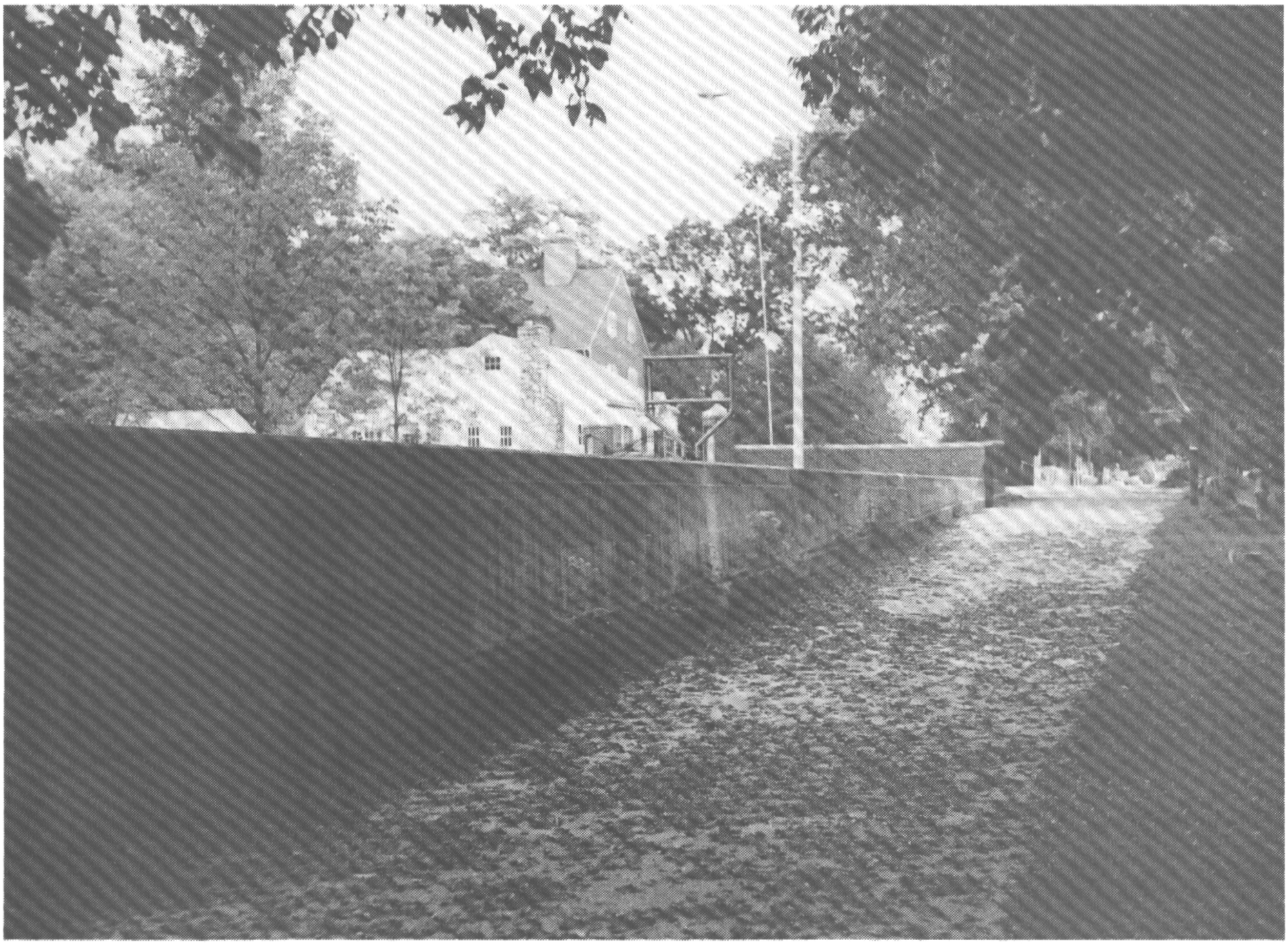
CHAPTER III



FLOODPROOFING DESIGN PERMANENT AND CONTINGENT MEASURES

A. INTRODUCTION

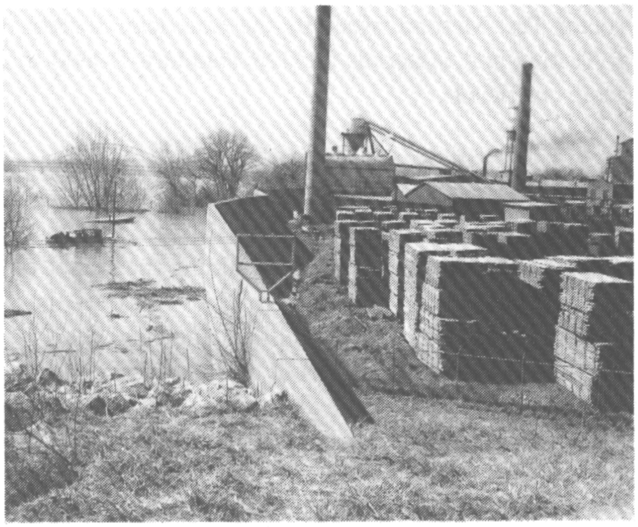
Based on information contained in Chapters I and II, several alternatives for floodproofing a given structure can be identified. This chapter provides guidelines for the technical evaluation of permanent and contingent floodproofing alternatives and for the assessment of required construction materials. Chapter IV provides similar guidelines for emergency floodproofing measures. Information provided in Chapter V can then be used to develop preliminary cost estimates for floodproofing alternatives.

Many of the design aids in Chapters III and IV are based on general and conservative assumptions. These guidelines are sufficient for preliminary studies, but they are not intended to replace necessary detailed site investigations and professionally prepared construction design documents.

B. ELEVATION ON FILL.

Structures may be placed on elevated fill to protect them from flood damages. Fill placed in a floodplain may, however, cause increased flood heights or velocities. In this case, the potential damage to structures in the area is increased. In particular, fill material cannot be placed within a designated 'floodway' (as specified by the National Flood Insurance Program), unless it can be shown that such placement will not cause a significant increase in flood levels. When placement of fill will not increase flood levels, construction on fill can be a viable flood protection method.

1. FILL STABILITY. Structures on fill may be designed and constructed using standard materials and procedures, however, the effect of soil saturation on foundations may still have to be considered. This potential problem would be applicable for fill areas that are highly permeable and subject to extended periods of flooding. If soil saturation is probable, the foundation support and components of the structure should be designed to withstand all hydrostatic pressures, including uplift forces (see Performance Criteria in Appendix D).



Traditional construction practices can generally be used for the structure itself, with the exception of the case noted above. Therefore, the following presentation is limited to the design of the earth fill. A properly constructed fill may often provide a better building foundation than the original material underlying the fill.

The preliminary design of a fill should include laboratory testing to determine the bearing capacity of the foundation soil and the soil to be used as fill. Soil tests can also establish the potential for long and short term settlement. Well-graded sands and gravels that may contain a small percentage of fine clay materials are the most suitable soil materials for fills used to support buildings. However, most inorganic soils are acceptable with the exception of some of the highly plastic, expanding clays. Cohesionless silts and very fine uniform sands are undesirable because they are very difficult to compact.

To safeguard against excessive settlement, fill should be placed when it is at or near the optimum moisture content for compaction. All vegetation and unstable topsoil must be removed from the area to be filled. The fill should be placed in layers not exceeding 12 inches, and each layer should be compacted with appropriate equipment (i.e., pneumatic rollers, sheepsfoot rollers, or vibrating compaction equipment). For most building applications, compaction to 95 percent of the maximum density obtainable with the Standard Proctor Test Method issued by the American Society for Testing and Materials (ASTM Standard D-698) is usually sufficient.

2. FILL DESIGN. After the analyses of the fill material and foundation soils are completed, the design of an earth fill primarily consists of establishing its geometry. In determining the height of fill, some amount of freeboard (margin of safety) may be appropriate between the finished floor and the Design Flood level. The amount of freeboard depends on the incremental damage above the Design Flood level, safety considerations, the incremental cost of fill, and local regulations.

Riprap of the slopes is generally required where the velocity of the stream is greater than 5 feet per second (fps). A one foot thick layer of riprap with a maximum stone size of 150 pounds is considered adequate for most inland flooding situations. The riprap should have a smooth size distribution with a median rock size of about 25 pounds (eight inch diameter), with 80% of the rocks larger than four inches in diameter and ranging down to gravels.

With a distributed size range, the spaces formed by the larger stones are filled with smaller sizes which prevents the formation of open pockets. Angular stones are more suitable for riprap than rounded stones. The rock should be hard, dense, and durable to withstand long exposure to weathering. Rock should be dumped directly from trucks to minimize segregation of rock sizes.

Vegetation may provide acceptable levels of protection for velocities exceeding 5 fps depending on the type, condition and density of vegetation, and the erosive characteristics of the soil. A more detailed discussion of erosion protection and embankment slope stability is provided in Section E, Item 6, below.

3. FILL MAINTENANCE. Little maintenance is required for elevated fills. Fills in high stream velocity areas may require some repair to the riprap embankment protection. The frequency of repair is a function of the frequency of flooding and the adequacy of the original erosion protection. Some fills may include perforated drain pipe as part of a subdrain system. A well-designed subdrain system needs to be cleaned out once every twenty to thirty years.

C. ELEVATION ON POSTS, PILES, PIERS, OR WALLS

1. GENERAL. In situations where a structure cannot be elevated on fill, the functional floors of the structure may be raised above the Design Flood on supporting posts, piles, piers or walls. This solution is particularly appropriate where fill material is not available, where the space below the elevated structure can be used for a secondary purpose such as parking, or where fill cannot be used due to flood characteristics.

Elevated building support systems may be constructed of a variety of materials including wood, steel, masonry, and concrete. Concrete and masonry systems are generally considered most durable under all environmental conditions; but steel and wood will perform satsifactorily if these materials are protected from the elements. Local construction practice and the intended function of the elevated structure will generally indicate the most economical and suitable building material for a particular area.

Whatever materials are used, the elevated structure must be capable of meeting the performance criteria provided in Appendix D. The support system must be designed to minimize the effects of floodwater forces from moving water, debris, impact forces, and accumulation of flood debris without compromising the strength and stability of the total structure. Special attention should be given to the effect of wind loads in combination with floodwater forces, and to the impact loads that may be exerted on exterior structure supports. It may be necessary to 'over design' the exterior upstream supports of a structure to withstand impact forces if a significant amount of debris will be present. It may also be necessary to add a bracing system to the elevated foundation to withstand all anticipated forces. Ideally, braces should be installed above expected flood levels.

2. POSTS. Light frame structures may be elevated on wood, steel, or concrete 'posts'. Posts are generally installed in pre-dug holes. After the post has been lowered into position, the hole may be backfilled with soil, gravel, crushed rock, or some other loose fill material. The backfilling technique, however, does not generally provide adequate bearing capacity, stability, or uplift resistance for non-residential elevated structures. Because the bearing capacity of a post is primarily derived from its end bearing capacity, the capacity may be increased by enlarging the surface that acts on the underlying soil. Bearing capacity may also be increased by using concrete for a portion or all of the backfill operation as shown in Figure III-1. Total encasement will result in maximum stability and resistance to uplift. As shown in the figure, the posts should be anchored to the concrete backfill to increase uplift resistance.

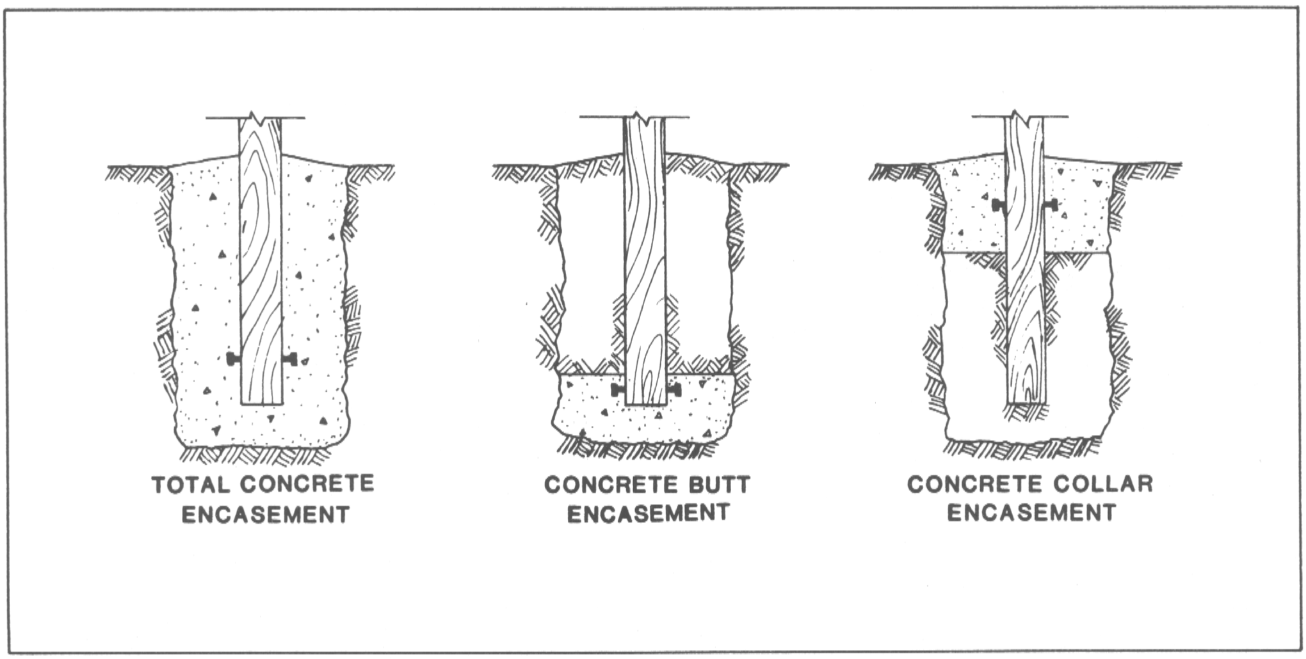


Figure III-1. Concrete Backfill

If poor soil conditions are encountered, the bearing capacity of the post may be improved by the use of a pile or spread footing foundation as shown in Figure III-2. As shown in Figure III-3, the post may be attached above ground level to a reinforced concrete friction pier or a pier that is designed to rest on some other type of footing. If this technique is used it is critical that the post be firmly anchored to the elevated pier to resist overturning and uplift forces.

