$$

where:

f_p = passive saturated soil force over the toe (lb/ft)

γ_{soil} = unit weight of the soil (lb/ft^3)

D_t = depth of the soil over the floodwall toe (ft)

t_{ftg} = thickness of the floodwall footing (ft)

k_p = passive soil pressure coefficient

γ_w = specific weight of water (62.4 lb/ft^3 for fresh water and 64.0 lb/ft^3 for saltwater)

(Refer to Figure 5F-15)

NOTE

The passive soil pressure coefficient, k_p, typically ranges from 2–5. Typical values are 2 for plastic clays, 3 for clayey silts and poorly graded gravels, and 3–4 for well graded soils. Consult a geotechnical engineer for more precise values.

The sum of the resisting forces to sliding, f_R, is calculated as the sum of the individual resisting forces to sliding, computed in Equations 5F-10 through 5F-12, as shown below.

EQUATION 5F-13: SUM OF RESISTING FORCES TO SLIDING

$$f_R = f_{fr} + f_c + f_p \qquad \text{(Eq. 5F-13)}$$

where:

f_R = resisting force to sliding (lb/lf)

f_{fr} = friction force between the footing and the soil (lb/ft)

f_c = cohesion force between the base and the soil (lb/ft)

f_p = passive saturated soil force over the toe (lb/ft)

(Refer to Figure 5F-15)

f. **Factor of Safety Against Sliding:** For the stability of the wall, the sum of resisting forces to sliding, f_R, should be larger than the sum of the sliding forces, f_{comb}. The ratio of f_R over f_{comb} is called the Factor of Safety against sliding, $FS_{(SL)}$, and is calculated as:

EQUATION 5F-14: FACTOR OF SAFETY AGAINST SLIDING

$$FS_{(SL)} = \frac{f_R}{(f_{comb} + f_i)} \geq 1.5 \qquad \text{(Eq. 5F-14)}$$

where:

$FS_{(SL)}$ = factor of safety against sliding (should be greater than 1.5)

f_R = sum of the forces resisting sliding in (lb/ft)

f_{comb} = sum of the sliding forces (cumulative lateral hydrostatic force) (lb/ft)

f_i = normal impact force (lb/ft)

Note: The f_i is expressed in lb/lf based on impact the force F_i (lbs) computed in Equation 4-13 conservatively applied over a unit length of 1 ft.

The factor of safety against sliding in Equation 5F-14 should be at least 1.5. If the factor of safety is determined to be less than 1.5, the designer should lower the footing, increase the amount of fill over the footing, and/or change the footing dimensions, then go back to Step 3 and try again (as illustrated in Figure 5F-11).

Step 5: Calculate factor of safety against overturning.

The potential for overturning should be checked about the bottom of the toe (Figure 5F-5). For a stable condition, the sum of resisting moments, M_R, should be larger than the sum of the overturning moments, M_O. The ratio of M_R over M_O is called the Factor of Safety against overturning, $FS_{(OT)}$.

a. **Overturning Moments:** The overturning moments are due to hydrostatic and hydrodynamic forces, impact loads, saturated soil, and the buoyancy forces acting on the footing. The sum of the overturning moments, M_O, is calculated as:

EQUATION 5F-15: SUM OF OVERTURNING MOMENTS

$$M_O = f_{sta}\left(\frac{(H+t_{ftg})}{3}\right) + f_{dif}\left(\frac{(D_h+t_{ftg})}{3}\right) + f_{buoy1}\left(\frac{2B}{3}\right) + \left[f_{dh}\left(\frac{(H+t_{ftg})}{2}\right) \text{ or } f_d\left((H+t_{ftg}) - \frac{(D_h+t_{ftg})}{2} + (D_h+t_{ftg})\right)\right]$$
$$+ f_i(H+t_{ftg}) + f_{buoy2}\left(\frac{B}{3}\right) \qquad \text{(Eq. 5F-15)}$$

where:

M_O = sum of the overturning moments (ft-lbs/lf)

f_{sta} = lateral hydrostatic force due to standing water (lb/lf) (Eq. 4-4)

f_{dif} = differential soil/water force acting due to combined free-standing water and saturated soil conditions (lb/lf) (Eq. 4-5)

f_{bouy1} = buoyancy force, in lb/lf, due to hydrostatic pressure at the floodwall heel acting at an approximate distance of $B/3$ from the heel (Eq. 5F-1)

f_{bouy2} = buoyancy force, in lb/lf, due to hydrostatic pressure at the floodwall toe, acting at an approximate distance of $B/3$ from the toe (Eq. 5F-1)

f_{dh} = low velocity force (lb/ft) (Eq. 4-9)

f_d = hydrodynamic force (lb/ft) (Eq. 4-12)

f_i = normal impact force (lb/ft) (Eq. 4-13)

B = width of the footing (ft)

H = height of the wall (ft)

D_h = height of the soil above the heel (ft)

t_{ftg} = thickness of the floodwall footing (ft)

Note: The computations of f_{sta}, f_{dif}, f_{dh}, f_d, and f_i are presented in Equations 4-4, 4-5, 4-9, 4-12, and 4-13. However, for floodwall design, H used in equations 4-4 and 4-9 should be replaced with $(H + t_{ftg})$, and D used in equation 4-5 should be replaced with $(D_h + t_{ftg})$. Additionally, the f_i presented in Equation 4-13 is expressed in lb/lf based on impact force F_i (lbs) conservatively applied over a length of 1 ft.

NOTE

When hydrostatic input loads act on the floodwall sections parallel to the flow and the downstream facing wall, Equations 5F-9 and 5F-15 will produce conservative results. Further detailed analysis may result in smaller sections and a corresponding reduction in cost.

b. Resisting Moments: The resisting moments are due to all vertical downward forces and the lateral force due to soil over the toe. The sum of resisting moments, M_R, is calculated as:

EQUATION 5F-16: SUM OF RESISTING MOMENTS

$$M_R = w_{wall}\left(C+\frac{t_{wall}}{2}\right)+w_{ftg}\left(\frac{B}{2}\right)+w_{st}\left(\frac{C}{2}\right)+w_{sh}\left(B-\frac{A_h}{2}\right)+w_{wh}\left(B-\frac{A_h}{2}\right)+f_p\left(\frac{(D_t+t_{ftg})}{3}\right) \qquad \text{(Eq. 5F-16)}$$

where:

M_R	=	sum of the resisting moments in (ft-lbs/lf)	
w_{wall}	=	weight of the wall (lb/lf)	
t_{wall}	=	wall thickness (ft)	
t_{ftg}	=	footing thickness (ft	
w_{ftg}	=	weight of the footing (lb/lf)	(Eq. 5F-3)
B	=	width of the footing (ft)	
w_{st}	=	weight of the soil over the toe (lb/lf)	(Eq. 5F-4)
C	=	width of the footing toe (ft)	
D_t	=	depth of the soil above the floodwall toe (ft)	
w_{sh}	=	weight of the soil over the heel (lb/lf)	(Eq. 5F-5)
A_h	=	width of the footing heel (ft)	
w_{wh}	=	weight of the water above the heel (lb/lf)	(Eq. 5F-6)
f_p	=	passive saturated soil force over the (lb/lf)	(Eq. 5F-12)

c. **Factor of Safety Against Overturning:** As mentioned earlier, for a stable condition, the sum of resisting moments, M_R, should be larger than the sum of the overturning moments, M_O, resulting in a factor of safety greater than 1.0. However, the factor of safety against overturning, $FS_{(OT)}$, computed in Equation 5F-17 should not be less than 1.5. If $FS_{(OT)}$ is found to be less than 1.5, the designer should increase the footing dimensions, then go back to Step 3 and try again (see Figure 5F-11).

EQUATION 5F-17: FACTOR OF SAFETY AGAINST OVERTURNING

$$FS_{(OT)} = \frac{M_R}{M_O} \geq 1.5 \qquad \text{(Eq. 5F-17)}$$

where:

$FS_{(OT)}$ = factor of safety against overturning (should be greater than 1.5)

M_R = sum of the resisting moments in ft-lbs/lf (Equation 5F-16)

M_O = sum of the overturning moments in ft-lbs/lf (Equation 5F-15)

Step 6: Calculate eccentricity.

The final resultant of all the forces acting on the wall and its footing is a force acting at a distance, *e*, from the centerline of the footing. This distance, *e*, is known as eccentricity. The calculation of eccentricity is important to ensure that the bottom of the footing is not in tension. The eccentricity value is also needed for the calculation of soil pressures in Step 7. The eccentricity, *e*, is calculated as:

EQUATION 5F-18: ECCENTRICITY

$$e = \left(\frac{B}{2}\right) - \left(\frac{(M_R - M_O)}{f_v}\right) \qquad \text{(Eq. 5F-18)}$$

where:

e = eccentricity (ft)

B = width of the footing (ft)

f_v = net vertical force acting on the footing (lb/ft) (Eq. 5F-8)

M_O = overturning moment (ft-lbs/lf) (Eq. 5F-15)

M_R = resisting moment (ft-lbs/lf) (Eq. 5F-16)

(Refer to Figure 5F-15)

This eccentricity, *e*, should be less than 1/6 of the footing width. If *e* is found to exceed *B*/6, change the footing dimensions, go back to Step 3, and try again (see flow chart for design of floodwall).

Step 7: Calculate soil pressures.

The soil pressures, q, are determined from the following equation.

EQUATION 5F-19: SOIL PRESSURE

$$q = \left(\frac{f_v}{B}\right)\left(1 \pm \left(\frac{6e}{B}\right)\right) \quad \text{(Eq. 5F-19)}$$

where:

q = soil pressure created by the forces acting on the wall (lb/ft^2)

f_v = net vertical force acting on the footing (lb/lf) (Eq. 5F-8)

B = width of the footing (ft)

e = eccentricity (ft) (Eq. 5F-18)

(Refer to Figure 5F-15)

The maximum value of q should not exceed the allowable soil bearing capacity. The bearing capacity of soil varies with the type of soil, moisture content, temperature, and other soil properties. The allowable values should be determined by a geotechnical engineer. Some conservative allowable bearing values for a few soil types are given in Table 5F-1. If the computed value of q is more than the allowable soil bearing value, increase the footing size, then go back to Step 3 and try again (see Figure 5F-11).

Step 8: Select reinforcing steel.

Select an appropriate reinforcing steel size and spacing to resist the expected bending moment, M_b. Figure 5F-16 illustrates a typical floodwall reinforcing steel installation. The cross-sectional area of steel reinforcing required can be computed using Equation 5F-20. This equation assumes use of steel with a F_y = 60 ksi.

NOTE

The bending moment (M_b) for sizing reinforcing steel in the vertical floodwall component is the product of the lateral hydrostatic force (F_{sta}) and the distance between the point of force application and the bottom of the vertical floodwall component ($H/3 - t_{ftg}$).

EQUATION 5F-20: CROSS-SECTIONAL AREA OF REINFORCING STEEL

$$A_s = \left(\frac{\left(\frac{M_b}{1{,}000} \right)}{1.76 d_f} \right) \qquad \text{(Eq. 5F-20)}$$

where:

A_s = cross-sectional area of reinforcing steel required per foot width of wall (in.^{2})

M_b = bending moment (ft-lbs/lf)

1,000 = factor used to convert ft-lbs to ft-kips

d_f = distance between the reinforcing steel and the floodwall face opposite retained material (in.)

(Refer to Figure 5F-16)

NOTE

d_f is typically the floodwall thickness minus 3½ inches to allow a minimum of 3 inches between the reinforcing steel and the floodwall edge.

NOTE

The selection of reinforcing steel in the footing portion of a floodwall is computed using Equation 5F-20 while modifying M_b for top and bottom steel considerations. For top steel, the moment is the product of the weight of soil and water over the heel (w_{sh}+w_{wh}) and the heel length (A_h) divided by 2.

The selection of bottom steel is a function of the soil bearing pressure. The moment can be computed by adding the soil bearing pressure at the toe edge of the vertical floodwall section to twice the maximum soil bearing pressure ($q + 2q_{max}$) and multiplying this sum by toe length squared over 6 ($C^2/6$). The soil bearing pressure at the toe edge of the vertical floodwall section (q) can be computed by ratio from the calculations (for q_{min}, q_{max}) shown in Step 7.

Using the computed cross-sectional area of reinforcing steel, refer to ACI 318 to select the most appropriate steel reinforcing bar size and spacing.

Refer to Appendix C Example C8 for a sample cantilever floodwall design for a residential building that was developed using this approach.

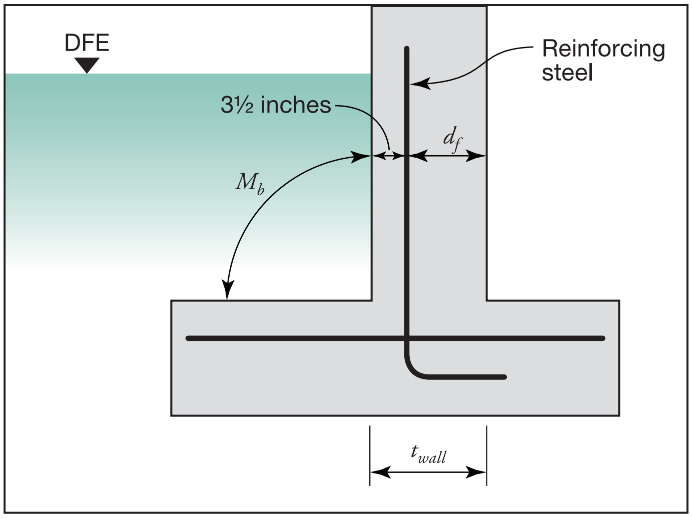


Figure 5F-16: Typical floodwall reinforcing steel configuration

5F.1.3.2 Floodwall Design (Simplified Approach)

Table 5F-2 presents general factors used in developing a standardized approach to floodwall design. If the soil conditions at the site in question do not reflect the assumed conditions below, the standard criteria approach cannot be utilized, and the detailed design process presented in Figure 5F-11 must be used.

Based on the stability requirements (assuming no cohesion), footing dimensions for various wall heights, footing depths, and two different soil types have been calculated. The calculation results are shown in Tables 5F-3 and 5F-4. The designer can utilize the following tables to specify floodwall/footing dimensions required for heights up to 7.0 feet, which reflect flooding levels from 1.0 to 4.0 feet (including a minimum of 3 feet of soil over the footing). Flooding levels can be computed as ($H - D_t$). It is important to note that these dimensions are very conservative and the designer may be able to reduce the dimensions.

In these calculations, the following assumptions have been made:

- wall and footing are of concrete;
- wall thickness (t_{wall}) = 1.0 foot;
- footing thickness (t_{ftg}) = 1.0 foot;
- minimal debris impact potential;
- minimal velocity (<5 feet per second); and
- reinforcing consists of #4 steel bars on 12-inch centers in both the wall and footing.

NOTE

This simplified approach uses assumed site conditions. The designer should be aware that the previous process is normally used in the design of most floodwalls. However, this design process can be shortened for floodwalls of less than 3 feet in height by assuming certain site-specific soil conditions and design parameters. Tables 5F-3 and 5F-4 show typical floodwall design sizes and reinforcement schemes that would be applicable in certain situations. The designer should be aware that, unless the situation in question meets the assumptions and standard design criteria established herein, it would be prudent to complete the entire design process for the floodwall application.

Table 5F-2. Assumed Soil Factors for Simplified Floodwall Design

USCS Soil Type	Allowable Bearing Pressure (lb/ft^2)	k_p, Passive Soil Pressure Coefficient	C_f, Friction Factor	Equivalent Fluid Pressure for Saturated Soil	γ_{soil}, Unit Weight of Soil, (lb/ft^3)
Clean, dense sand and gravel, GW, GP, SW, and SP	2,000	3.70	0.55	75	120
Dirty sand and gravel of restricted permeability, GM, GM-GP, SM, and SM-SP	2,000	3.00	0.45	77	115

USCS = United Soil Classification System

Table 5F-3. Typical Floodwall Dimensions for Clean, Dense Sand and Gravel Soil Types (GW, GP, SW, SP)

Height of Floodwall* H (ft)	Depth of Soil on Water* Side D_b (ft)	Depth of Soil on Water* Side D_t (ft)	Base Width* B (ft)	Heel Width* A_t (ft)	Toe Width C (ft)
3.0	2.0	2.0	2.6	1.0	0.6
4.0	2.0	2.0	4.6	2.6	1.0
	3.0	2.0	4.0	2.0	1.0
	3.0	3.0	4.6	2.6	1.0
5.0	2.0	2.0	6.6	3.6	2.0
	3.0	2.0	6.0	3.6	1.6
	4.0	2.0	5.6	3.0	1.6
	3.0	3.0	4.6	2.6	1.0
	4.0	3.0	4.0	2.6	0.6
6.0	2.0	2.0	9.0	6.6	1.6
	3.0	3.0	7.0	3.6	2.6
	4.0	3.0	6.6	3.6	2.0
	3.0	2.0	8.0	5.0	2.0
	5.0	2.0	7.0	4.0	1.6
	5.0	3.0	6.0	3.6	1.6

* See Figure 5F-15

Table 5F-4. Typical Floodwall Dimensions for Dirty Sand and Gravel of Restricted Permeability Soil Types (GM, GM-GP, SM, SM-SP)

Height of Floodwall* H (ft)	Depth of Soil on Water* Side D_b (ft)	Depth of Soil on Water* Side D_t (ft)	Base Width* B (ft)	Heel Width* A_t (ft)	Toe Width C (ft)
3.0	2.0	2.0	2.6	1.0	0.6
4.0	2.0	2.0	5.0	2.6	1.0
	3.0	2.0	4.6	2.6	1.0
	3.0	3.0	4.0	2.0	1.0
5.0	2.0	2.0	8.0	5.6	1.6
	3.0	2.0	7.6	5.6	1.0
	4.0	2.0	7.0	5.6	0.6
	3.0	3.0	5.6	3.0	1.6
	4.0	3.0	5.0	3.0	1.6
6.0	3.0	3.0	8.0	5.0	2.0
	4.0	3.0	7.0	4.0	2.0
	5.0	3.0	6.6	4.0	1.5

* See Figure 5F-15

5F.1.4 Floodwall Appurtenances

Floodwall appurtenances include drainage systems, stair details, wall facings, patios, existing structure connections (sealants), existing structure support (posts and columns), and closure details. Each will be discussed with illustrations, details, and photographs provided to help the designer develop details that meet the needs of their specific situation. The designer is reminded that it is likely that a local building code may have standards for the design and construction of many of these items.

5F.1.4.1 Floodwall Closures

In designing floodwall closures, many of the principles discussed earlier in Chapter 5D apply. Watertight closures must be provided for all access openings such as driveways, stairs, and ramps, and seals should be provided for all utility penetrations. Figure 5F-17 illustrates typical floodwall closures. Structural analysis for the design of closures should follow the procedures previously outlined for shield design.

As shown in Figure 5F-18, the type of closure used depends primarily on the size of the opening that needs to be protected. This will determine the type of material to be used and how the closure is to be constructed and operated.

Longer and larger closures, such as for a driveway, must be able to withstand significant flood forces and, therefore, should be made of a substantial material. Normally this would be steel plate, protected against rust and corrosion. Heavy aluminum plate may also be used, although it will likely need to be reinforced. In either case, due to the weight of the closure, is usually best that it be hinged so that it can swing into place. Hinging can be located along the bottom so the closure lies flat when not in use, or it can be placed along one side, so the closure can fold back out of the way.

Figure 5F-17.
Typical floodwall closures

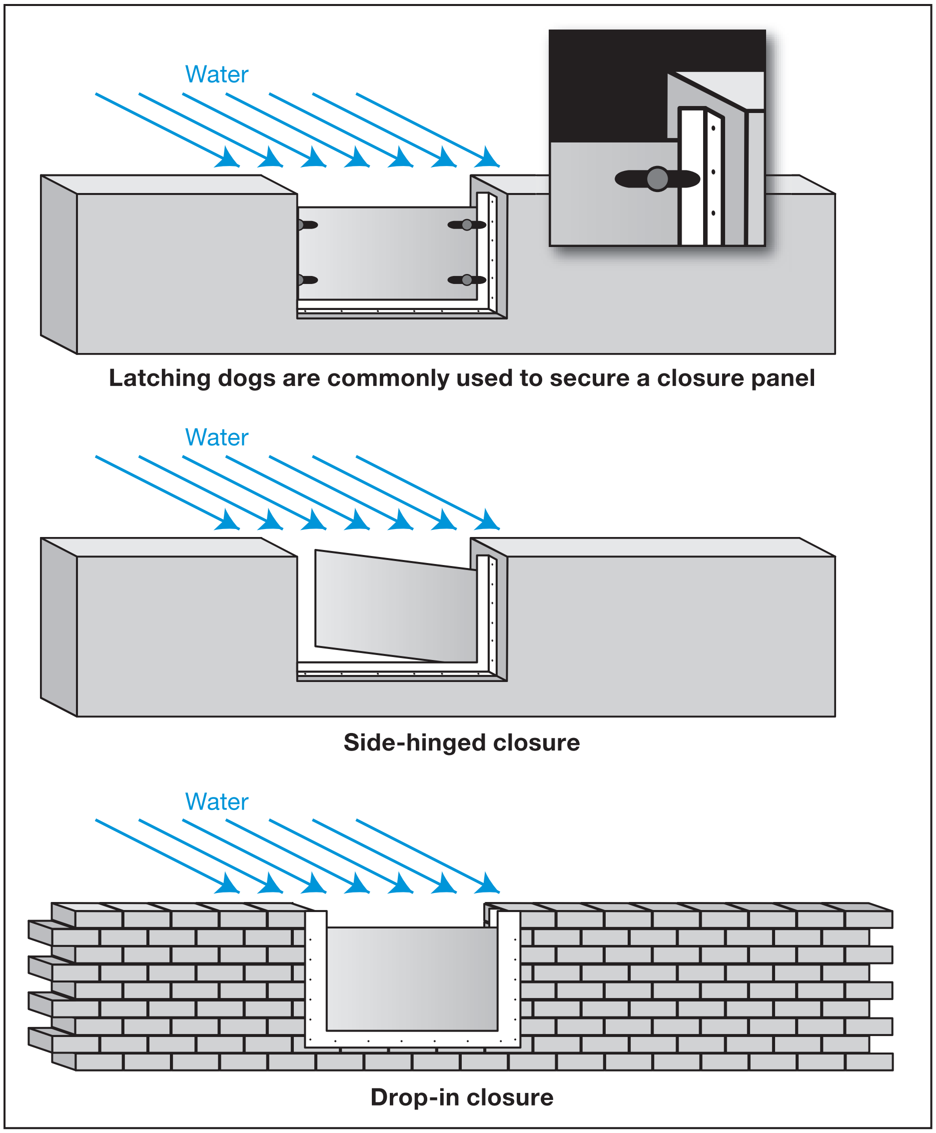
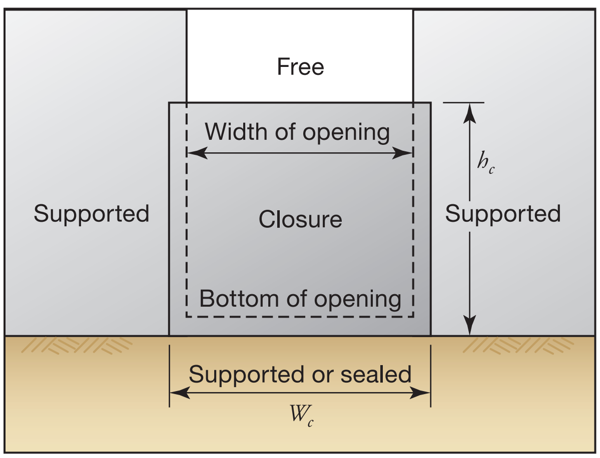