Posts are generally square or rectangular as these types are easiest to frame into. However, round posts are also used in many cases. As shown by Figure III-4, an elevated structure may be designed to rest on top of the posts (platform construction); or, they may be designed to extend through the structure deck to the roof (pole frame construction) as shown in Figure III-5. Pole frame construction generally increases a structure's resistance to lateral loads.

The number of posts that will be required depends on the diameter and length of the posts, and the amount of load that each column is required to support. Figure III-6 may be used to estimate the approximate size of wood posts after the load per post and the length of the post has been calculated. Although this nomograph considers only square and rectangular members, round members may be used provided their cross sectional area is equal to or greater than that found in the chart. The nomograph also shows the minimum size post that may be used for a given load and/or a given length.

3. PILES. Piles are slender shafts that are driven to a predetermined design depth (friction pile) or to a stable load bearing strata (hardpan, bedrock, etc.). Piles differ from posts in that piles are driven into the ground whereas posts are set in pre-drilled holes. Pile construction generally results in a much greater degree of strength, stability, and resistance to scour than can be achieved with post construction.

Piles can be placed by driving with a steady succession of blows applied by a drop hammer or compressed-air powered hammer. Piles have also been placed by vibration methods, by the aid of water jets in sandy soils (i.e., displacing the soil at the pile point by using a stream of water under high pressure) and by augering in clayey or silty soils. Longer piles are usually required with the latter two methods, because tamping around the pile is required, and load resistance is less than that achieved with driving.

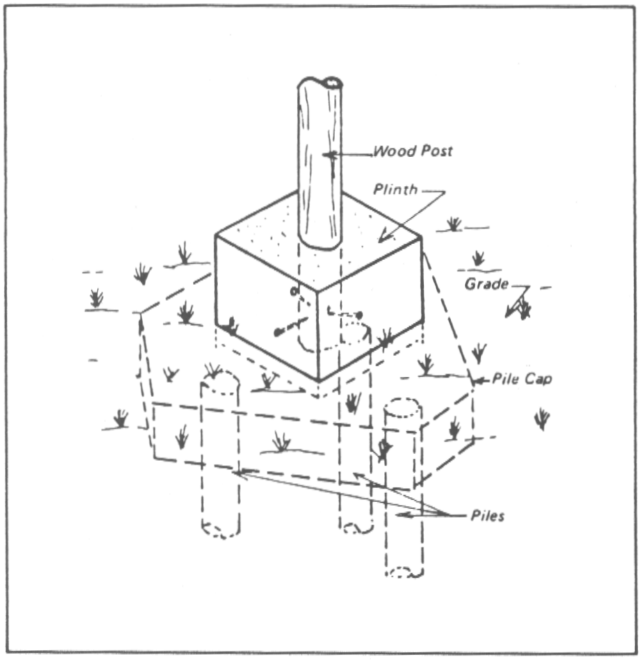


Figure III-2. Pile/Pole Foundation for Low Load Capacity Soils
Source: *Elevated Residential Structures.*

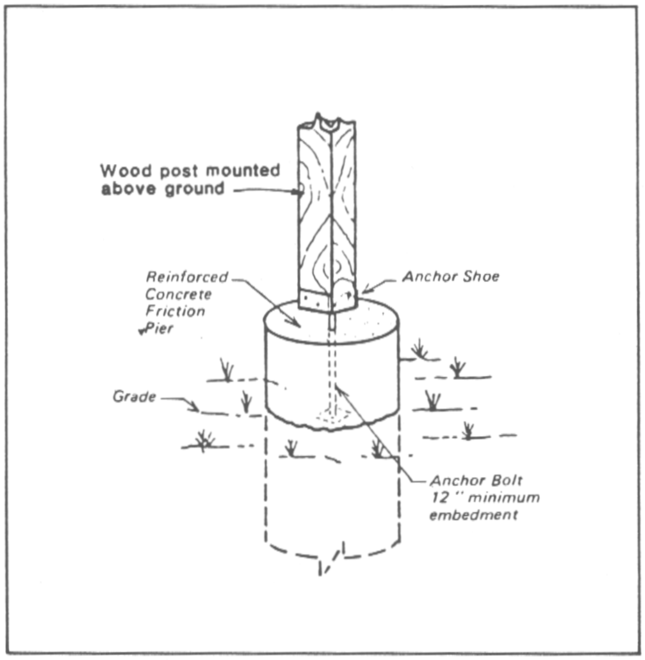


Figure III-3. Reinforced Concrete Friction Pier
Source: *Elevated Residential Structures*

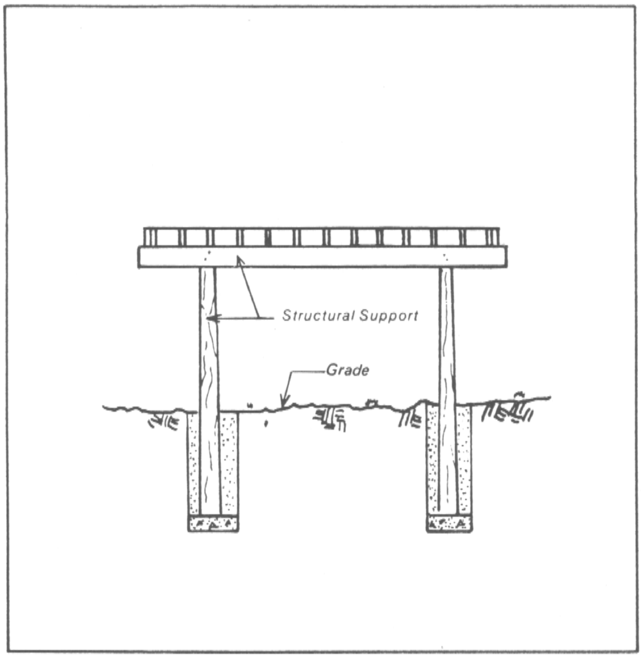


Figure III-4. Platform Construction
Source: *Elevated Residential Structures*

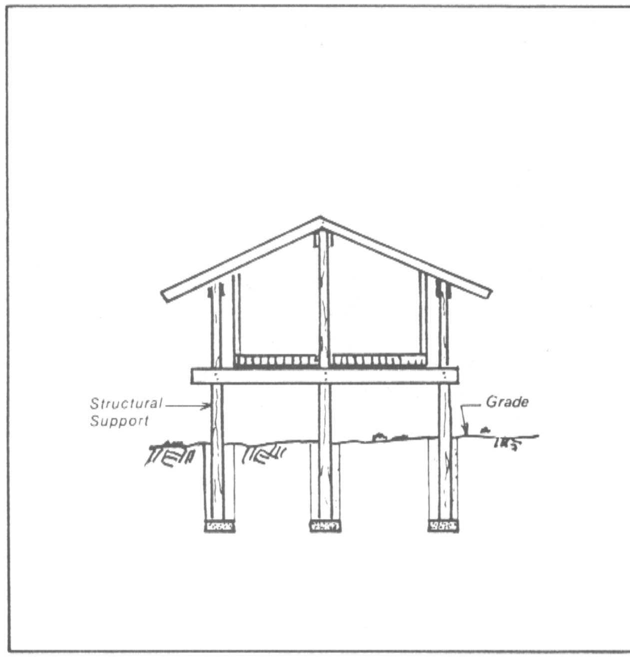
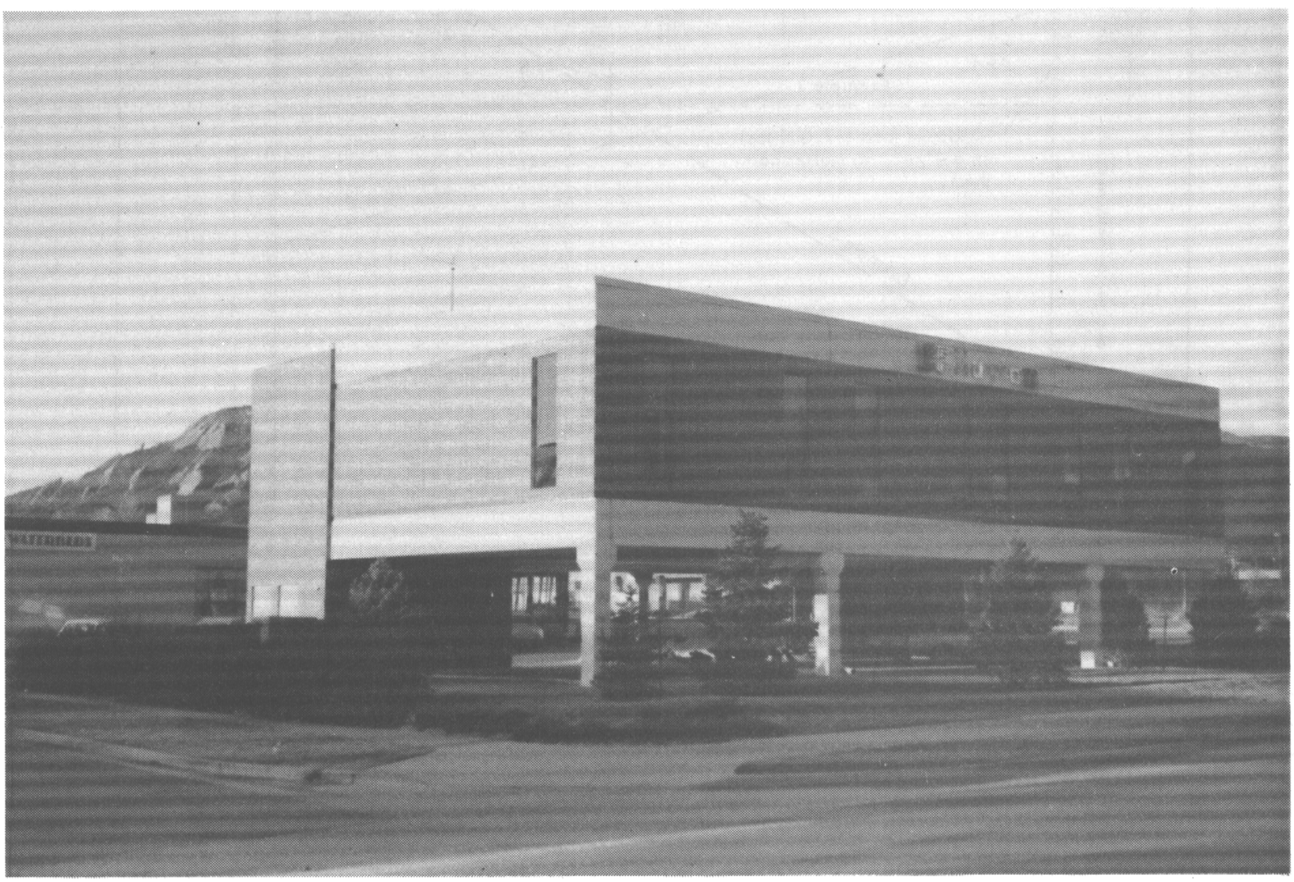
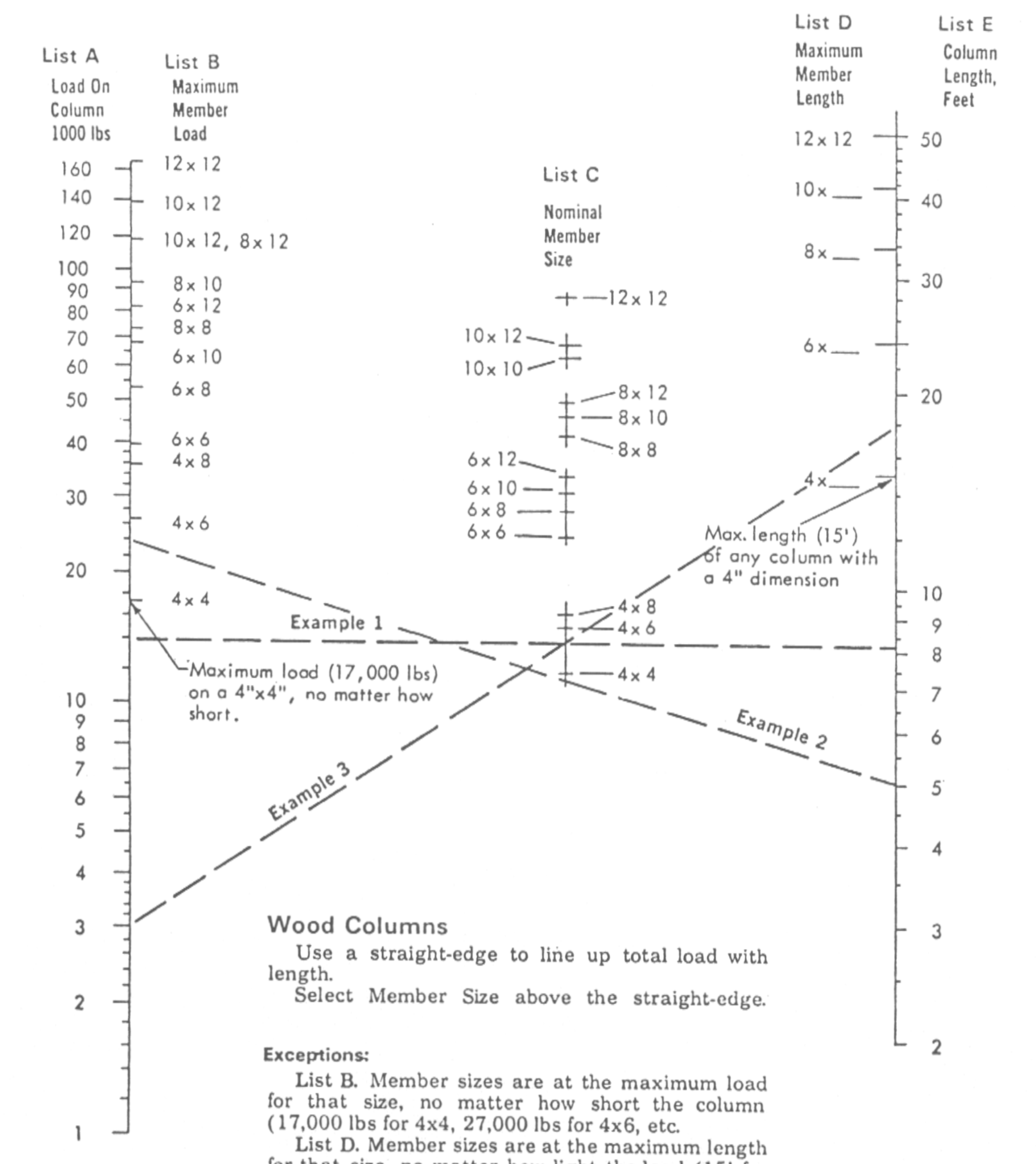


Figure III-5. Pole Framing Construction
Source: *Elevated Residential Structures*



Pre-1970 lumber sizes
Lumber dried below 19%
No. 1 Southern Pine, Douglas Fir or equiv.

Examples:
1. 14,000 lbs, 8' length: 4x6 needed
2. 24,000 lbs, 5' length: 4x6 needed (load exceeds max for 4x4, List B)
3. 3,000 lbs, 20' length: 6x6 needed (length exceeds max for 4" thick columns, List D).



Wood Columns

Use a straight-edge to line up total load with length.
Select Member Size above the straight-edge.

Exceptions:

List B. Member sizes are at the maximum load for that size, no matter how short the column (17,000 lbs for 4x4, 27,000 lbs for 4x6, etc.

List D. Member sizes are at the maximum length for that size, no matter how light the load (15' for 4x4, 4x6, . . .; 25' for 6x6, 6x8, . . .).

Figure III-6. Approximate Loads on Wood Posts

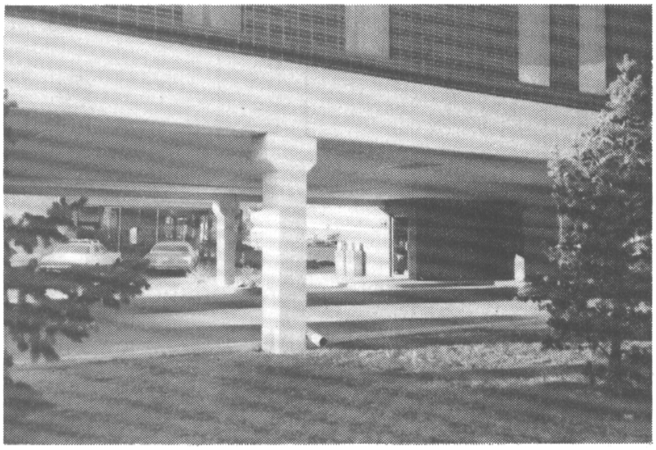
Source: Timber Construction Manual American Institute of Timber Construction

Piles transmit surface loads to the lower levels in the soil mass through a complex soil structure interaction. This load transfer is commonly accomplished by soil-pile friction, pile-tip bearing, or a combination of the two methods. Actual soil conditions will govern the number of piles required to support a given load and the depth of embedment.

The use of timber piles is somewhat restricted by the hardness of the receiving material. Damage to the ends of timber piles may be reduced by using a steel tip or shoe, however it is still possible to break a timber pile under hard driving conditions. For these reasons, timber piles are generally limited to applications where the maximum load will not exceed 30 tons per pile. Southern yellow pine, Douglas fir, and oak are among the principal species used for piling. On the other end of the strength scale, open-end concrete-filled pipe piles are capable of withstanding maximum single pile loads of up to 250 tons.

Piles may also be driven to or below ground level to provide a foundation for posts or piers, or they may extend out of the ground to a level that is at or near the Design Flood and used to support the structure floor (see Figure III-4, Platform Framing). Although piles may be designed to extend to the roof line of a structure (exterior framing construction as shown by Figure III-5) this procedure is generally more difficult because of problems encountered in maintaining precise alignment of the pile as it is driven.

The number of piles that will be required to carry a given load will generally be determined by the ability of the piles to transmit their load to the soil or bearing strata. Pile size and strength is important in resisting lateral loads from wind and floods. Figure III-7 summarizes typical characteristics of timber, steel, and concrete piles.



4. PIERS AND WALLS. Structures may also be elevated on a system of piers and/or wall components. Piers are essentially heavy columns that are constructed out of brick, masonry block, or cast-in-place concrete. Supporting walls may be constructed from these same materials.

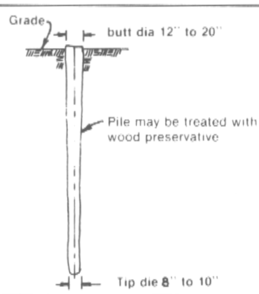
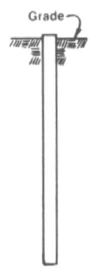
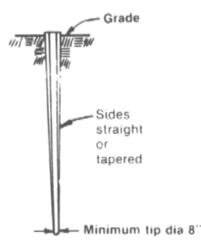
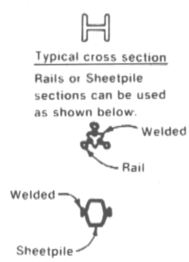
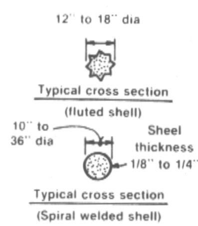
	PILE TYPE		
	TIMBER	STEEL	CLOSED-END PIPE CAST-IN-PLACE CONCRETE
General Working Length	30-60 ft.	40-160 ft.	30-80 ft.
Maximum Design Load Per Pile: Piles on Rock Friction Pile	 25 tons 30 tons	 150 tons 60 tons	 120 tons 60 tons
Application	Best suited for friction pile in granular material	Best suited for end bearing on rock or where extreme depths are required to develop adequate friction	Best suited for medium length friction piles
Advantages	Low initial cost Ease of handling	Easy to splice High Capacity Small Displacement	Can be redriven Shells not easily damaged
Disadvantages	Difficult to splice Vulnerable to damage in hard driving Vulnerable to decay	Vulnerable to Corrosion Easily damaged or deflected by major obstructions	Considerable Displacement Hard to splice after concrete has been placed
Typical Elevation			
Typical Cross Section	 Cross Section		

Figure III-7. Typical Pile Characteristics

Source: Adapted from Foundation Analysis & Design by Joseph E. Bowles

Piers constructed of brick (Figure III-8) or concrete masonry block (Figure III-9) must be anchored to an appropriate footing and voids must be filled with concrete and reinforced as required to withstand anticipated loading conditions. The minimum size of brick or reinforced masonry block pier is recommended to be 12 " X 12 ". Masonry piers should be limited in height to a maximum of ten times their smallest dimension.

Cast-in-place concrete piers (see Figure III-10) can be either reinforced or non-reinforced. High lateral loading conditions will require reinforcing. The recommended minimum size of a cast-in-place concrete pier is 10 " X 10 ", or 12 " in diameter.

In cases where extreme loading conditions exist and floodwater velocities are low to moderate, additional strength may be obtained by using pier (shear) wall sections. These walls should be constructed of cast-in-place concrete or reinforced

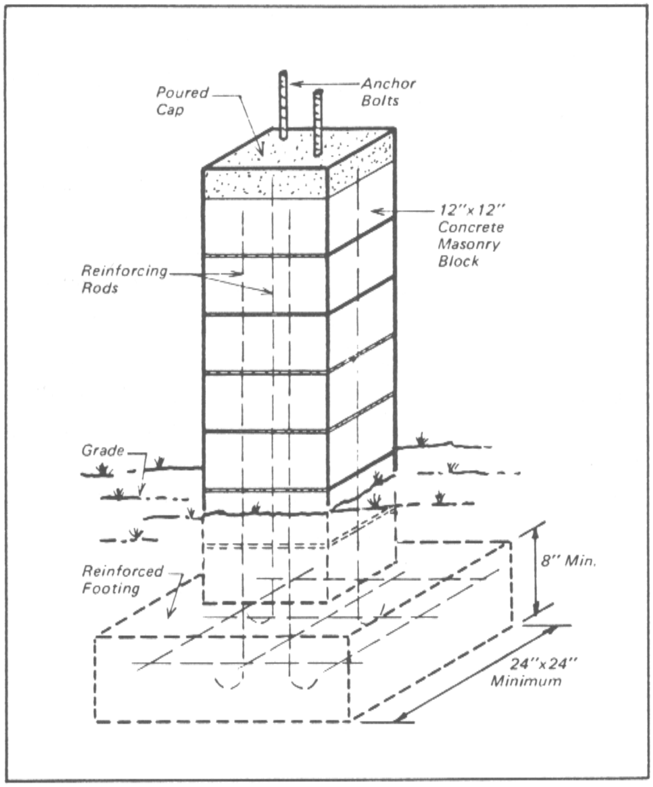


Figure III-9. Reinforced Concrete Masonry Pier
Source: *Elevated Residential Structures*

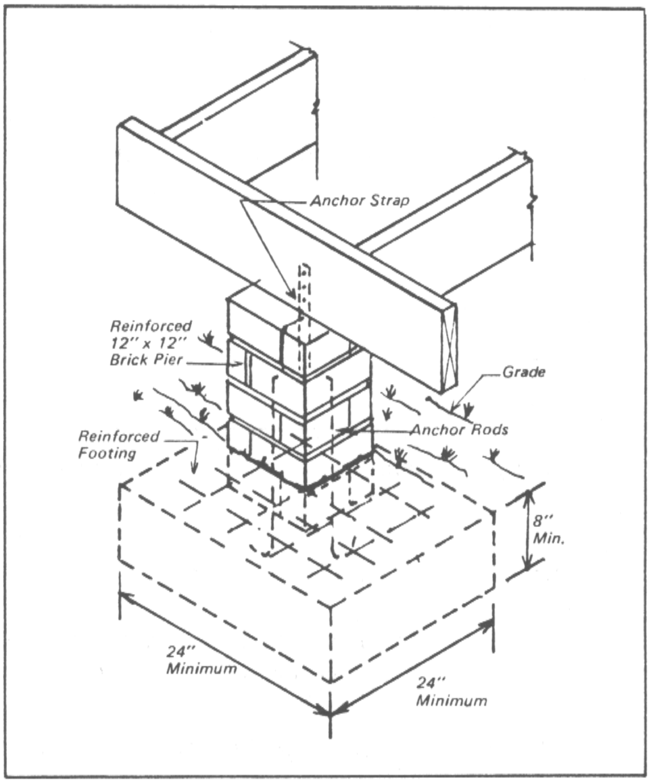


Figure III-8. Brick Pier
Source: *Elevated Residential Structures*

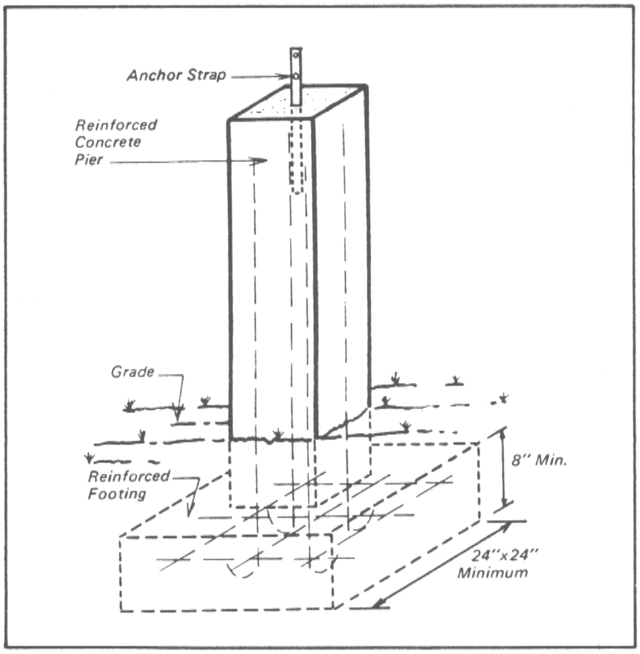


Figure III-10. Reinforced Concrete Pier
Source: *Elevated Residential Structures*

masonry. Wall sections should be placed parallel to the direction of flood flow as shown in Figure III-11, and should be spaced to provide the least obstruction to flow and the least potential for trapping floating debris. Shear wall sections may also be attached to posts or piles in the above manner to increase the lateral stability of the post or pile system.

Piers may be supported on isolated spread footings (Figure III-11) or a deep pile foundation (Figure III-2). The bottom of the footing should be placed below the local extreme frost penetration level and at a depth that is capable of resisting anticipated lateral, uplift, and scour forces. Table III-1 summarizes some of the major requirements for reinforced pier construction.

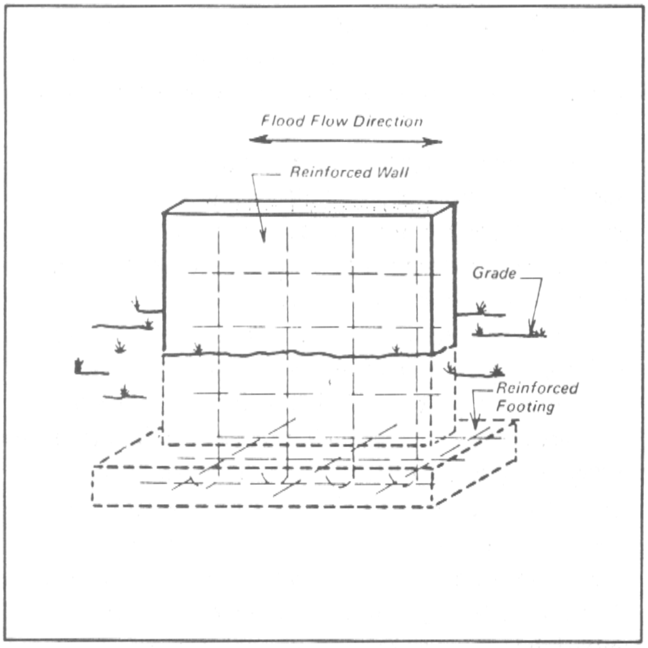


Figure III-11. Reinforced Pier or Solid Wall
Source: *Elevated Residential Structures*