Figure 5F-18.
Closure variables



For normal passage openings, aluminum is probably the most common material used. It is lightweight, allowing for easy fabrication and transport, and is resistant to corrosion. Aluminum can buckle under heavy water pressure, so it may need some additional reinforcement.

For smaller openings, exterior grade plywood is also commonly used. It is relatively inexpensive and is easily fabricated. However, plywood is subject to warping if not properly stored. In addition, it will collapse under relatively low flood forces, and will usually require significant reinforcement, usually some type of wood frame.

Aluminum and plywood are both light enough to be used for temporary closures that can normally be stored in a safe location and installed only when floodwaters threaten. There are many different arrangements that can be used to install these movable closures. The more common methods include the "drop-in" shield that fits into a special slot arrangement and the "bolt-on" shield that is affixed over an opening. There are several different types of hardware that can be used to secure a closure in place, such as T-bolts, wing nuts on anchored bolts, or latching dogs.

It is absolutely essential that closures be made watertight. This is normally accomplished through the use of some type of gasket. Neoprene and rubber are commonly used, but there are a number of other materials readily available that perform equally as well.

The successful performance of a closure system also requires that it be held firmly against the opening being protected. Although the hydrostatic pressure of the water may help to hold the closure in place, floodwater surges can result in negative pressure that can pull off an improperly installed closure.

Whatever material is used, it must be of sufficient strength and thickness to resist bending and deflection failures. The ability of a specific material to withstand bending stresses may be substantially different from its ability to withstand deflection stresses. Therefore, to provide for an adequate factor of safety, the required closure thickness should be calculated twice: first taking into account bending stresses, and second taking into account deflection stresses. The resulting thicknesses should be compared and the larger value specified in the final closure design.

WARNING

Orientation of Openings: It is highly recommended that openings in floodwalls and levees **not** be placed on the upstream side. In the event that they are, Equations 5F-21, 5F-22, 5F-23, and 5F-25 should be modified to include the expected hydrodynamic forces. Closures should not be used on upstream sides where impact loads are expected.

One method of determining the thickness of the closure for steel and aluminum is presented in *Roark's Formulas for Stress and Strain* (Roark and Young, 1989). For a flat plate supported on three sides, the plate thickness required due to bending stresses may be determined by the following equation:

EQUATION 5F-21: PLATE THICKNESS DUE TO BENDING STRESSES

$$t = \sqrt{\frac{P_h + (P_{dh} \text{ or } P_d)W_c^2 \beta}{Max\sigma}} \qquad \text{(Eq. 5F-21)}$$

where:

t = plate thickness (in.)

P_h = hydrostatic pressure due to standing water (psi) from Equation 4-4

W_c = width of closure (in.)

$Max\,\sigma$ = allowable stress for the plate material (from material handbooks) (lb/in.^{2})

β = moment coefficient from Table 5F-5

P_{dh} and P_d = are defined in Equations 4-9 and 4-11, respectively

(Refer to Figure 5F-18)

Similarly, for a steel or aluminum flat plate supported on three sides, the plate thickness required due to deflection stresses may be determined by the following Equation:

EQUATION 5F-22: PLATE THICKNESS DUE TO DEFLECTION STRESSES

$$t = \frac{\sqrt{360\alpha P + (P_{dh} \text{or } P_d)W_c^3}}{E} \qquad \text{(Eq. 5F-22)}$$

where:

α = deflection coefficient from Table 5F-5

E = modulus of elasticity for the plate material (from material handbooks) (lb/in.^{2})

P_h = hydrostatic pressure due to standing water (psi) from Equation 4-4

P_{dh} and P_d = are defined in Equations 4-9 and 4-11, respectively

(Refer to Figure 5F-18)

The variables used in the above equations for plate thickness are illustrated in Figure 5F-18. Table 5F-5 details the moment and deflection coefficients as a function of the ratio of plate height to width.

Table 5F-5. Moment (β) and Deflection (α) Coefficients

Hc/Wc*	0.05	0.67	1.00	1.50	2.00	2.50	3.00	3.50	4.00
α	0.11	0.16	0.20	0.28	0.32	0.35	0.36	0.37	0.37
β	0.03	0.03	0.04	0.05	0.06	0.06	0.07	0.07	0.07

* See Figure 5F-18

Allowable values for α and E may be found for steel plates in the *Steel Construction Manual* (AISC, 2005), and for aluminum plates in the *Aluminum Construction Manual* (AA, 1959).

The method of designing plywood closure plates is similar to that for steel and aluminum closure plates except that the varying structural properties of plywood make using a single equation inappropriate. Because these structural properties are dependent upon the grades of plywood sheet, the type of glue used, and the direction of stress in relation to the grain, determination of the thickness and grade required for a plywood closure is best achieved by assuming a thickness and grade of plywood and calculating its ability to withstand bending, shear, and deflection stresses. This involves calculating the actual bending, shear, and deflection stresses in the plywood closure plate for the thickness and grade specified. These actual stress values are then compared with the maximum allowable bending, shear, and deflection stresses (taken from *Plywood Design Specifications* [APA, 1997]).

WARNING

The designer is referred to *Plywood Design Specifications*, published by the Engineered Wood Association (APA, 1997), for a detailed discussion of design guidelines.

If the actual stresses computed are less than the maximum allowable stresses for bending, shear, and deflection, the thickness and grade specified are acceptable for that application. However, if either of the actual bending or shear stresses or deflection exceeds the maximum allowable values, the closure plate is not acceptable and a new thickness and/or grade of plywood closure plate should be specified and the calculations repeated until all actual stresses are less than the maximum allowed. The following guidance has been prepared to illustrate one method of designing plywood closure plates. Note that a one-way horizontal span is assumed because the variability of plywood properties is dependent upon grain and stress direction.

Compute bending moment on horizontal one-way span (supported on two sides only).

EQUATION 5F-23: BENDING MOMENT

$$M_b = \frac{[P_h + (P_{dh} \text{ or } P_d)]W_c^2}{8} \geq 1.5 \qquad \text{(Eq. 5F-23)}$$

where:

M_b = bending moment in (in.-lbs/in.);

P_h = hydrostatic pressure due to standing water (psi) from Equation 4-4

W_c = width of the closure (in.)

P_{dh} and P_d = are defined in Equations 4-9 and 4-11, respectively

(Refer to Figure 5F-18)

Check bending stress.

EQUATION 5F-24: BENDING STRESS

$$f_b = \frac{M_b}{KS} \qquad \text{(Eq. 5F-24)}$$

where:

f_b = bending stress (lb/in.^{2})

M_b = bending moment (in.-lbs/in.)

KS = effective section modulus from a reference (in.^{3}/in.)

If the calculated bending stress for the specified plate (f_b) is less than the maximum bending stress allowed (F_b) by the plate manufacturer, the closure plate is adequately designed for bending applications. If not, the closure should be redesigned and the calculation repeated.

Compute shear force.

EQUATION 5F-25: SHEAR FORCE

$$V_s = \frac{[P_h + (P_{dh} \text{ or } P_d)]W_c^{\,2}}{2} \qquad \text{(Eq. 5F-25)}$$

where:

V_s = shear force (lbs)

P_h = hydrostatic pressure due to standing water from Equation 4-4 (lb/in.^{2})

W_c = width of the closure plate (in.)

P_{dh} and P_d = are defined in Equations 4-9 and 4-11, respectively

(Refer to Figure 5F-18)

Check shear stress.

EQUATION 5F-26: SHEAR STRESS

$$F_s = \frac{V_s}{C_{RS}} \qquad \text{(Eq. 5F-26)}$$

where:

F_s = shear stress (lbs)

V_s = shear force (lbs)

C_{RS} = rolling shear constant dimensionless

If the calculated shear stress for the specified plate (F_s) is less than the maximum shear stress allowed ($F_{s\ (allowable)}$), the closure plate is adequately designed for shear applications. If not, the closure should be redesigned and the calculations repeated.

Compute deflection for a single one-way span.

EQUATION 5F-27: PLATE DEFLECTION FOR A ONE-WAY SPAN

$$\Delta_b = \frac{[P_h + (P_{dh} \text{ or } P_d)](W_c + y)^4}{921.6(E)(I)} \qquad \text{(Eq. 5F-27)}$$

where:

Δ_b = computed deflection (in.)

P_h = hydrostatic pressure from Equation 4-4 (lb/in.^{2})

W_c = unsupported width (in.)

y = support width factor (in.)

E = Modulus of Elasticity (lb/in.^{2})

I = Effective Moment of Inertia (in.4/ft)

P_{dh} and P_d = are defined in Equations 4-9 and 4-11, respectively

(Refer to Figure 5F-18)

Check deflection.

A customary and acceptable level of deflection may be expressed as:

EQUATION 5F-28: ALLOWABLE DEFLECTION

$$\Delta_{b\ (allowable)} = \frac{W_c}{240} \qquad \text{(Eq. 5F-28)}$$

where:

$\Delta_{b\ (allowable)}$ = allowable deflection (in.)

W_c = unsupported width (in.)

(Refer to Figure 5F-18)

If the calculated deflection (Δ_b) is less than the allowable deflection ($\Delta_{b\,(allowable)}$), the closure plate is adequately designed for deflection situations. If not, the closure should be redesigned and the calculations repeated.

Closure plates of plywood are limited to short spans and low water heights. It should also be noted that most plywood will deteriorate when exposed to high moisture. Therefore, plywood closure plates should be periodically examined and replaced as necessary.

5F.1.4.2 Drainage Systems

When designing a floodwall system, the designer must verify that it will not cause the flooding of adjacent property by blocking normal drainage. Specific information and local requirements can be obtained from the local zoning commission, building inspector, or water control board. Before deciding on a design, the designer should check local building codes, floodplain and/or stormwater management ordinances, zoning ordinances, or property covenants that may prohibit or restrict the type of wall planned.

The flood protection design should be developed to divert both floodwater and normal rainfall away from the structure. By directing the floodwater and rainfall away from the structure, the designer can minimize potential erosion, scour, impacts, and water ponding. Typical design provisions include:

- regrading the site;
- sloping applications; and
- drainage system(s).

Regrading the site basically involves contouring. The surface can be contoured to improve the drainage and minimize floodwater turbulence. Ground covers or grasses, especially those with fibrous root systems, can be effective in holding soil against erosion and scour effects of floodwaters.

Sloping applications include providing a positive drainage for engineered applications such as patios, sidewalks, and driveways. The material is slightly inclined, typically at a 1 percent to 2 percent grade, to an area designed for collection, which includes inlets, ditches, or an existing storm drain pipe system. Figures 5F-19 and 5F-20 show two patio drainage options, and Figure 5F-21 shows a floor drain section typically used to provide positive drainage for patio areas enclosed by floodwalls. These configurations can also be used with sump and sump pump installations.

Drainage systems are a series of pipes that collect and route interior drainage to a designated outfall. Usually the drainage operation is underground and works through a gravity process. However, when grading and sloping will not allow the gravity system to function, provisions for a pumping method, such as a sump pump, should be made. Information on the design of sumps and sump pump applications is provided in Chapter 5D.

For example, in its simplified form, a gutter and downspout outlet, which can be found on almost all houses, is a type of storm drainage system. Provisions at the downspout outfall should also be developed in the site drainage design.

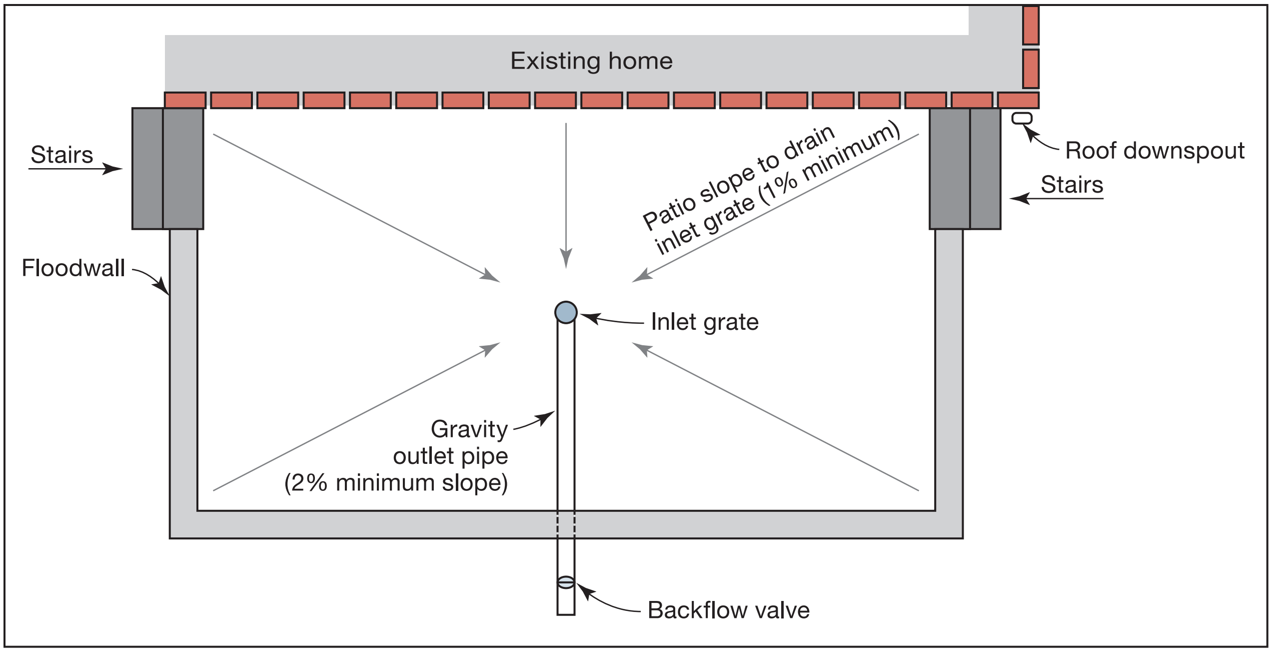


Figure 5F-19. Sample patio drainage to an outlet

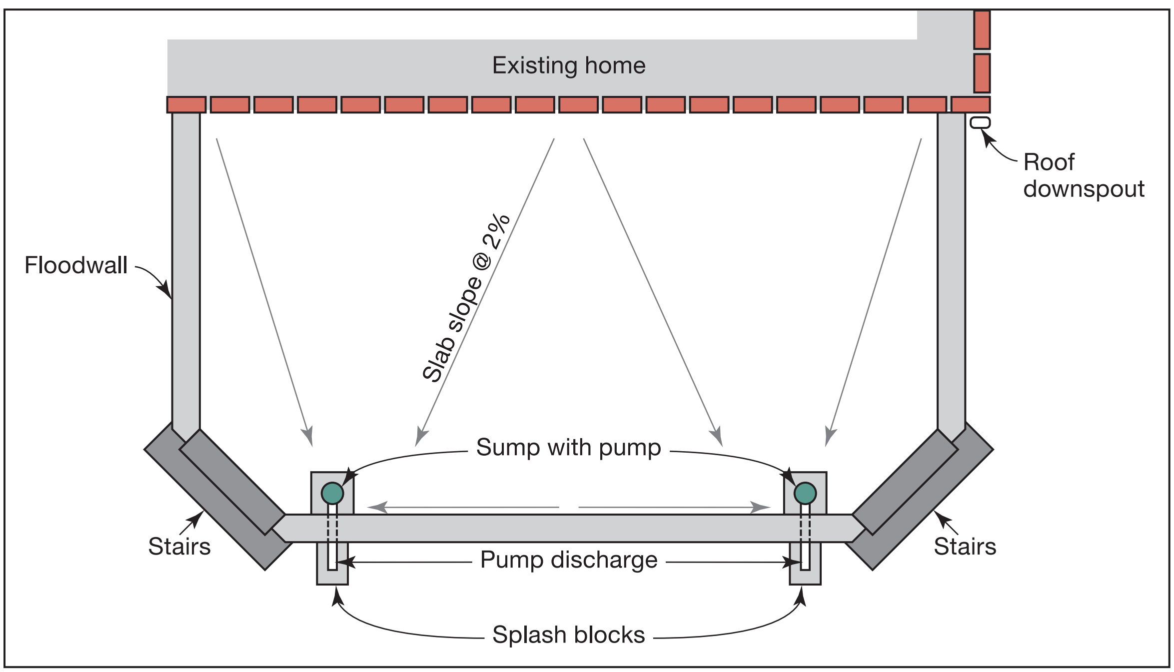


Figure 5F-20. Sample patio drainage to a sump

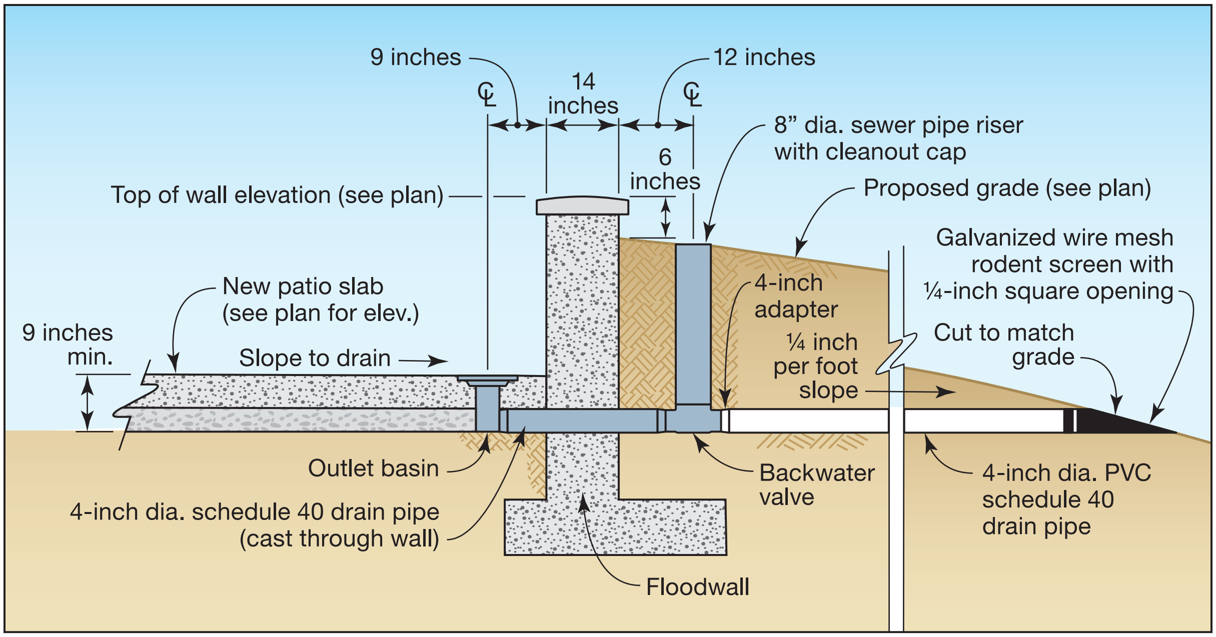


Figure 5F-21. Typical gravity floor drain

Included in the drainage system application is a backflow valve. The unit, sometimes referred to as a check valve, is a type of valve that allows water to flow one way, but automatically closes when water attempts to flow in the opposite direction. Figure 5F-22 shows a typical floodwall with a check valve for gravity drainage. The elevation of the drain outlet should be as high as possible to delay activating the backflow valve, while maintaining a minimum of 2 percent slope on the drain pipe.