TABLE III-1

MINIMUM REQUIREMENTS FOR REINFORCED PIERS

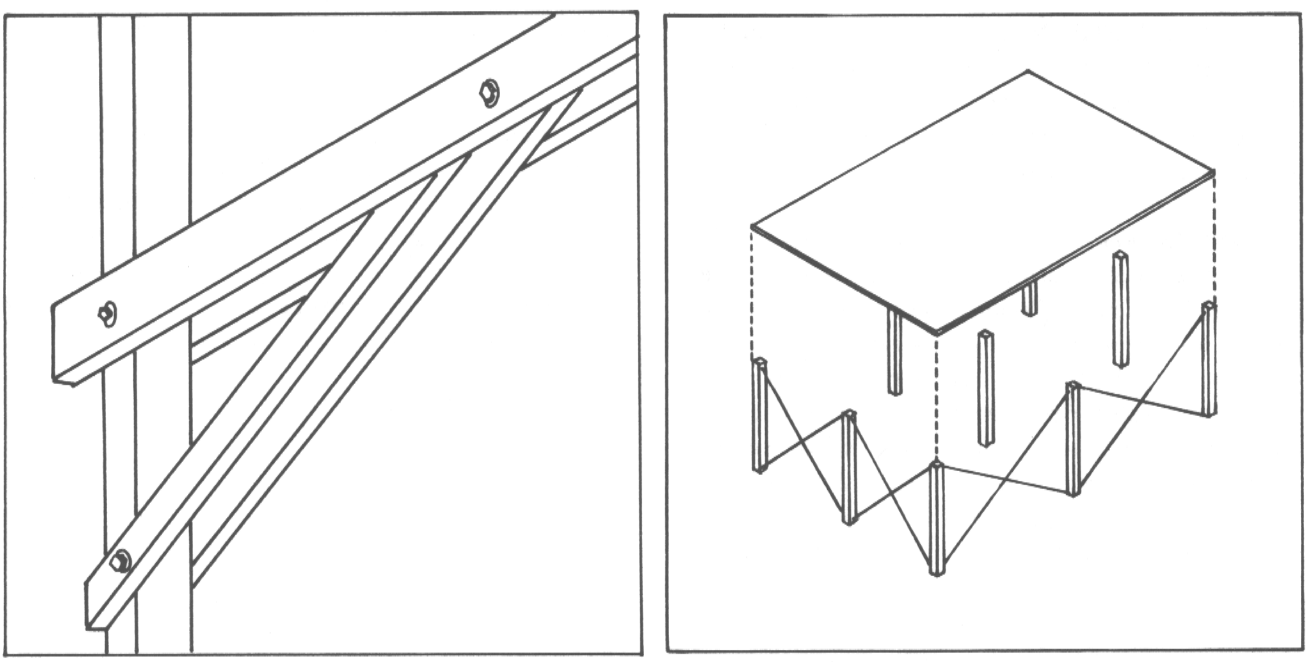
Pier Material	Min. Pier Size	Min. Footing Size	Pier Spacing: Right Angles to Joists	Pier Spacing: Parallel to Joists	Useful Elevation Range
Brick	12" x 12"	24" x 24" x 8"	8' o.c.	12' o.c.	18" to 6'
Concrete Masonry	12" x 12" or 8" x 16"	24" x 24" x 8" 20" x 24" x 8"	8' o.c.	12' o.c.	18" to 8'
Poured-in-Place Concrete	Min. 12" dia. or 10" x 10"	20" x 20" x 8"			18" to 12' +

Source: *Elevated Residential Structures*

5. BRACING. Additional lateral support for elevated structures may be provided through the use of knee and diagonal bracing and shear wall bracing.

Knee and diagonal braces (Figure III-12) are bolted to the base of one post or pile and just below or to the floor beam at the adjacent post or pile. Lumber (recommended to be greater than 2 inches thick) or steel rods can be used to brace wood posts or piles. The rods can be fitted through holes filled with wood preservative and fastened with nuts and cast beveled washers. Welded connections or drill holes can be used to provide rod bracing in steel post or pile foundations. Such rods are usually 5/8 to 3/4 inches in diameter. Maintenance requirements for steel bracing are greater due to corrosion. Although diagonal bracing is more likely to be struck by debris than knee bracing, this disadvantage is usually outweighed by the greater stability provided by diagonal bracing.

6. MAINTENANCE. Structures elevated on posts, piles, piers or walls will require more maintenance attention than those elevated on fill. Repair requirements are a function of the frequency of flooding and the adequacy of the original design and construction. If concrete piers are used, maintenance may never be necessary. If steel columns of piers are used, painting will be required at least every three to five years. Timber piers will also require treatment at these intervals. Timber needs to be protected from insect infestations and organic deterioration. Scoured areas around the piers need to be repaired after each flood. The degree of scour repair will be a function of floodwater velocities.



Knee Brace

Diagonal Bracing

Figure III-12. Knee and Diagonal Bracing

D. WATERPROOF CONSTRUCTION (CLOSURES, FLOOD SHIELDS, SEALANTS, AND MEMBRANES).

1. INTRODUCTION. The term 'watertight construction' (or, 'waterproof construction'), as used in this manual, denotes the floodproofing of a structure to prevent floodwaters from reaching its interior. This approach can result in extreme loading on the exterior surfaces (walls and floor) of a structure. Because of the variety and magnitude of forces that are applied to a watertight structure, all structural components must be carefully analyzed.

Appendix D provides appropriate design criteria. The following sections present structure strength and stability characteristics, waterproofing techniques, closure and flood shield design, and building support systems that must all be evaluated in the design process. The information presented herein may be used to develop initial design concepts. However, the complexity of designing a safe and effective waterproofing system is extremely great. Because of this complexity, final system design must be prepared by an appropriate design professional.

2. WALL STRENGTH. In terms of strength characteristics, there are three basic wall types that may be considered for watertight construction: brick veneer, unreinforced masonry and concrete, and reinforced masonry and concrete.

a) Brick Veneer. Tests have shown that standard brick veneer walls can be used to protect against very low flooding depths. Because the common brick veneer wall leaks excessively, this type of wall must be waterproofed. Best results can be obtained by installing a water barrier between two layers of brick. Without modifications, a standard brick veneer wall should not be expected to withstand more than 2 feet of hydrostatic pressure. If a safety factor is desired, the protection height should be limited to 1.5 feet of water.

b) Unreinforced Masonry and Concrete. Unreinforced concrete and concrete block masonry walls are generally 8 - 12 inches in thickness and contain no vertical or horizontal reinforcement to enhance loading capabilities. These materials are normally used for structures that are under 24 feet in height. Dead loads for 1-2 story 8" block structures typically range from 500-1500 pounds per linear foot. Dead loads for similar concrete wall structures typically range from 800-2000 pounds per linear foot.

As the vertical load on a wall increases, the water height it can withstand increases. For example, an unreinforced wall 8 feet high and 8 inches thick, subjected to a dead load of 1,000 pounds per linear foot, may withstand water heights up to 3.2 feet, whereas the same wall with a load of 3,000 pounds per linear foot may withstand water heights up to 4 feet. As the height of the wall increases, resistance to failure is lowered.

The *maximum* protection depth for any unreinforced walls, regardless of their thickness, height, or vertical loading characteristics, should be no more than 6 feet. *However, the reader should be cautioned that the strength characteristics as discussed in the paragraph above are based only on lateral forces imposed by non-velocity water loads.* This maximum must also be reduced to allow for forces imposed by soil, impact loads, floodwater velocity, etc. For example, floodwater velocity effects on the recommended maximum protection are shown in Figure III-13.

Additional reductions in the protection heights shown in Figure III-13 would be required by soil, impact, and other loads and discussed in Section D, Item 2. *It is necessary that an evaluation of the wall strength capabilities be made by qualified personnel before any watertight protection measures are applied to unreinforced masonry or concrete.*

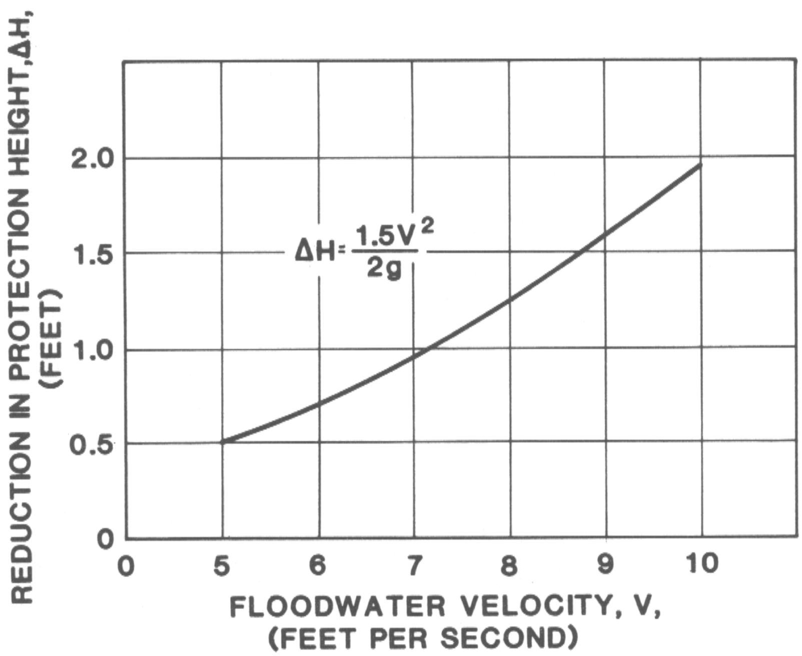


Figure III-13. Reduction in Protection Height as a Function of Floodwater Velocity.

c) Reinforced Masonry and Concrete. The design of reinforcement for masonry and concrete walls for commercial and industrial structures cannot be addressed in detail in this manual. The wide range of loading conditions and configurations require that a structural analysis be performed for each design. *Typical re-bar configurations for simple block and concrete walls are given in this section for illustrative purposes only.*

Reinforced masonry walls are generally constructed of 8 or 10 inch thick blocks (Figure III-14). The block units are set in mortar with vertical reinforcing bars grouted into the block cavities. In some cases, horizontal mild steel wire reinforcing is also grouted between every second or third block course, and a block bond beam is often placed on the top course with reinforcing bars. Reinforced cast-in-place concrete walls are also generally 8 - 10 inches thick and are reinforced with vertical mild steel reinforcing bars for bending loads and horizontal temperature and shrinkage steel (Figure III-15). Reinforced wall systems for new structures can be designed to withstand large hydrostatic and hydrodynamic flood loads. For existing walls, it will be necessary to assume that no reinforcement exists, unless original design plans showing the reinforcement can be found.

d) Determination of Strength. The strength of a wall is determined through a series of calculations that require the expertise of a registered engineer. Maximum flood protection depth and flood velocity are factors which need to be determined in addition to consideration of the two common modes of wall failure. The first consideration would be a translation of the bottom of a wall, most probably at the floor line (Shear Failure), driven by an outside horizontal force such as a hydrostatic or a soil force.

The second would be a failure of the block wall somewhere near the mid-height of the wall (Flexural Failure). In determining whether either one of these modes of failure are possible for a given non-reinforced wall, an engineer would calculate the total weight of all of the vertical loads applied to the top of the wall, such as the contributing portion of the weight of the building (i.e., dead loads).

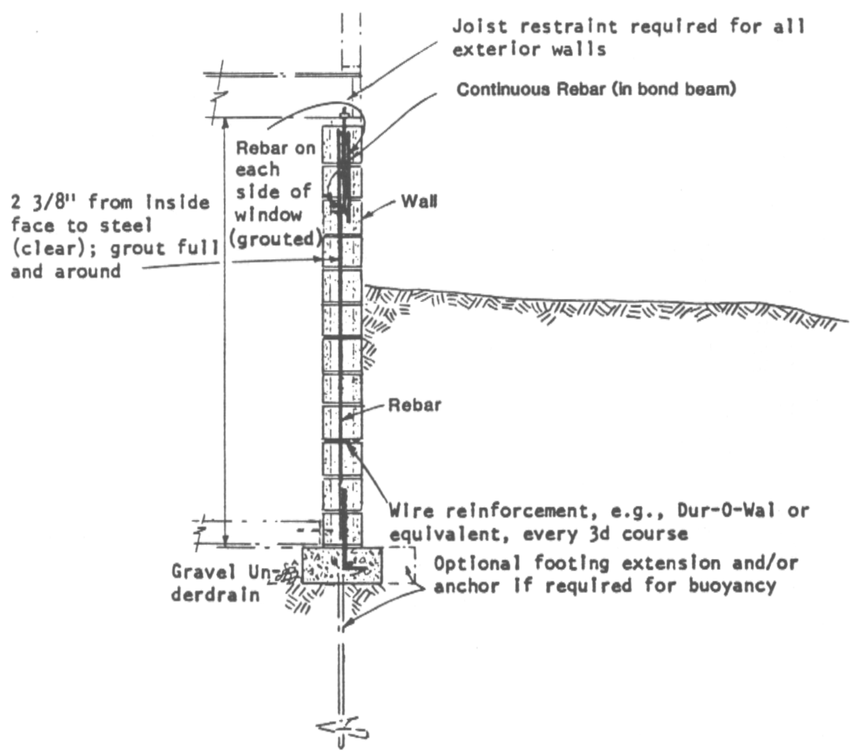


Figure III-14. Typical Reinforced Masonry Block

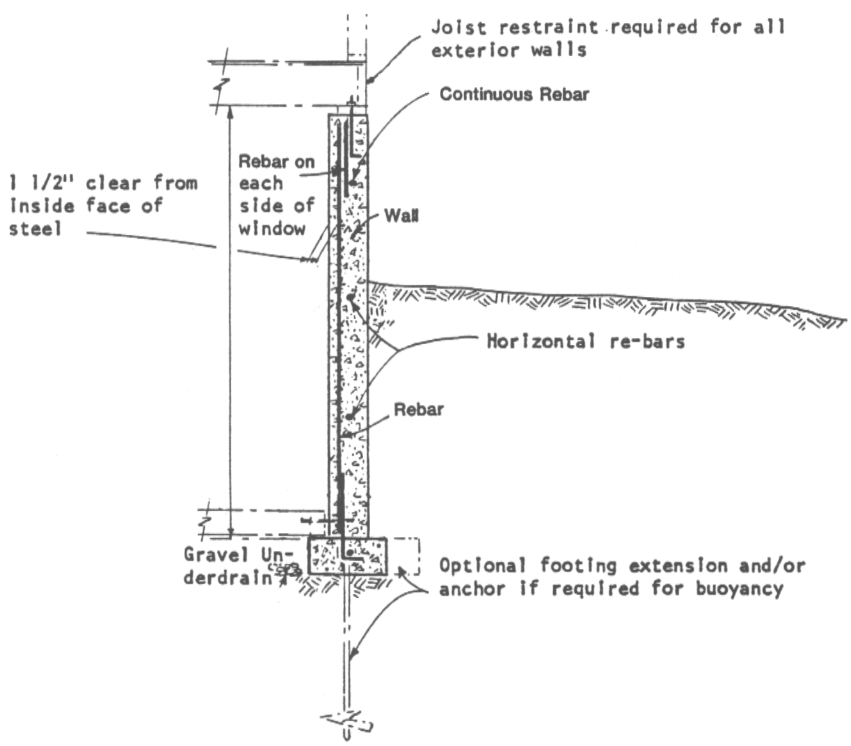


Figure III-15. Typical Reinforced Concrete

One would then sum all of the horizontal loads applied to the wall, such as hydrostatic pressure. The ratio of horizontal to vertical loads is an important parameter in determining the capacity of a wall. The more vertical load the wall is carrying, the more horizontal load it can resist. Knowing the external applied loads, the physical properties of the masonry wall need to be checked. These properties (or variables) include: height, thickness, and tensile and compressive strengths of the mortar and of the block. The relationship of the loads and physical properties are described in other engineering manuals.

The unreinforced wall is usually good for small horizontal hydrostatic pressures such as three feet or less. The usual mode of failure is a tensile failure where the mortar fails in tension. The compressive capacity of mortar is at least 10 times greater than the tensile capacity.Therefore, to offset this deficiency, reinforcing steel bars are grouted into the cells of the masonry block. Once again, the formulas necessary for proportioning the correct amount of steel and where to place it can be found in numerous engineering text books and publications by the Masonry Institute including :

Building Code Requirements for Concrete Masonry Structures (ACI 531-79) & Commentary (ACRI 531R-79), American Concrete Institute, 1978.

Masonry Structural Design for Buildings, TM 5-809-3, AFM 88-3, Chap. 3, Departments of the Army and the Air Force, December 1973.

Partially Reinforced Concrete Masonry Walls, National Concrete Masonry Association, 1975.

Reinforced Concrete Masonry Design Tables, National Concrete Masonry Association, 1971.

Reinforced Masonry Design by Robert R. Schneider & Walter L. Dickey, Prentice-Hall Inc., 1980.

The reader should note that the more vertical load a non-reinforced masonry wall is carrying, the more horizontal load it can resist, and reinforcing a masonry wall with steel bars is always a desirable alternative for a plain masonry block wall.

3. FLOOR STRENGTH AND STRUCTURAL STABILITY. Cast-in-place concrete is the only construction material that has the design capability to resist full hydrostatic uplift pressures. Slab floors can resist uplift pressures in two ways. First, an unreinforced slab can be designed to be thick enough to have sufficient strength and dead load to resist the uplift pressures. Unreinforced concrete slabs can withstand a hydrostatic head approximately 2.25 times their thickness above the bottom of the slab. Reliance upon the thickness and weight of the floor slab may be applicable for upgrading the strength and stability of an existing floor system, or for relatively small new structures where the total weight of the proposed structure is not adequate to resist maximum uplift forces. However, this solution is generally not cost-effective.

The second, and preferred technique involves the use of a reinforced concrete slab that is tied into the structure walls, columns, and footings so that the total weight of the structure is used to counteract uplift pressures. This type of construction (see Figure III-16) is generally referred to as a mat or raft foundation. The raft foundation acts as a combined footing that covers the entire area beneath the structure and supports all walls and columns. If the raft is reinforced to resist all applied loads this type of construction provides additional stability and resistance against overturning and flotation forces as a result of the total structural dead and live loading forces on the slab. This technique is generally a very cost-effective way to provide adequate stability for relatively large heavy non-residential structures. Raft construction can also be supported on pile or pier foundations where additional bearing capacity is required.

If detailed analyses show that a structure cannot be stabilized by the slab design techniques described above, it may be possible to reduce uplift pressures or to anchor the structure. These techniques are described in the following sections.

4. CONTERACTING OF HYDROSTATIC FORCES. In many cases, hydrostatic uplift forces represent a critical loading force that must be reduced if a structure is to be waterproofed successfully. Excessive uplift pressures may be reduced to tolerable levels through the use of impervious blankets and cutoffs, and subsurface drainage systems, and anchorage.

a) Impervious Cutoffs. Various types of impervious cutoffs may be used to decrease the amount of seepage that can flow under a floodproofed structure and to reduce hydrostatic pressures. Cutoffs may be constructed of steel sheet piling, cement grout curtains, impervious compacted soil, or similar materials. The cutoff may be placed directly beneath the foundation footing or it may be placed some distance away from the footing. For new structures, it may be possible to extend the foundation system to connect to an impervious stratum as shown in Figure III-17. This approach is cost-effective only where an appropriate impervious stratum is encountered at a shallow depth. In all cases, when floodwaters are expected to rise above ground level, the cutoff must be designed as an integral part of the

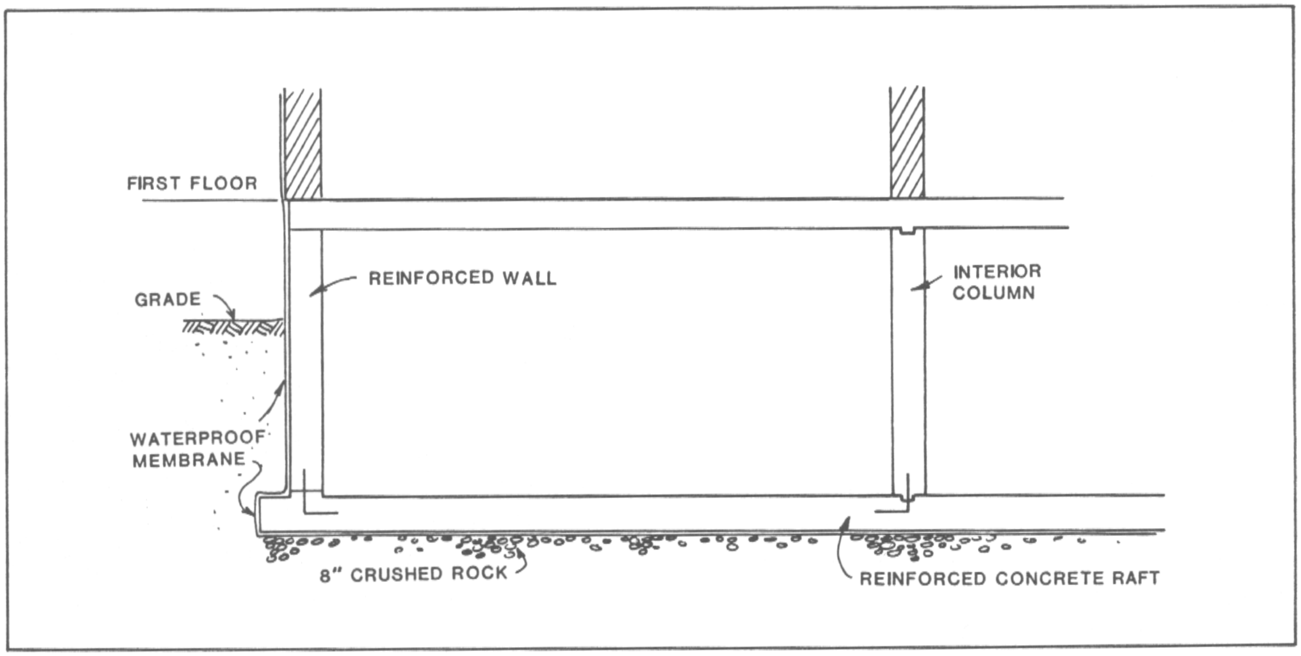


Figure III-16. Raft or Mat Foundation

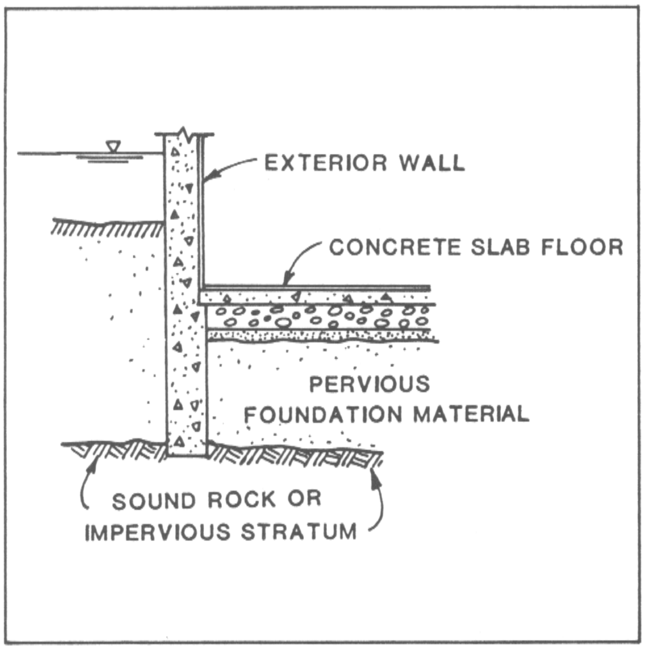


Figure III-17. Wall Extension to Reduce Seepage and Hydrostatic Pressures

structure, or it must be tied into the structure with impervious blankets or membranes as shown in Figure III-18. In addition, the cutoff must extend to an impervious stratum to be effective. Cutoffs, impervious blankets, and membranes must be carefully installed as even a minor defect in the system can result in application of full hydrostatic pressure loading on the foundation wall and floor system.

b) Subsurface Drainage. Subsurface drainage systems may be used alone or in combination with cutoff systems to reduce hydrostatic pressures. Drainage systems are generally not effective in reducing lateral pressures on walls during severe flooding conditions, and even the best foundation drain system is likely to be ineffective when an infinite source of water exists. However, drainage systems can be used to significantly reduce uplift pressures on the floor slab. The degree that pressure can be reduced depends on the permeability of adjacent soils and the adequacy of the subdrainage system design.

The most effective subdrain system requires a blanket drain extending under the total structure foundation as shown in Figure III-19. The blanket drain material must provide adequate bearing capacity while maintaining a high degree of permeability. A system of perforated drain pipes may be used to direct seepage to a sump pump for discharge above the flood level. Provisions for cleaning the drains should be incorporated in the design. The size of the pump (or pumps) required for this purpose will depend on many factors including the permeability of the soil, the length of the seepage path and the depth of flood water exerting pressure on the system. If the pumping system is critical to the stability of the structure, standby equipment must be provided in case the pump fails or the power supply is disrupted.

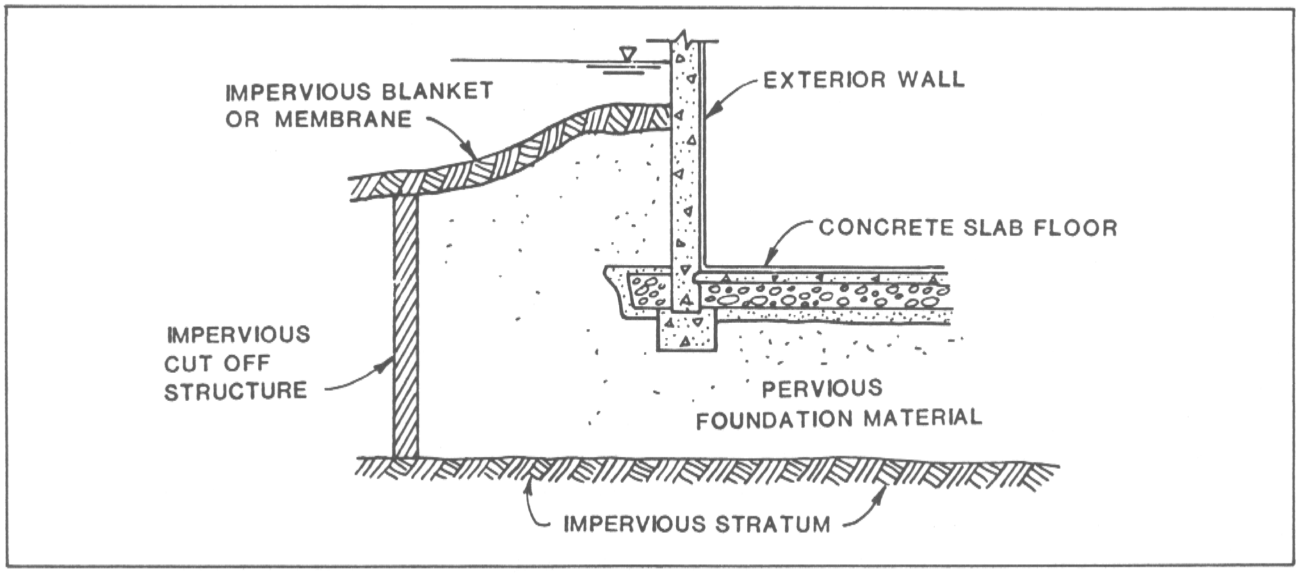


Figure III-18. Impervious Blanket and Cutoff to Reduce Seepage and Hydrostatic Pressures

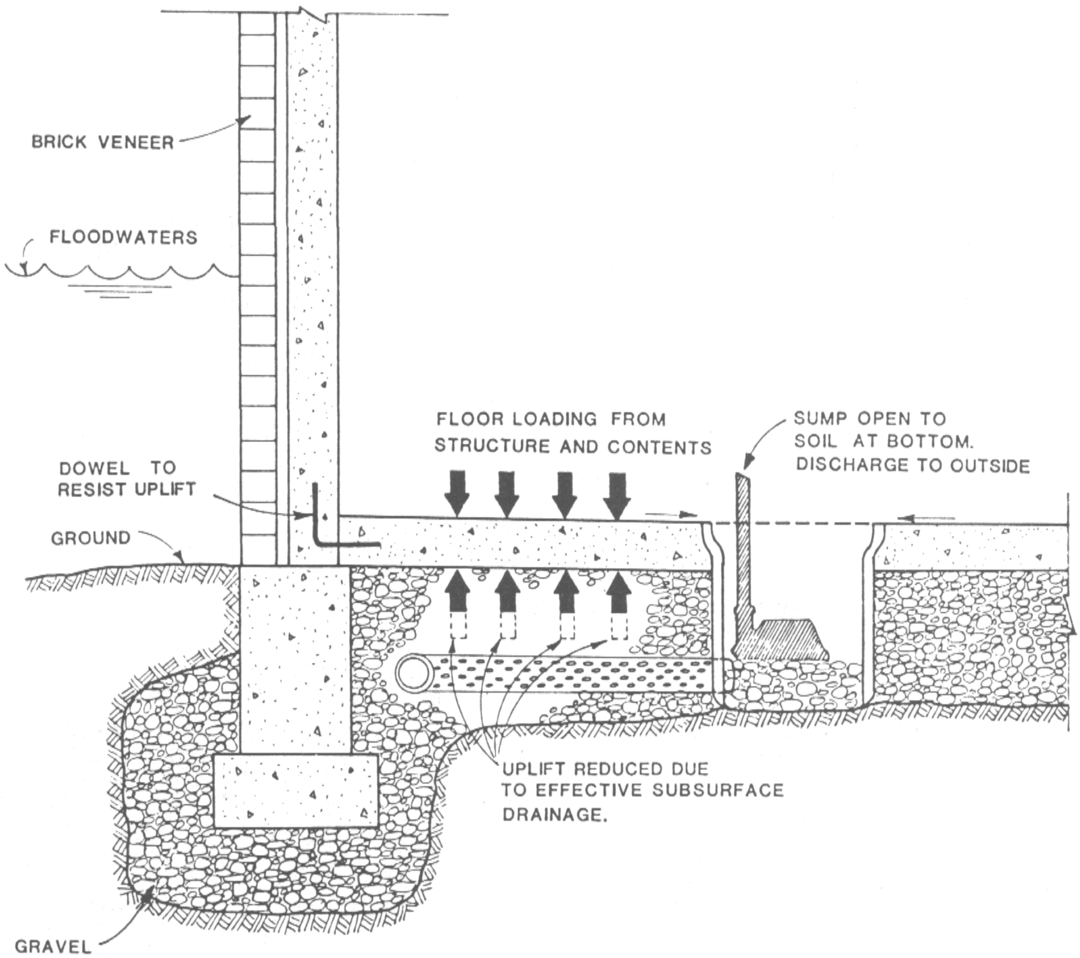


Figure III-19. Drained Subfloor Detail

c) Pressure Relief Systems. As an added degree of protection against structural failure of a new building, or for an existing structure that cannot be modified to reduce uplift pressures, it is generally desirable to install some tye of pressure relief system.