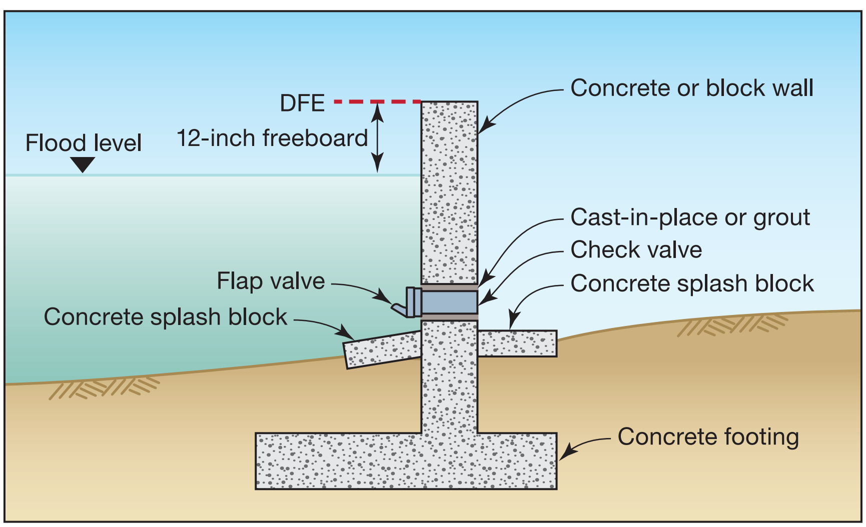


Figure 5F-22. Typical floodwall with check valve

The success of the gravity drainage system is predicated on the fact that the floodwater will reach its maximum height after the rainfall at the site has lessened or stopped. Therefore, when the backflow valve is activated, little or no water will accumulate on the patio slab (usually after the rainstorm). However, should this condition not exist, the use of a sump pump and/or design of runoff storage within the enclosed area should be provided.

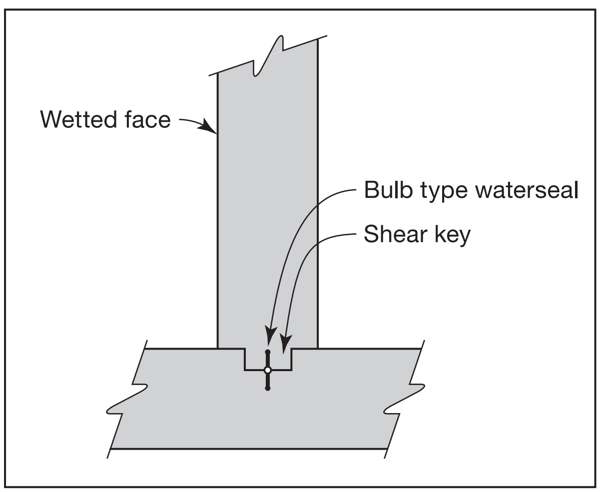
5F.1.5 Floodwall Seepage and Leakage

Floodwalls should be designed and constructed to minimize seepage and leakage during the design flood. Without proper design considerations, floodwalls are susceptible to seepage through the floodwall, seepage under the floodwall, leakage between the floodwall and residence, and leakage through any opening in the floodwall.

5F.1.5.1 Seepage Through the Floodwall

All expansion and construction joints shall be constructed with appropriate waterstops and joint sealing materials. To prevent excess seepage at the tension zones, the maximum deflection of any structural floor slab or exterior wall shall not exceed 1/500 of its shorter span. Figure 5F-23 illustrates the use of waterstops to prevent seepage through a floodwall.

Figure 5F-23. Waterstop



5F.1.5.2 Seepage Under the Floodwall

The structure design may also include the use of impervious barriers or cutoffs under floodwalls to decrease the potential for the development of full hydrostatic pressures and related seepage. These cutoffs must be connected to the impervious membrane of the building walls to effectively operate.

To meet these requirements, it may be necessary to provide impervious cutoffs to prevent seepage beneath the floodwall. This requirement is critical for structures that are designed on highly pervious foundation materials. It may also be necessary to construct a drainage system parallel to the interior base of the floodwall to collect seepage through or under the structure and normal surface runoff from the watershed. All seepage and storm drainage should be diverted to an appropriate number of sumps or gravity drains, or pumped to the floodwater side of the structure. Normal surface runoff (during non-flood conditions) must also be taken into account in the drainage system.

5F.1.5.3 Leakage Between the Floodwall and Residence

The connection between the existing house wall and the floodwall is normally not a fixed connection, because the floodwall footing is not structurally tied to the house foundation footing. Therefore, a gap or expansion joint may exist between the two structures that creates the potential for leakage. This gap should be filled with a waterproof material that will work during seasonal freeze-thaw cycles.

NOTE

The effectiveness of house floodproofing can be increased by placing fill against the house to keep floodwaters from coming into direct contact with the structure.

One alternative, illustrated in Figure 5F-24, is to utilize a 1/2-inch bituminous expansion material, 1/2-inch high-density caulking, and a bulb type water seal.

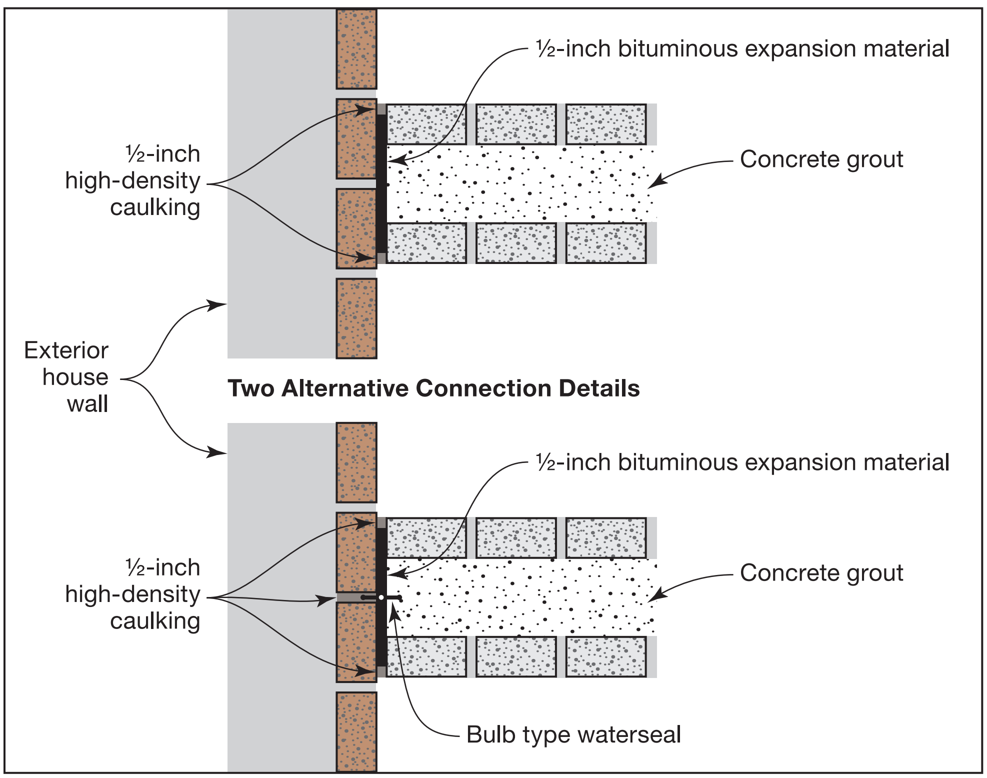


Figure 5F-24 **Floodwall to house connection**

5F.1.6 Floodwall Architectural Details

Floodwalls can be constructed in a variety of designs and materials. By taking into account the individual house design, topography, and construction materials, along with some imagination, the designer can build a floodwall to not only provide a level of flood protection, but also enhance the appearance of the house.

The floodwall design can be a challenge to landscape or to blend into the terrain. By using natural topography and employing various types of floodproofing techniques, such as waterproofing, sealants, or decorative bricks or blocks, the designer can make a floodwall not only blend in with the house and landscape, but also make an area more attractive by creating a privacy fence or by outlining a patio or garden area.

The two most common applications of cosmetic facing of a floodwall consist of brick facing and decorative block facing. (This was shown in Figure 5F-1.)

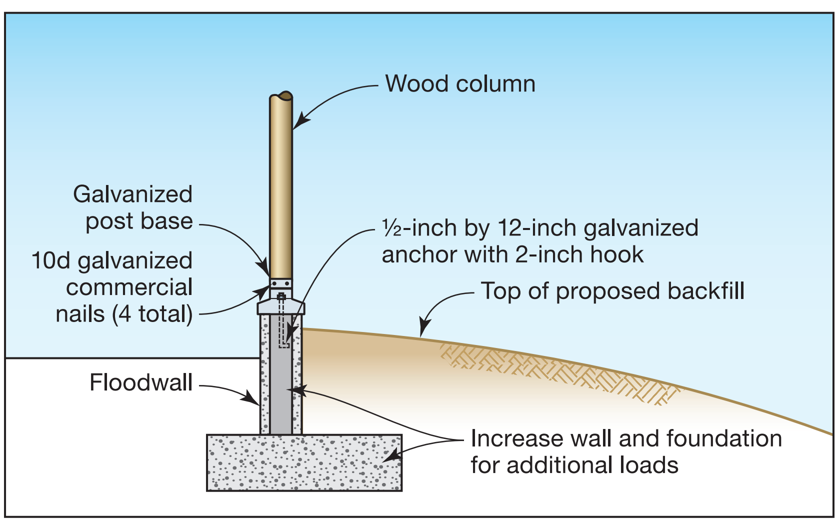
Typical floodwall design often incorporates the use of a patio, which is enclosed by the floodwall. A concrete slab-on-grade or decorative brick paving can be constructed between the house and the floodwall, which will create an attractive and useful feature. The slab-on-grade or brick paving can serve four functional purposes:

- patio area for the homeowner;
- additional bracing for the floodwall;
- positive drainage away from the building towards drainage collection points; and
- impervious barrier inside the floodwall to reduce infiltration of water into the soil adjacent to the structure.

The patio floor or slab-on-grade is set 4 inches below the door openings to provide for a reasonable amount of water storage to accommodate rainfall and roof-gutter spillage that may occur after the floodwaters have reached the elevation that will have closed the backflow valve on the patio drain. The concrete slab is sloped to a floor drain (or drains) which discharge, if existing grade allows, through a gravity pipe or sump pump installation.

In addition to designing patio applications, a qualified design professional can develop architectural and structural modifications that will accommodate existing/future wood decks or roof overhangs (illustrated in Figure 5F-25). These supports can bear on the floodwall's cap, provided additional structural modifications to the floodwall and foundations are furnished to sustain the increased load from above.

Figure 5F-25. Floodwall supporting columns



Residential access requirements, such as driveways, sidewalks, doors, and other entrances, will need to be examined during the design. These entrances may create gaps in the floodwall. Every effort should be made to design passages that extend over the top of the wall and not through it. A stile stairway over a floodwall provides access while not creating an opening in the floodwall.

The stile is a series of steps up and over the floodwall and to the designed grades, which thereby closes the floodwall gap and provides permanent flood protection. A typical step detail is shown in Figure 5F-26. Note that handrails, railings, and stair treads and other safety features must be incorporated into the stile stairway in accordance with local building codes.