If sump pumps are used, the bottom of the sump area may be left open to the foundation soils or relief pipes may be used to direct water from beneath the floor slab to an enclosed sump area. These provisions are required to provide an exit point to relieve pressures that might develop if the drainage system fails. Another method is to install pressure relief valves in the floor slab as shown in Figure III-20. These valves are designed to allow water to flow into the structure at some pressure that is below the structure failure point. Experience has shown that a 4″ diameter valve should be installed for every 750 square feet of floor slab space. More valves should be located near the exterior walls than toward the center of the slab.

d) Anchorage. Another technique that can be used to stabilize a structure against flood forces is the integrated anchorage of all structural elements. For example, concrete foundation walls, piers or posts may be anchored to footings with hooked 1/2″ rods extending from the footing to the cap. Anchor bolts 4′ to 6′ apart may be used to anchor sills or plates to the foundation walls. (See details in Figures III-14 and III-15.)

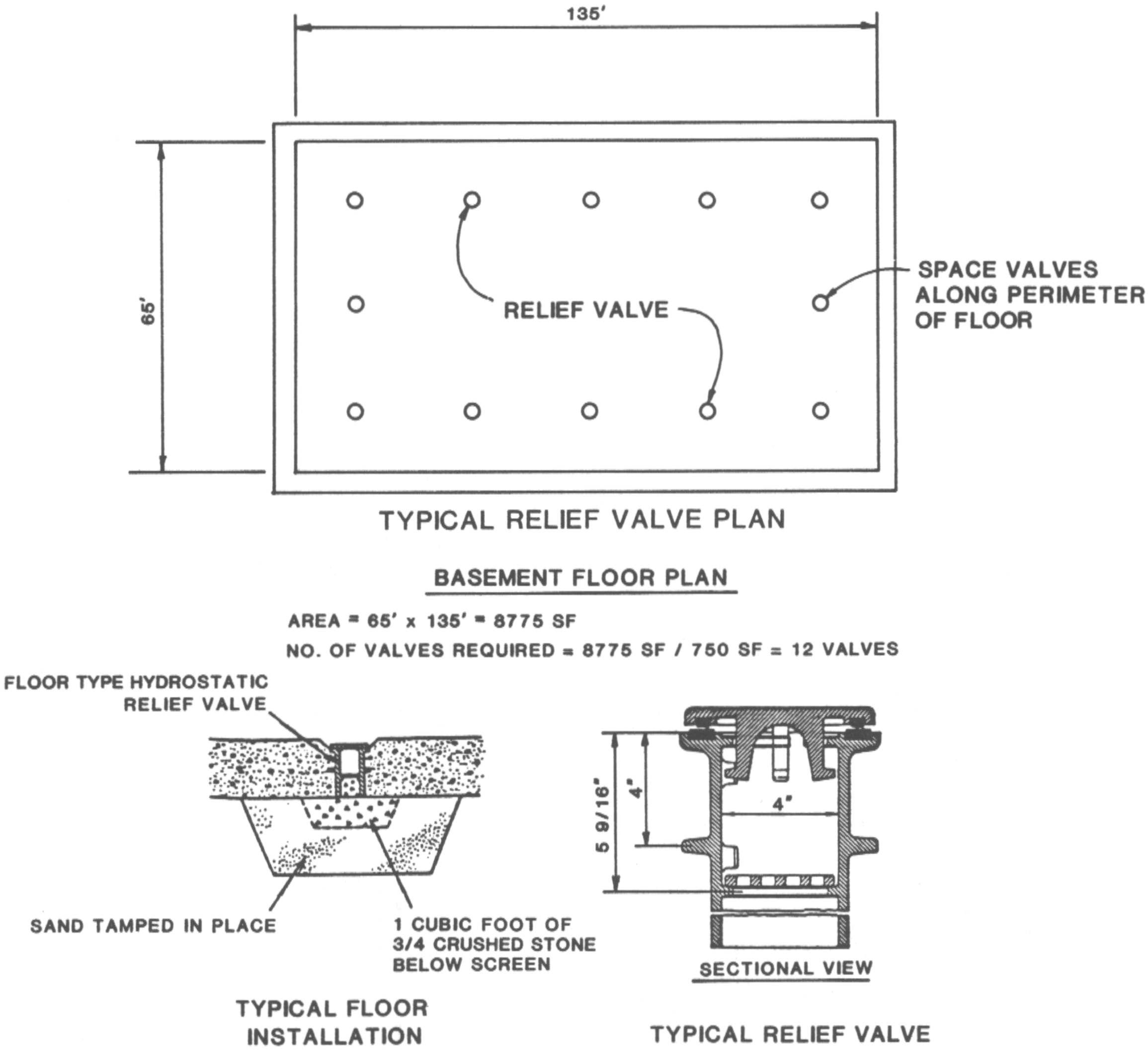


Figure III-20. Typical Pressure Relief Valve System

5. WATERPROOFING. Concrete and masonry walls are not generally impermeable unless special construction techniques are applied. Waterproofing can be accomplished through the use of (a) high-quality concrete, (b) sealant materials, and/or (c) impermeable membranes.

Sealing existing walls and floors can significantly increase hydrostatic pressures unless an alternative drainage system is provided. If an existing structure cannot be designed to withstand anticipated pressures, the most feasible course of action may be to allow water to continue to enter through existing structural faults and remove the water with a sump pump.

a) Integral High Quality Concrete Construction. An impervious concrete can generally be obtained by using a richer cement mix than normal with well-graded fine aggregate. The consistency of the concrete should be as stiff (low water content) as possible and the mixture should be thoroughly worked as it is placed. Leakage through joints can be prevented by the use of grouted structural keys and non-corrosive waterstops. Typical water-tight construction details are shown in Figure III-21.

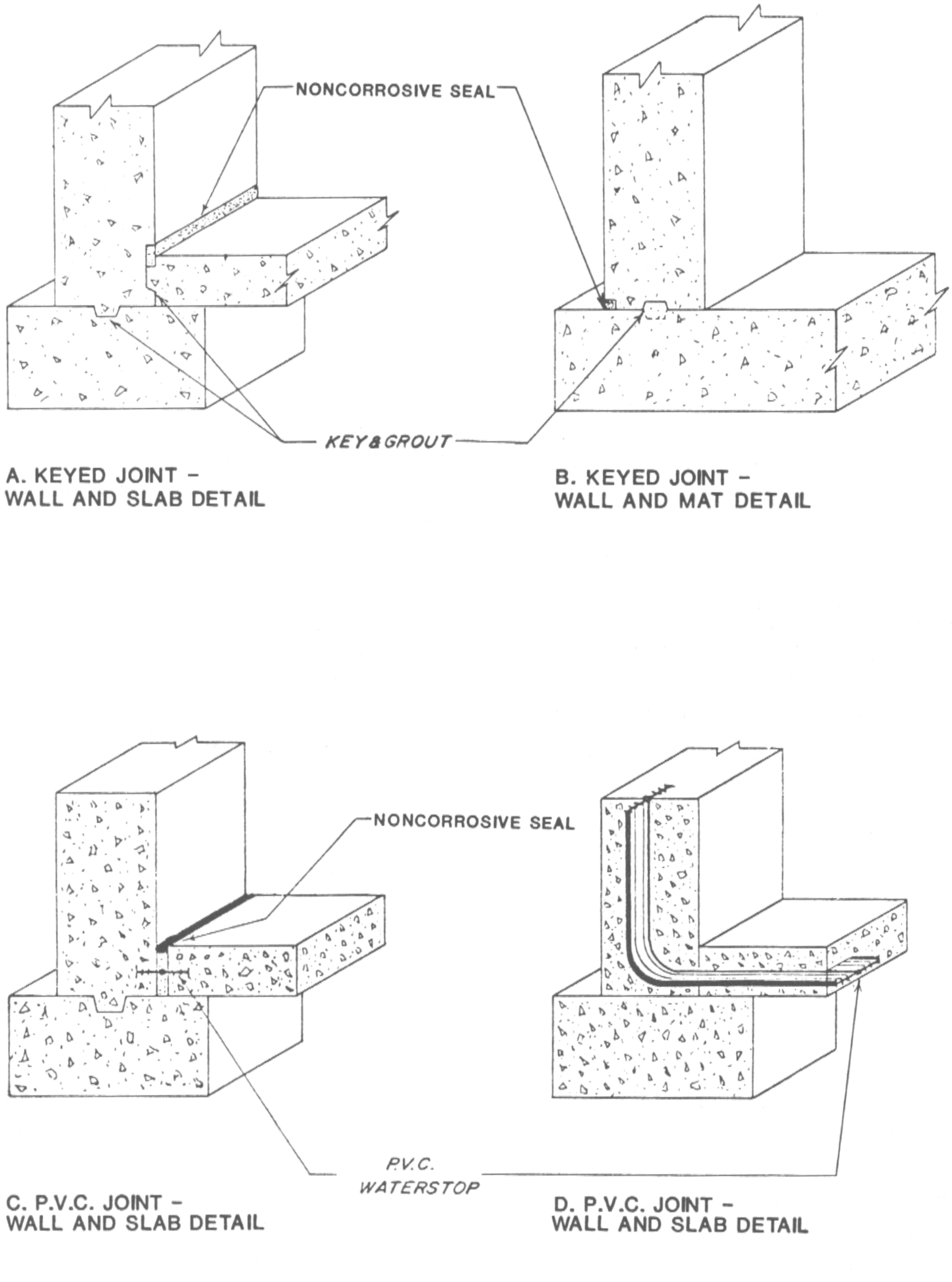


Figure III-21. Waterproof Wall and Foundation
Joint Details Integral Concrete Waterproofing

Source: Anti Hydro Company

The water-tightness of very lean (low in cement) concrete mixtures will be improved by the addition of almost any fine, inert material. Their function is to fill the voids or pores of the concrete with a more or less soapy, insoluble filler, and thus prevent the percolation of water through the concrete. Substances that may be used included finely ground clay or sand, hydrated lime, chloride of lime, oil emulsions, and lime soaps. The increased plasticity resulting from the use of this material will reduce segregation and improve workability. Water-repellent admixtures reduce absorption and retard moisture penetration by capillary action, but are not effective against water under pressure.

Waterproofing admixtures are commercially available. A typical mix design consists of 1 part portland cement and approximately 5 1/2 parts of clean, well-graded fine and coarse aggregates designed for maximum strength and denseness. Each cubic yard contains a minimum of 5.6 bags of portland cement and not more than 39 gallons of total liquid, which includes 1 1/2 gallons of the manufacturer's admixture. (Source: Anti-Hydro Company.)

b) Sealants. Masonry and concrete structures may be waterproofed by applying sealants to interior and/or exterior surfaces that are exposed to floodwaters (see Figure III-22). Common sealant materials include hydraulic or portland cements and a variety of bituminous materials that may be applied hot or cold. Exterior applications are generally preferred. Sealants may also be used between structural elements (i.e., between a structural floor slab and a concrete topping slab, or between a concrete masonry wall and a layer of brick veneer as shown in Figure III-23).

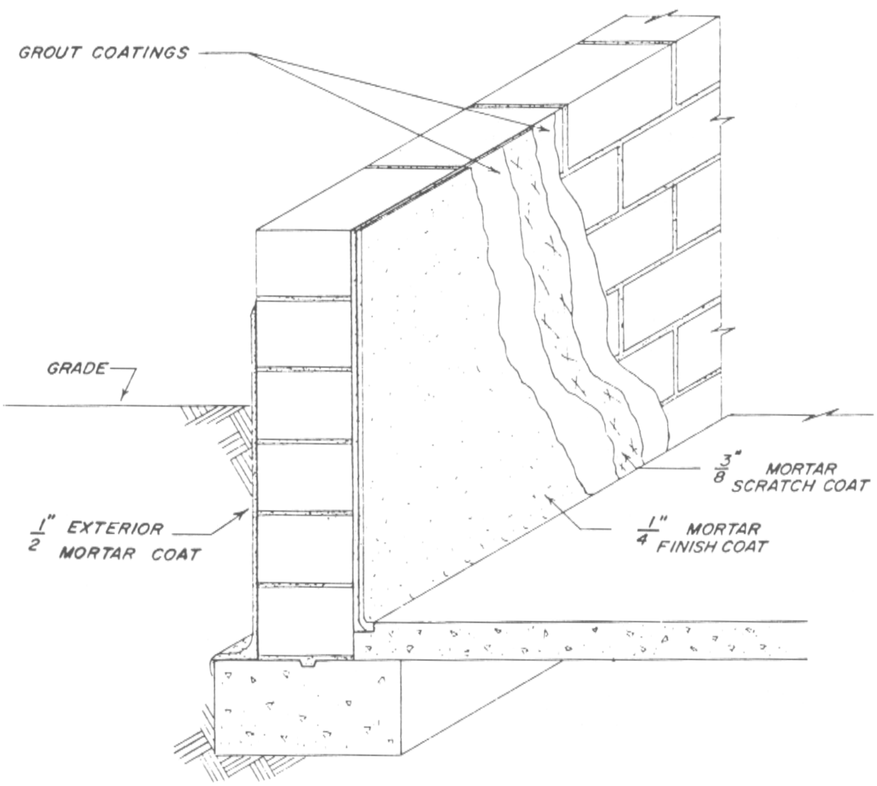


Figure III-22. Waterproofing With Mortar Sealant Coatings
Source: Anti Hydro Company

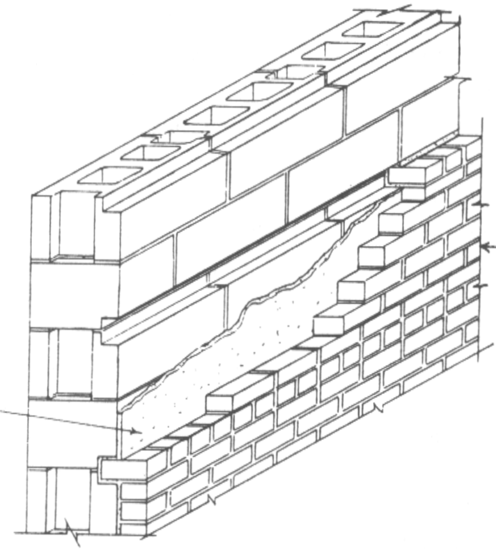


Figure III-23. Sealant Applied Between Masonry Block Wall and Exterior Veneer
Source: Anti Hydro Company

c) Membranes. The membrane method of waterproofing consists of surrounding all flood-prone surfaces of a structure with an impermeable membrane. Common membrane materials include PVC sheets, or coatings of felt, canvas or similar materials that are set in layers of hot bituminous coatings (coal tar, pitch, or asphalt). The membrane method of waterproofing is applicable to all types of masonry and concrete construction. To be effective, the membrane must be continuous and it should be protected against injury by a layer of brick, concrete or sand (Figure III-24). An existing building may be waterproofed on the inside by applying a membrane and then constructing an additional wall and slab within the existing wall and slab.