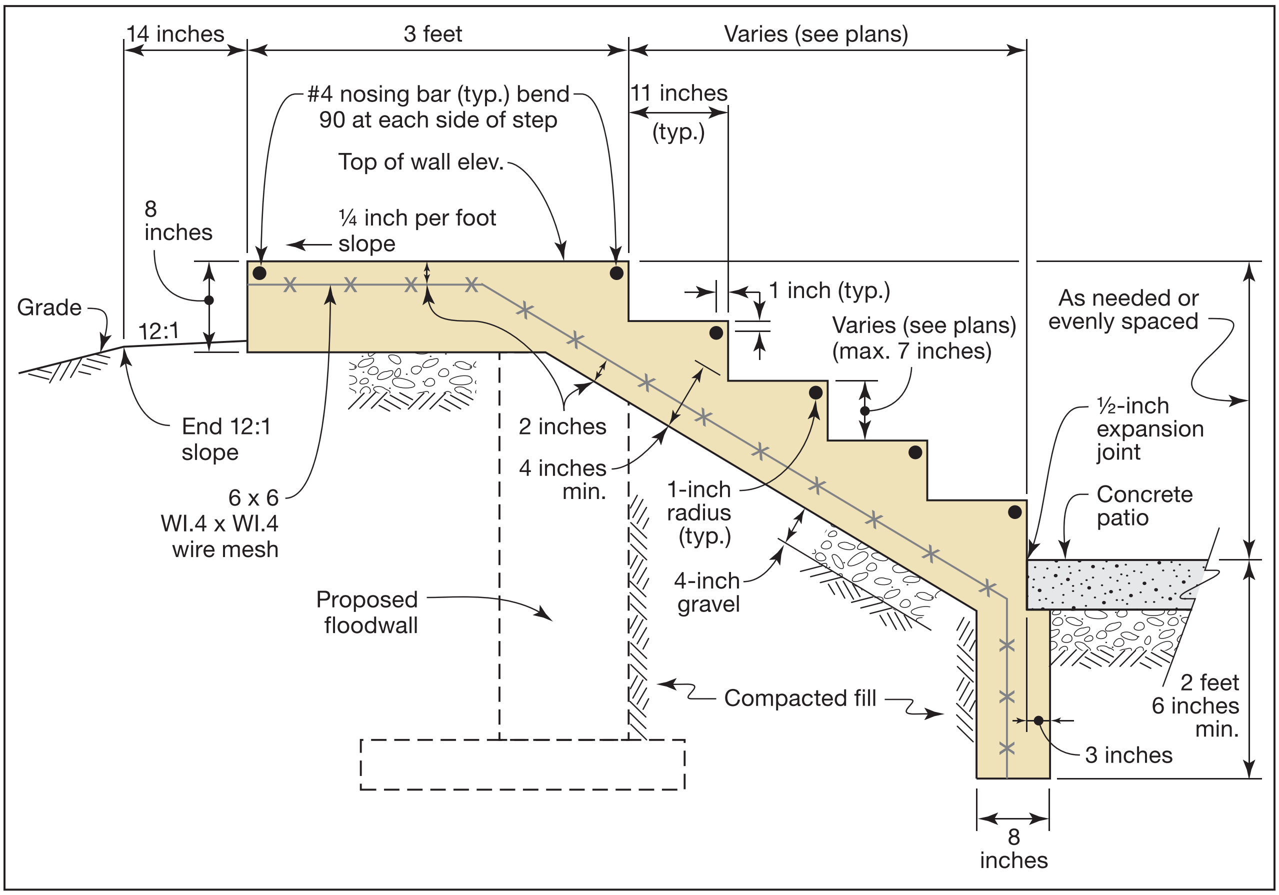


Figure 5F-26. Typical step detail

In addition to the architectural qualities the floodwall can provide, the entire site area can be finished with landscaping features such as planter boxes, trees, and shrubs. Vegetative cover and stone aggregate can also be utilized not only to enhance the flood protection, but also as a method of erosion and scour prevention. A qualified landscape architect should be consulted when selecting material coverage for a particular

NOTE

Landscaping inside and hanging over a protected area may generate organic debris that could clog drains. Plants should be selected that do not result in clogged drains from falling leaves or fruit.

area. Roots, foliage, leaves, and even potential growth patterns of certain trees and shrubs should be accounted for in the selection of landscaping materials.

Once the flood protection has been constructed, a maintenance schedule should be adopted to ensure the system will remain operational during flooding conditions. Floodwalls should be inspected annually for structural integrity. The visual investigation should include a checklist and photographic log of:

- date of inspection;
- general floodwall observations involving wall cracking (length, width, locations), deteriorated mortar joints, misalignments, chipping, etc.;
- sealant observation, including displacement, cracking, and leakage;
- overall general characteristics of the site, including water ponding/leakage, drain(s), and drainage and site landscaping;
- operation of the sump pump, generator/battery, and installation of any closures; and
- testing of drains and backflow valves.

Additionally, the entire flood protection system should be inspected after a flood. A complete observation, including a photographic record similar to the annual report, should be developed and may also include:

- damages associated with impacts and flood;
- excessive erosion and scour damage;
- floodwater marks; and
- functional analysis regarding the flood protection system.

The following Floodwall Inspection Worksheet (Figure 5F-27) can be used to record observations during the annual and post-flood inspections.

Floodwall Inspection Worksheet			
Owner Name:________________ Prepared By:________________			
Address: ________________ Date: ________________			
Property Location: ________________			
Floodwall Component	**Yes**	**No**	**Observations**
Cracking in wall			
Motar joint separation			
Wall misalignment			
Miscellaneous chippng and spalling			
Possible leakage spots			
Sealant displacement			
Water ponding			
Drains functional			
Sump pump operational			
Landscaping			
Sketch area:			
General observations and summary:			

Figure 5F-27. Floodwall Inspection Worksheet

5F.1.7 Floodwall Construction

During the construction of a floodwall, periodic inspections should be conducted to ensure that the flood protection measure has been built per the original design intent. As a minimum, the designer, owner, or owner's representative should inspect and observe the following improvements:

- confirm adequate slope drainage, including drain pipes, patio, and grading outside the floodwall;
- confirm that floodwall foundation was prepared in accordance with plans and specifications;
- confirm that sealants, waterproofing, and caulking were applied per the manufacturer's requirements for installation;

- confirm that the sump pump is operational;
- check sample brick or decorative block (before installation) for patterns or match to existing conditions; and
- confirm that a maintenance requirement checklist was developed and used that included all of the manufacturer's recommendations for passive flood protection applications, sealants, drains, etc.

5F.2 Levees

Unlike floodwalls, levees are not made of manmade materials, but rather compacted soil. Levees are more commonly used than floodwalls for providing protection to individual or limited numbers of residential buildings, but given their relative cost and the amount of land that is required for their construction, they are a less common residential flood-mitigation measure than many of the other retrofitting options presented in this manual. The following sections provide details on important levee design and construction considerations.

5F.2.1 Levee Field Investigation

Certain conditions must exist before levees can be considered a viable retrofitting option. The questions that should be asked before proceeding any further are listed below.

WARNING

A settled height of 6 feet is the maximum elevation recommended for individual residential levees.

- Does the natural topography around the structure in question lend itself to this technique?
- A significant portion of the cost associated with the construction of a levee hinges upon the amount of fill material needed. If the topography around the structure is such that only one or two sides of the structure need to be protected, a levee may be economical.
- Is a suitable impervious fill material readily available?
- Is suitable impervious fill material, such as a CH, CL, or SC, being used? As defined in the ASTM International (ASTM) *Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System)* Designation D2487-10, (ASTM, 2010), such materials are required to eliminate concerns of seepage and stability.
- Do local, State, or Federal laws, regulations, or ordinances restrict or prevent the construction of a levee?
- Has coordination with local, State, and Federal officials been arranged? This may be necessary to determine if the levee retrofitting option is permissible. Certain criteria prohibit construction within a FEMA-designated floodway, the main portion of a stream or watercourse that conveys flow during a storm.
- Will the construction of a levee alter, impede, or redirect the natural flow of floodwaters?

- Have previous calculations from Chapter 4 to determine both the depth and velocity of flood flows around the structure in question been checked? This should be done to ensure that the levee will not result in increased flood hazards upstream. Also, in many cases, the local floodplain administrator may require an analysis of the proposed modification to the floodplain.
- Will flood velocities allow for the use of this technique?
- Do the flood velocities along the water side of the levee embankment exceed 8 feet per second? If so, the cost of protecting against the scour potential may become so great that a different retrofitting technique should be considered.

WARNING

If the assumptions listed in this chapter are not applicable to the site being considered, an experienced levee designer should be consulted or another method considered.

The designer of a levee should be aware that the construction of a levee may not reduce the hydrostatic pressures against a below-grade foundation. Seepage underneath a levee and the natural capillarity of the soil layer may result in a water level inside the levee that is equal to or above grade. This condition is worsened by increased depth of flooding outside the levee and increased flooding duration. Unless this condition is relieved, the effectiveness of the levee may be compromised. This condition, which involves the intersection of the phreatic line with the foundation, is illustrated in Figures 5F-9 and 5F-10.

It is important that the designer check the ability of the existing foundation to withstand the saturated soil pressures that would develop under this condition. The computations necessary for this determination are provided in Chapter 4.

The condition can be relieved by installation of foundation drainage (drainage tile and sump pump) at the footing level, and/or by extending the distance from the foundation to the levee. The land side seepage pressures can also be decreased by placing backfill against the flood side of the levee to extend the point where floodwaters submerge the soil away from the structure, but the effectiveness of this measure depends on the relative characteristics of the soils investigation. The design of foundation drains and sump pumps is presented in Chapter 5D. An experienced geotechnical engineer should compute the spacing required to obviate the problem.

5F.2.2 Levee Design

The following sections describe criteria and key steps that are part of the levee design process.

5F.2.2.1 Standard Levee Design Criteria

The following parameters are established to provide a conservative design while eliminating several steps in the USACE design process, thereby minimizing the design cost. These guidelines pertain to the design and construction of localized levees with a maximum settled height of 6 feet. Techniques of slope stability analysis and calculation of seepage forces are not addressed. The recommended side slopes have been selected based on experience, to satisfy requirements for stability, seepage control, and maintenance. The shear strength of suitable impervious soils compacted to at least 95 percent of the Standard Laboratory density as determined by ASTM Standard D698-07e1 (ASTM, 2007) will be adequate to ensure stability of such low levees, without the need for laboratory or field testing or calculation of safety factors.

The minimum requirements for crest width and levee side slopes are defined below. In combination with the toe drainage trench (see Section 5F.2.3) and the cutoff effect provided by the backfilling of the inspection trench, these minimum requirements will provide sufficient control of seepage, and do not require complex analyses. Flatter land side slopes are recommended for a levee on a sand foundation to provide a lower seepage gradient because a sand foundation is more susceptible to seepage failure than a clay foundation.

NOTE

These levee design recommendations—as illustrated in Figure 5F-28 for a typical residential levee—are conservative. Alternative parameters for a specific site may be developed by an engineer qualified in levee design.

Maximum Settled Levee Height of 6 Feet: This is a practical limit placed due to available space and material costs.

Minimum Levee Crest Width of 5 Feet: This is required to minimize seepage concerns and allow for ease of construction and maintenance.

Levee Floodwater Side Slope of 1 Vertical on 2.5 Horizontal: This is required to minimize the erosion and scour potential, provide adequate stability under all conditions (including rapid drawdown situations), and facilitate maintenance.

Levee Land Side Slope: The land side slope may vary, based on the soil type used in the levee. If the levee material is clay, a land side slope of one vertical to three horizontal is acceptable. If the levee material is sand, a flatter slope of one vertical to five horizontal is recommended to provide a lower seepage gradient.

One Foot of Levee Freeboard: This is required to provide a margin of safety against overtopping and allow for the effects of wave and wind action. These forces create an additional threat by raising the height of the floodwaters.

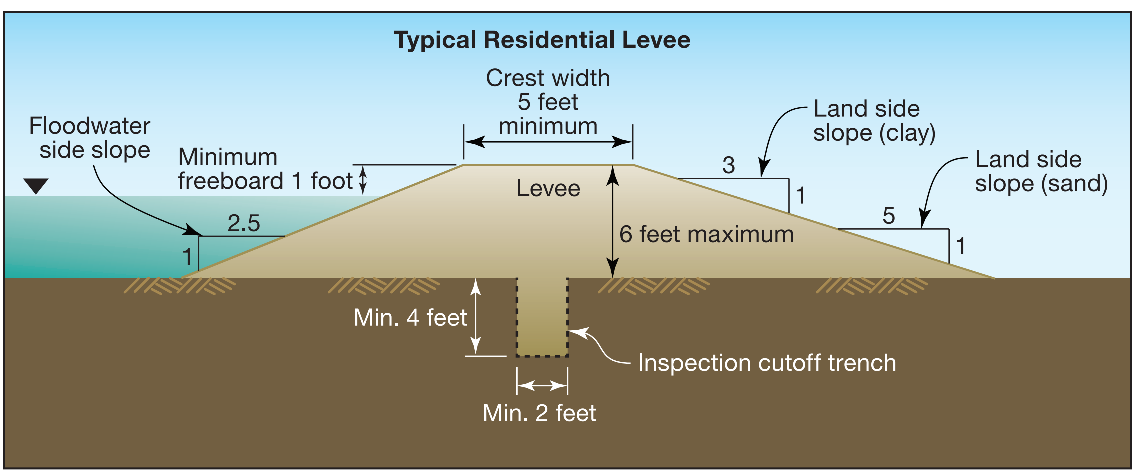


Figure 5F-28. Typical residential levee

5F.2.2.2 Initial Levee Design Phases

Because of the importance of the characteristics of the soil that makes up the levee foundation, the excavation of an inspection trench is required. The minimum dimensions of the inspection trench are shown in Figure 5F-28. The inspection trench, which shall run the length of and be located beneath the center of the levee, provides the designer with information that will dictate the subsequent steps in the design process. The mandatory requirement of an inspection trench is fundamental to the assumptions made for the rest of the design process. The inspection trench will accomplish the following objectives:

Locate Utility Lines That Cross Under the Levee: Once identified, these must be further excavated and backfilled with a compacted impervious material to prevent development of a seepage path beneath the levee along the lines.

Provide "Cut-Off" for Levee Foundation Seepage: The trench itself will be backfilled with a highly impervious soil, such as a CH, CL, or SC, as previously referenced, to create an additional buffer against levee foundation seepage.

Identify Levee Foundation Soil Type: The construction of the inspection trench should provide the designer with a suitable sample to identify the foundation soil type through the use of the ASTM Unified Soil Classification System (USCS). This variable will further direct the design of the levee.

- **Clay Foundation**: If, after inspection, it is determined that the in situ foundation material is composed of a clay soil, as defined by the NRCS, a land side slope of 1 vertical on 3 horizontal should be utilized.
- **Sandy Foundation**: If, after inspection, it is determined that the in situ foundation is composed of a sandy soil, as defined by the NRCS, a land side slope of 1 vertical on 5 horizontal should be utilized.

WARNING

Duration of flooding is a critical consideration in the design of levee seepage control measures. The longer the duration of flooding (i.e., the longer floodwaters are in contact with the levee), the greater the potential for seepage, and the greater the need for seepage control measures such as cutoffs, drainage toes, and impervious cores.

5F.2.3 Levee Seepage Concerns

Two types of seepage must be considered in the design of a residential levee system: levee foundation seepage and embankment seepage. The amount of seepage will be directly related to the type and density of soils in both the foundation and the embankment of the levee. While the installation and backfilling of the inspection trench with impervious material will help reduce concerns of foundation seepage, further steps must be taken to minimize any embankment seepage for levees between 3 and 6 feet in height. The mandatory inclusion of a drainage toe will control the exit of embankment seepage while also controlling seepage in shallow foundation layers.

WARNING

If inspection determines that the foundation consists of a deep deposit of sand or gravel that will permit seepage under the shallow inspection trench, a deeper trench would be required, especially if the protected structure has a basement founded in a NRCS-defined sand or gravel. This scenario may make the use of a levee uneconomical.

The inclusion of a drainage toe for a levee of varying height will be limited to those areas with a height greater than 3 feet. If the levee height varies due to the natural topography, a drainage toe will be required only for those portions of the levee that have a height greater than 3 feet.

The major reason for the inclusion of these measures is to relieve the pressure of seepage through or under the levee so that piping may be avoided. Piping is the creation of a flowpath for water through or under a soil structure such as a levee, dam, or other embankment, resulting in a pipe-like channel carrying water through or under the structure. Piping can lead to levee failure. Piping becomes a more serious problem as the permeability of the foundation soil increases.

The drainage toe should be sized as shown in Figure 5F-29, and should be filled with sand conforming to the gradation of standard concrete sand as defined by ASTM standards.

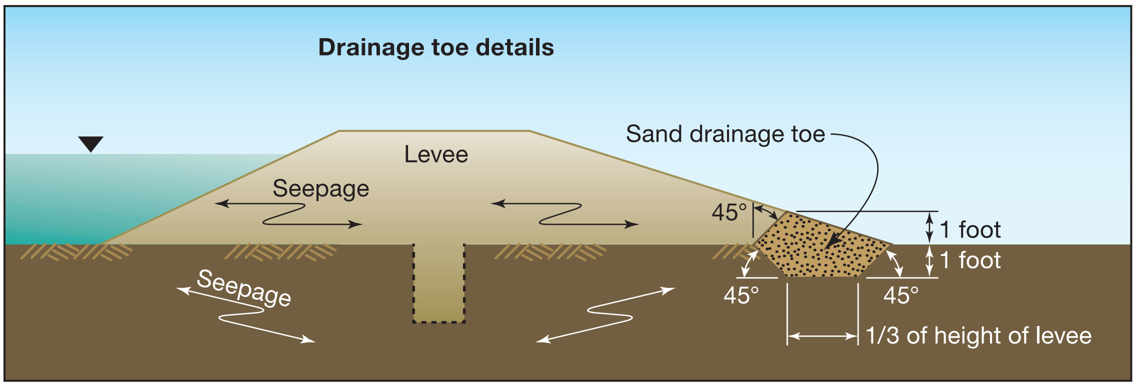


Figure 5F-29. Drainage toe details

NOTE

Long duration flooding may negatively impact the ability of the drainage toe and inspection trench to control the seepage through and under the levee.

5F.2.3.1 Scouring/Levee Slope Protection

The floodwater side of the levee embankment may require protection from erosion caused by excessive flow velocities. For flow velocities of up to 3 feet per second, a vegetatively stabilized or sodded embankment will generally provide adequate erosion protection. Some vegetative covers, such as Bermuda grass, Kentucky bluegrass, and tall fescue, provide erosion protection from velocities of up to 5 feet per second. The grasses should be those that are suitable for the local climate. An alternative or supplement to a vegetative cover is the use of a stone protection layer. The layer should be placed on the entire floodwater face of the levee and be sized in accordance with Table 5F-6.

Table 5F-6. Stone Protection Layer Guidance

Velocities Against Slope (ft/sec)	Minimum Diameter of Stone (in.)
< 2	0.5
< 5	2.0
< 8	9.0

SOURCE: DESIGN AND CONSTRUCTION OF LEVEES (USACE, 2000)

5F.2.3.2 Interior Levee Drainage

NOTE

Guidance on estimating interior drainage quantities is presented in Chapter 4.

Constructing a levee around a house will not only keep floodwaters out, but also will act to keep seepage and rainfall inside the levee unless interior drainage techniques are utilized. One method of draining water that collects from rain and from seepage through and under a levee is to install drain pipes that extend through the levee, as shown in Figure 5F-30. While this will allow for drainage by gravity, the drains must be equipped with flap gates, which close to prevent flow of floodwaters through the pipe. The flap gates will open automatically when interior floodwaters rise above exterior floodwaters.