6. WATERTIGHT CORES. When waterproofing of exterior walls is not feasible for either physical or economic reasons, it may be possible to create a watertight core around an interior area. Watertight cores are particularly effective when costly items are located together in a small part of the building. For example, vital utilities or expensive equipment might

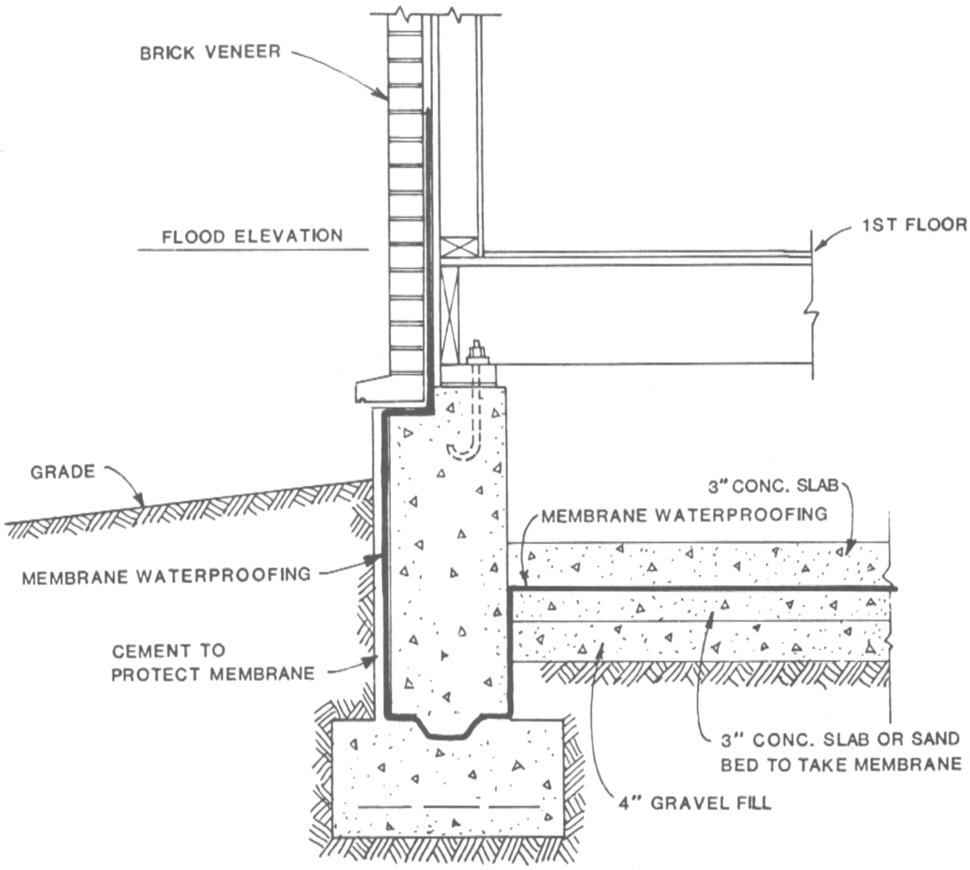


Figure III-24. Membrane Waterproofing

be enclosed by such a core. The top of the core wall must be as high as the Design Flood elevation plus suitable freeboard. A typical watertight closure is constructed of reinforced cast-in-place conrete. The design of core walls and the floor system follows the guidelines discussed in previous sections. The core system must be capable of withstanding uplift and lateral flood forces, and all water-proofing considerations discussed earlier must be met.

With the exception of very low walls (less than 18 inches), access openings, steps or ramps must be provided. The use of openings requires that flood shields be available (to be presented in the next section). An advantage of this type of access is that normal entry and exit to the area occurs during non-flood conditions. Disadvantages are the difficulty in assuring a watertight seal for the shield, storage of the shield, and insuring that the shield is properly installed in a timely manner.

Providing steps as access to the area eliminates the problems associated with flood shields, but entails more difficult entry and exit. This may be a problem for areas of heavy traffic. In addition, steps may not be feasible if bulky or large amounts of material must be moved in and out of the area. Access for handicapped personnel is also limited. Ramp access eliminates many of the problems of both openings and steps.

Ramps may even be made to accommodate machinery, if necessary. The primary disadvantages to a ramp system is the additional space required for the ramp.

The type of access provided for a watertight core is a function of the particular needs and usage of the area as discussed above and must be selected by the designer.

7. CLOSURES AND FLOOD SHIELDS. If the walls and floor of a structure can be designed or modified to provide the required impermeability and resistance to flood forces, then permanent or temporary closure systems may be used. Closures and shields must be able to support all flood loads that act on their surfaces. In addition, the closure or shield must be installed so that flood loads are uniformly transferred into the supporting walls or structural elements of the building.

For existing buildings, permanent closures are preferable if they do not alter the function or safety of the structure. Unused openings may be permanently sealed with concrete, masonry blocks or metal assemblies. All closure assemblies should be reinforced and keyed or anchored to the framing system, floor, or walls.

Flood shield assemblies must be used to protect openings that cannot be permanently closed. Shields may be constructed of any durable material that can withstand the design loads. The most common materials are steel and aluminum. Exterior grade plywood may also be used for openings that are not exposed to extreme loading conditions. For example, A-C grade exterior 3/4″ marine plywood may be used with a maximum recommended unsupported span of 24″. Plywood should be coated with fiberglass. Neoprene rubber gasket material may be used as a seal. Aluminum or steel reinforcement may also be used. Experience has indicated that it may be simpler and as cost effective to fabricate steel closures than to try to adapt plywood to this use.

The shields should normally be attached to the wet side of the opening so that the pressure of the water helps to seal the flood shield to the receiving frame. The frame, usually metal, should support the shield on at least three edges. Shields may be attached to their frames with standard bolts, T-bolts, latching dogs, wedge assemblies, or a variety of other latching devices. Preference should be given to simple, quick disconnect fasteners that can be activated with a minimum of time, effort, and skill. Regardless of the type of latching mechanism, the shield must be designed to ensure a watertight seal.

Several types of flood shields are illustrated in Figure I-10. Figure III-25 through III-31 summarized below, provide details of various framing, sealing and latching techniques.

FIGURE	TECHNIQUE DESCRIBED
III-25	Recommended reinforcement of masonry walls around small openings
III-26	Flood shield for small basement window
III-27	Bond beams & vertical reinforcement of flood shields at large openings
III-28	Flood shield for typical door openings
III-29	Typical flood shield for display windows
III-30	Typical flood shields for horizontal openings below Design Flood level
III-31	Typical flood shield fastening methods

These details have been adapted from *Floodproofing Regulations* as published by the U.S. Army Corps of Engineers.