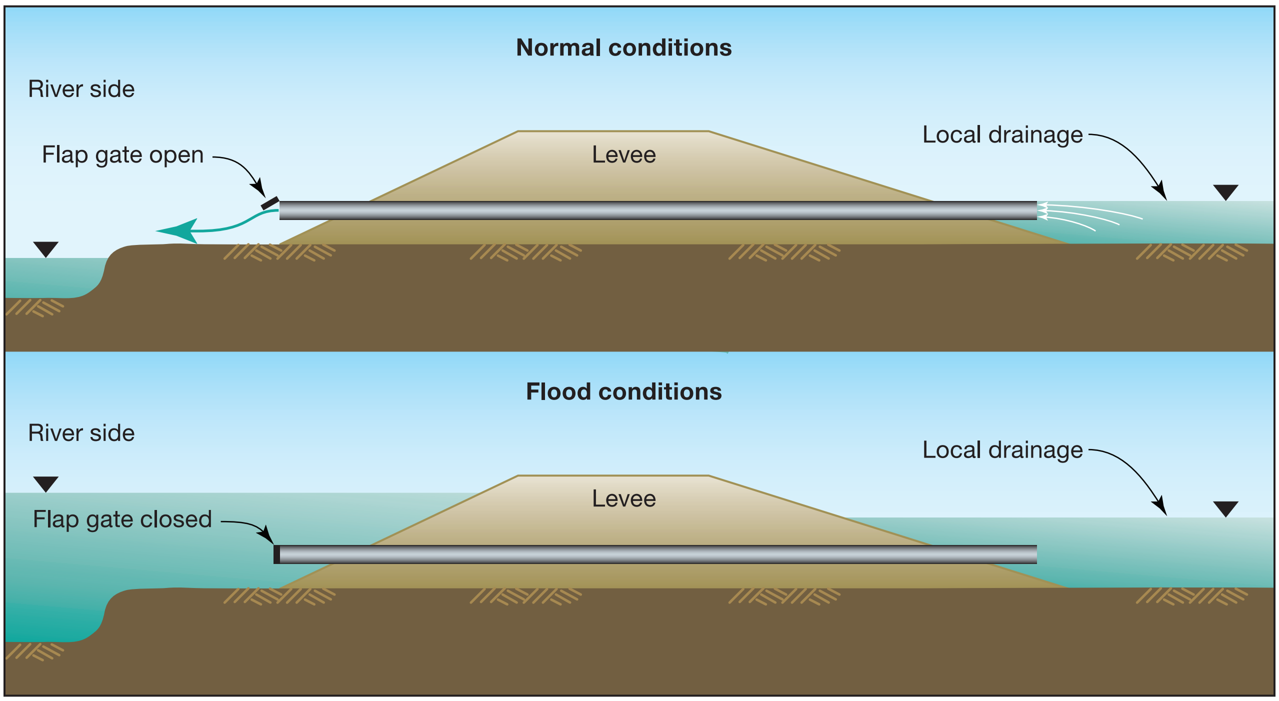
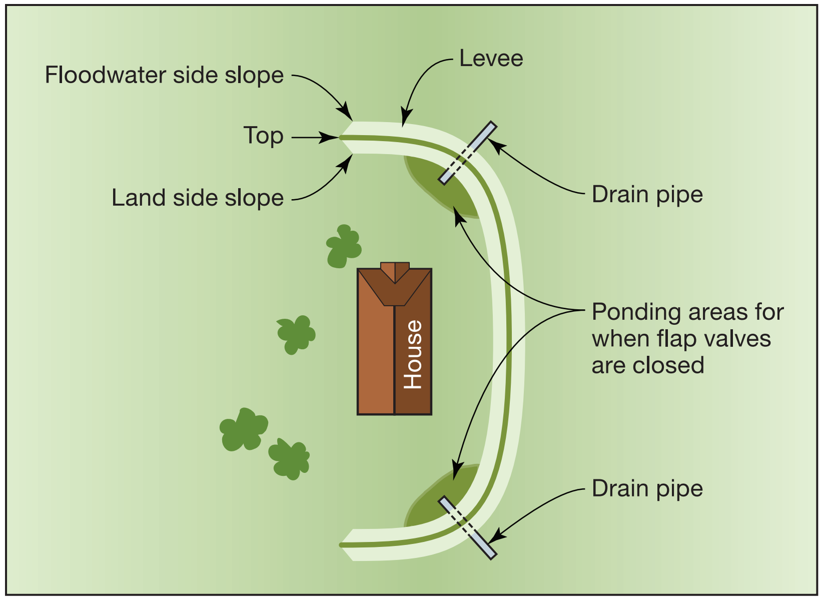


Figure 5F-30. Drain pipe extending through levee

To ensure that water from precipitation or seepage within a leveed area is removed during flooding, a sump pump should be installed in the lowest area encompassed by the levee. All interior drainage measures should lead to this pump, which will discharge the flow up and over the levee. The sump pump should have an independent power source so that it will remain in operation should there be an interruption of electrical power, a common event during a flood.

An alternative to the use of a sump pump (for minor storms), is the creation of an interior storage area that will detain all interior flow until the floodwaters can recede (see Figure 5F-31). Typically, the storage area is sized for the 2- or 10-year recurrence interval event.

Figure 5F-31.
Interior storage area



5F.2.3.3 Levee Maintenance

Levee maintenance should include keeping the vegetation in good condition and preventing the intrusion of any large roots from trees or bushes or animal burrows, since they can create openings or weak paths in the levee through which surface water and seepage can follow, enlarging the openings and causing a piping failure. Planting of trees and bushes is not permitted on the levee.

Any levee design should include a good growth of sod on the top and slopes of the levee to protect against erosion by wind, water, and traffic, and to provide a pleasing appearance. Regular mowing, along with visual inspection several times a year, should identify critical maintenance issues.

5F.2.3.4 Levee Cost

The accuracy of a cost estimate is directly related to the level of detail in a quantity calculation. The following example provides a list of the common expenses associated with the construction of a residential levee. Unit costs vary with location and wholesale price index. To obtain the most accurate unit prices, the designer should consult construction cost publications or local contractors. The designer should also budget an additional 5 percent of the total construction capital outlay annually for maintenance of the levee. Refer to Figure 5F-32 for a Levee Cost Estimating Worksheet.

Levee Cost Estimating Worksheet				
Owner Name:______________ Prepared By:______________				
Address: ______________ Date: ______________				
Property Location: ______________				
Item	**Unit**	**Unit Cost 2009 Dollars**	**# Units Needed**	**Item Cost**
Clearing and Grubbing	Acre	$6,115		
Stripping Topsoil	Cubic Yards	$0.70 to $2.35		
Seeding	Thousand Square Feet	$50 to $67		
Sod	Thousand Square Feet	$620 to $960		
Haul Fill (1–5 miles round trip)	Cubic Yards	$6.10 to $14.75		
Haul Fill (5–15 miles round trip)	Cubic Yards	$9.40 to $28.25		
Import Fill	Cubic Yards	$11.50 to $16.25		
Compact Fill	Cubic Yards	$1.00 to $2.70		
Riprap/Stone Slope Protection	Cubic Yards	$53		
Dig Inspection Trench: 2’ x 4’	Linear Feet	$5.70 to $15.75		
Steel Drain Gate Valve	Each	$825 to $2,590		
Steel Drain Check Valve	Each	$760 to $1,615		
Sump and Sump Pump (with backup battery)	Each	$1,140 to $1,880		
Drain Tile (4”–6” diameter polyvinyl chloride)	Linear Feet	$10.50 to $12.75		
Drain Tile (8”–10” diameter polyvinyl chloride/ reinforced concrete pipe)	Linear Feet	$13.50 to $16.25		
Discharge Piping (1”–2” diameter polyvinyl chloride) for Sump Pump	Linear Feet	$5.00 to $6.10		
			Total Cost	

Figure 5F-32. Levee Cost Estimating Worksheet

5F.2.4 Levee Construction

To prepare for the construction of a levee, all ground vegetation and topsoil should be removed over the full footprint of the levee. If sod and topsoil are present, they should be set aside and saved for surfacing the levee when it is finished.

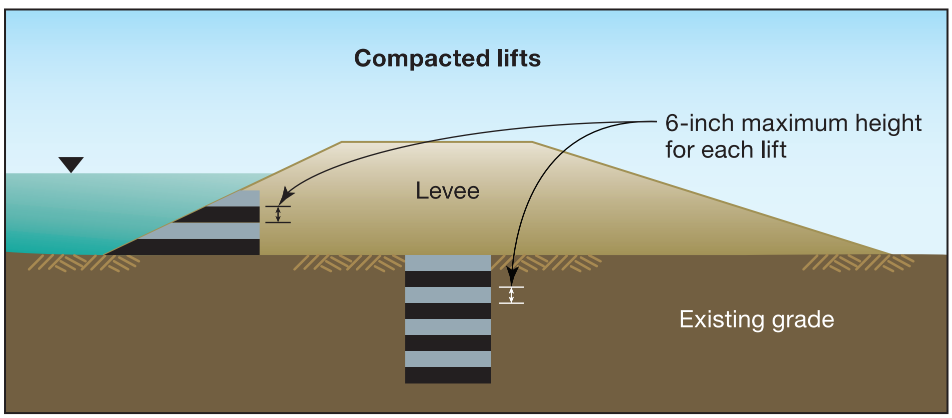
5F.2.4.1 Levee Soil Suitability

Most types of soils are suitable for constructing residential levees. The exceptions are very wet, fine-grained, or highly organic soils, defined as OL, MH, CH, and OH type soils by the NRCS. The best are those with a high clay content, which are highly impervious. Highly expansive clays should also be avoided because of potential cracking due to shrinkage.

5F.2.4.2 Levee Compaction Requirements

As the levee is constructed, it should be built up in layers, or lifts, each of which must be individually compacted. Each lift should be no more than 6 inches deep before compaction (see Figure 5F-33). Compaction to at least 95 percent of standard laboratory density should be performed at or near optimum moisture content with pneumatic-tired rollers, sheepfoot rollers, or other acceptable powered compaction equipment. In some situations, certain types of farm equipment can affect the needed compaction.

Figure 5F-33.
Compacted lifts



5F.2.4.3 Levee Settlement Allowance

The levee should be constructed at least 5 percent higher than the height desired to allow for soil settlement.

WARNING

Settlement allowances vary by geographic region and geologic conditions. Therefore, a 5 percent allowance may not be applicable in all situations. Consult the State or local floodplain management officials for further information.

5F.2.4.4 Levee Borrow Area Restrictions

A principle concern for the construction of the levee is the availability of suitable fill for levee construction, but caution should also be taken as to the location of the fill borrow area.

For the purpose of this manual, a general rule is to avoid utilizing a borrow area within 40 feet of the landward toe of the levee.

5F.2.4.5 Access Across Levee

The complete encirclement of a structure with a levee can create access problems not only for the homeowner but also for emergency vehicles. If the levee is low enough, additional fill material can be added to provide a flat slope in one area for a vehicle access ramp running over the levee as shown in Figure 5F-34. Care should be taken to prohibit high volumes of traffic across the levee, which could result in the formation of ruts or the wearing away of the vegetative cover.

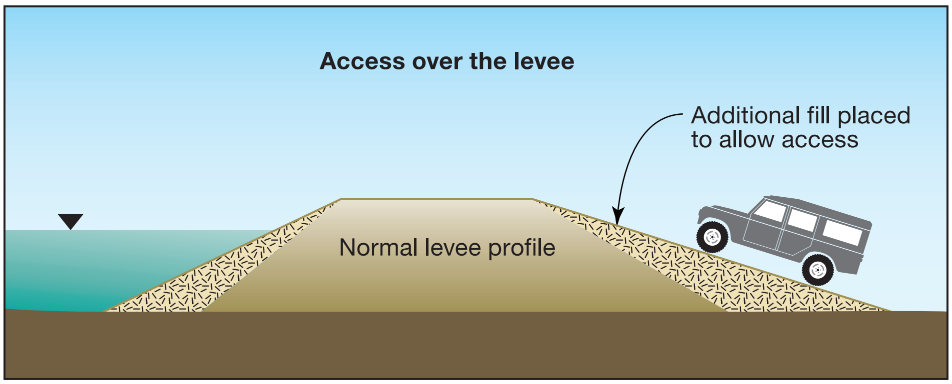


**Figure 5F-34.
Access over the levee**

If it is necessary to have a gap in the levee, this can be closed during flooding through the use of a gate or closure structure. Additional details are provided in Chapter 5D. It should be noted that the use of a closure structure requires human intervention. If the structure in question is susceptible to flood hazards with little or no warning time, or if human intervention cannot be guaranteed, the use of a closure is not recommended.