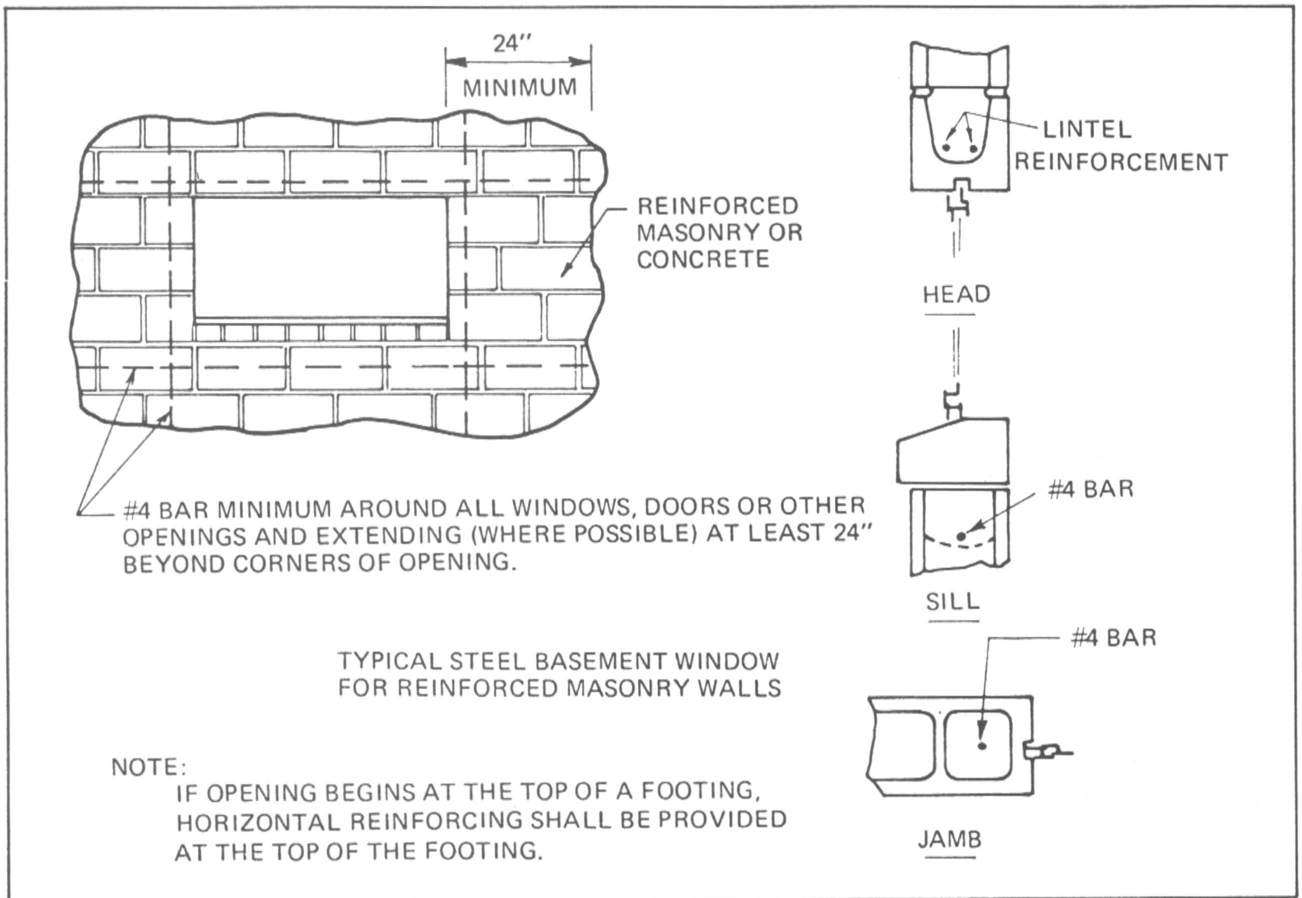


Figure III-25. Recommended Reinforcement of Masonry Walls Around Small Openings

Source: *Floodproofing Regulations*

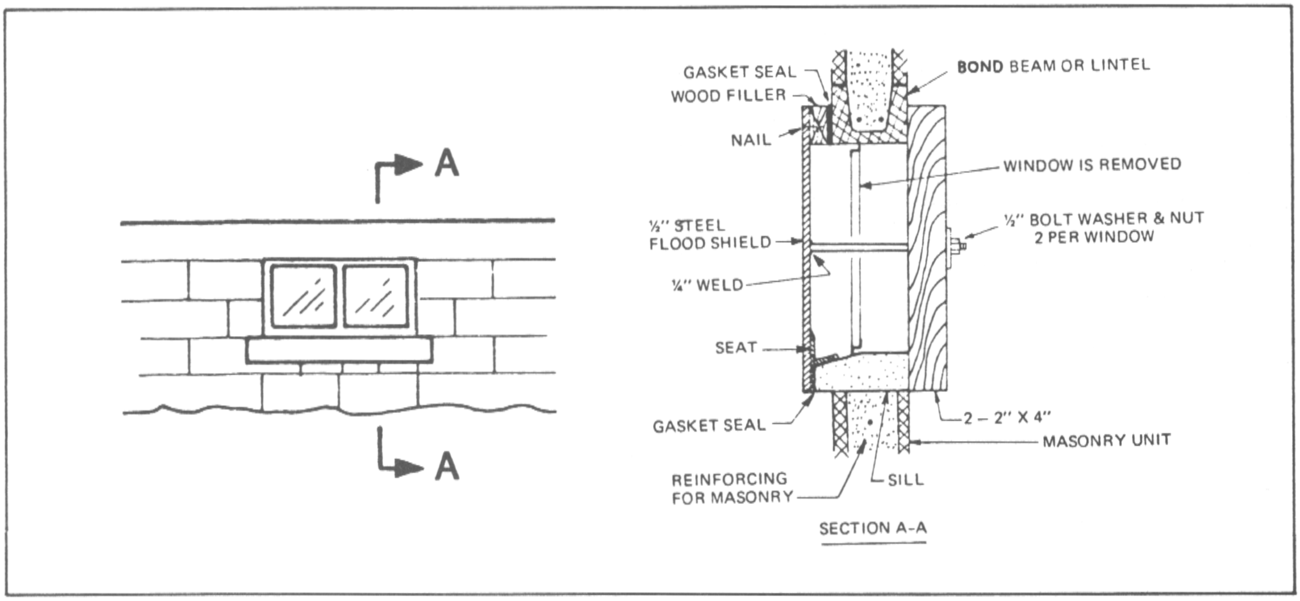


Figure III-26. Flood Shield for Small Basement Window

Source: *Floodproofing Regulations*

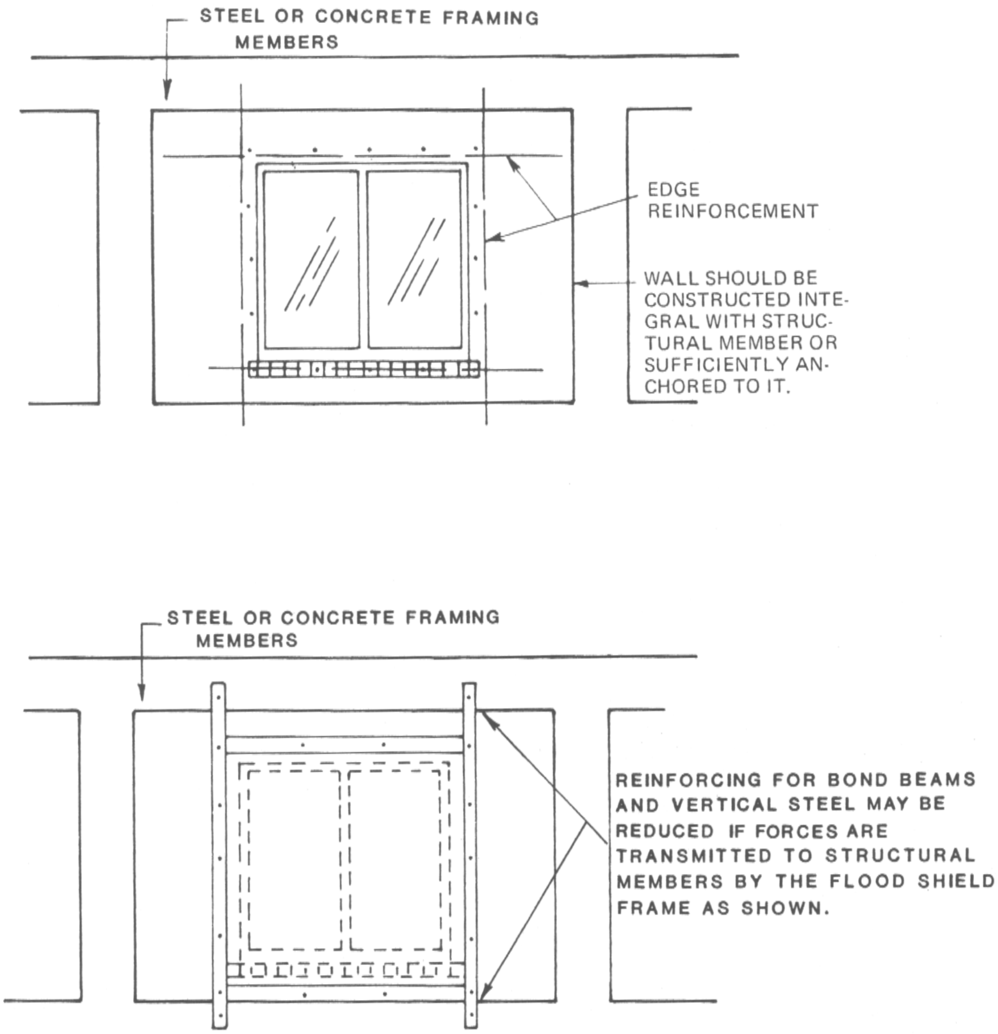
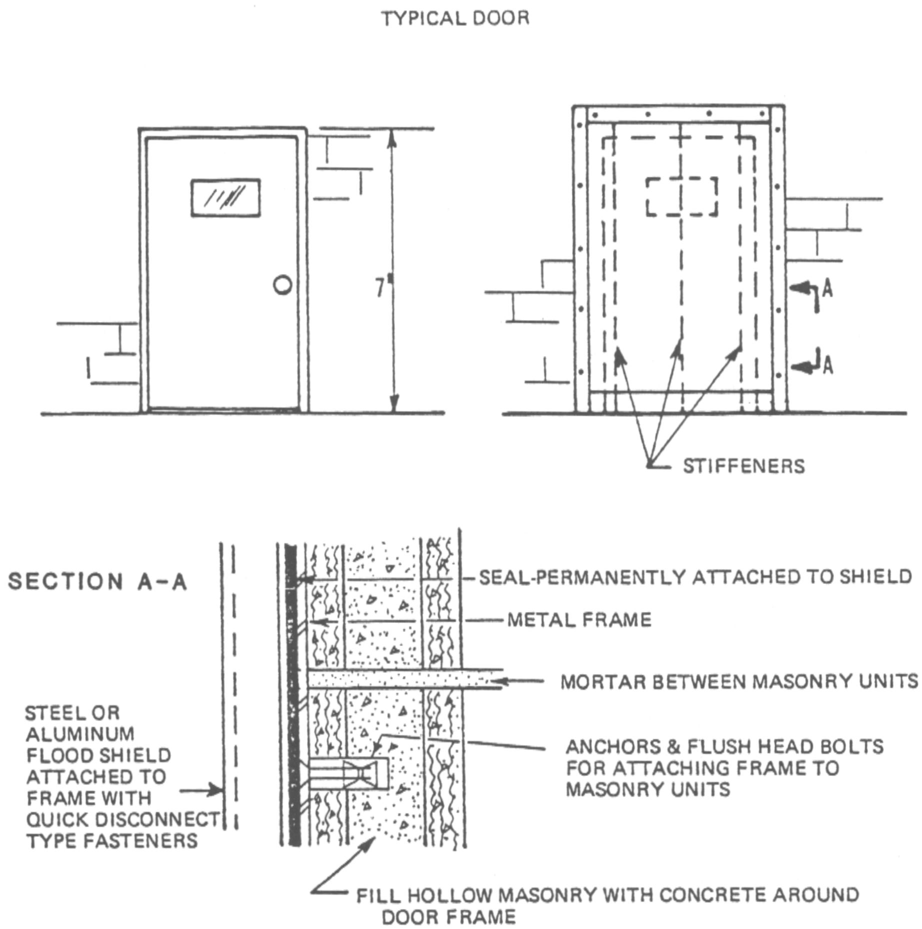


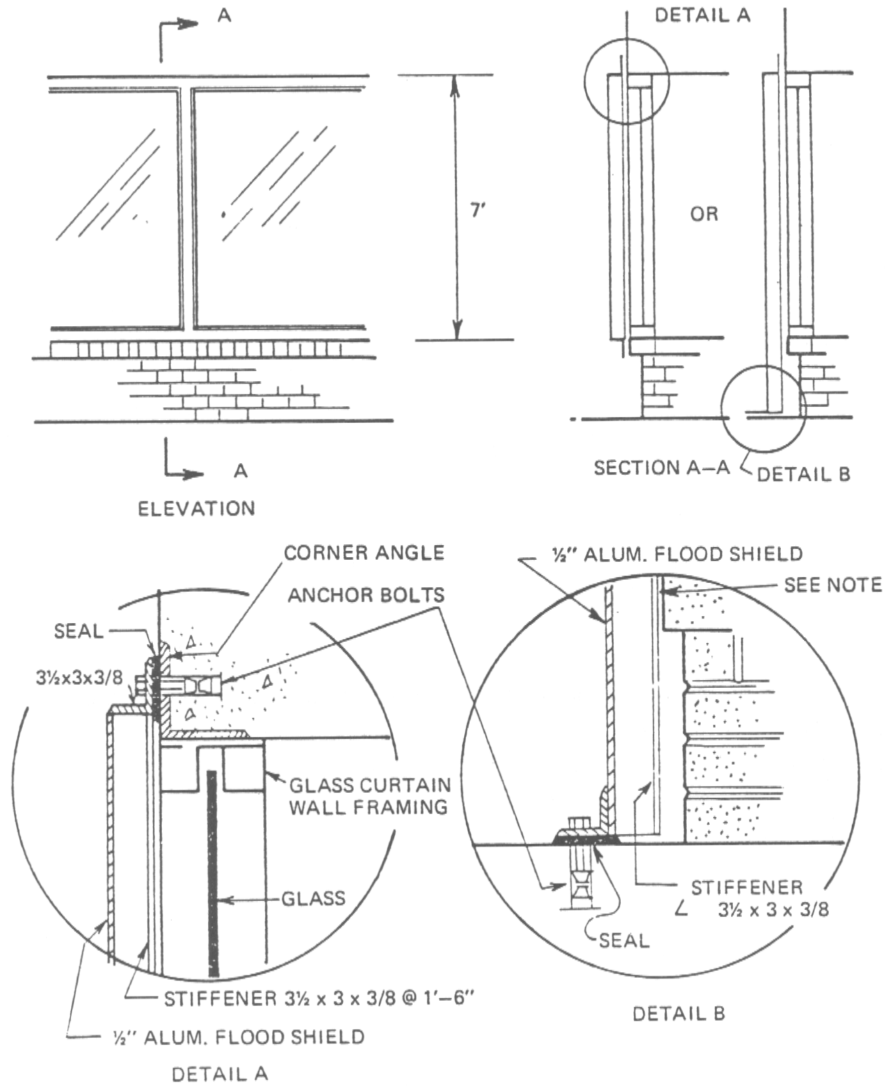
Figure III-27. Bond Beams & Vertical Reinforcement of Flood Shields at Large Openings
Source: *Floodproofing Regulations*



ALL CELLS AROUND OPENINGS IN HOLLOW MASONRY CONSTRUCTION SHOULD BE FILLED WITH CONCRETE. LARGE OPENINGS SHOULD HAVE BOND BEAMS, VERTICAL REINFORCEMENT, AND METAL FRAMES AROUND OPENING.

MORTAR JOINTS THAT LIE WITHIN FLOOD SHIELD SHOULD BE STRUCK FLUSH WITH THE MASONRY UNITS SO THERE WILL BE A BETTER SEAL.

Figure III-28. Flood Shield for Typical Door Opening
Source: *Floodproofing Regulations*



Note: The shield material specifications assume that support is available at the bottom of the display window (ie 7 ' high shield). If support is not available at this point, increase size or number of stiffeners and provide support at bottom. Members are sized for water level at top of display window.

Figure III-29. Typical Flood Shield for Display Windows
Source: Flood Proofing Regulations

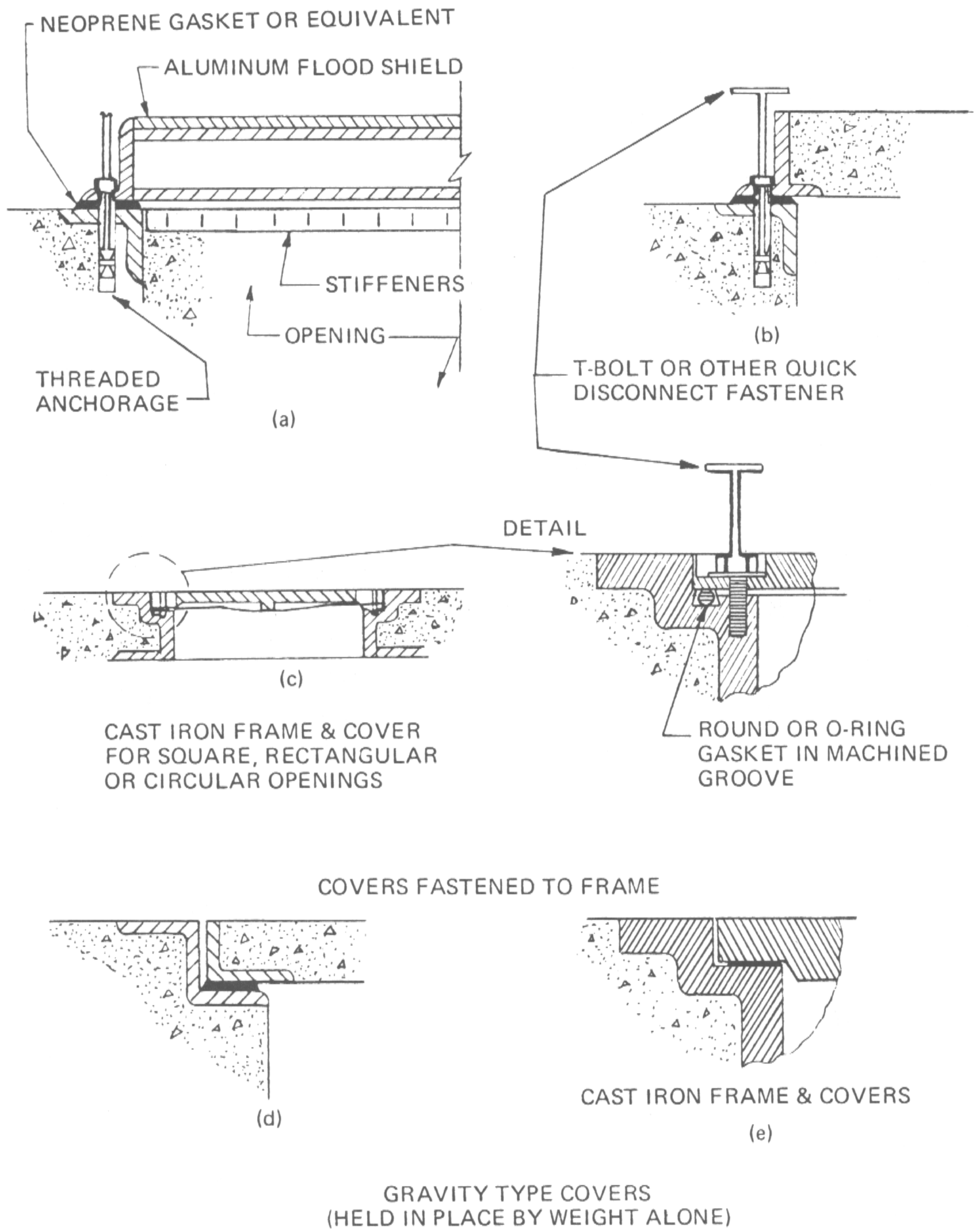


Figure III-30. Typical Flood Shields for Horizontal Openings Below Design Flood Level
Source: Flood Proofing Regulations

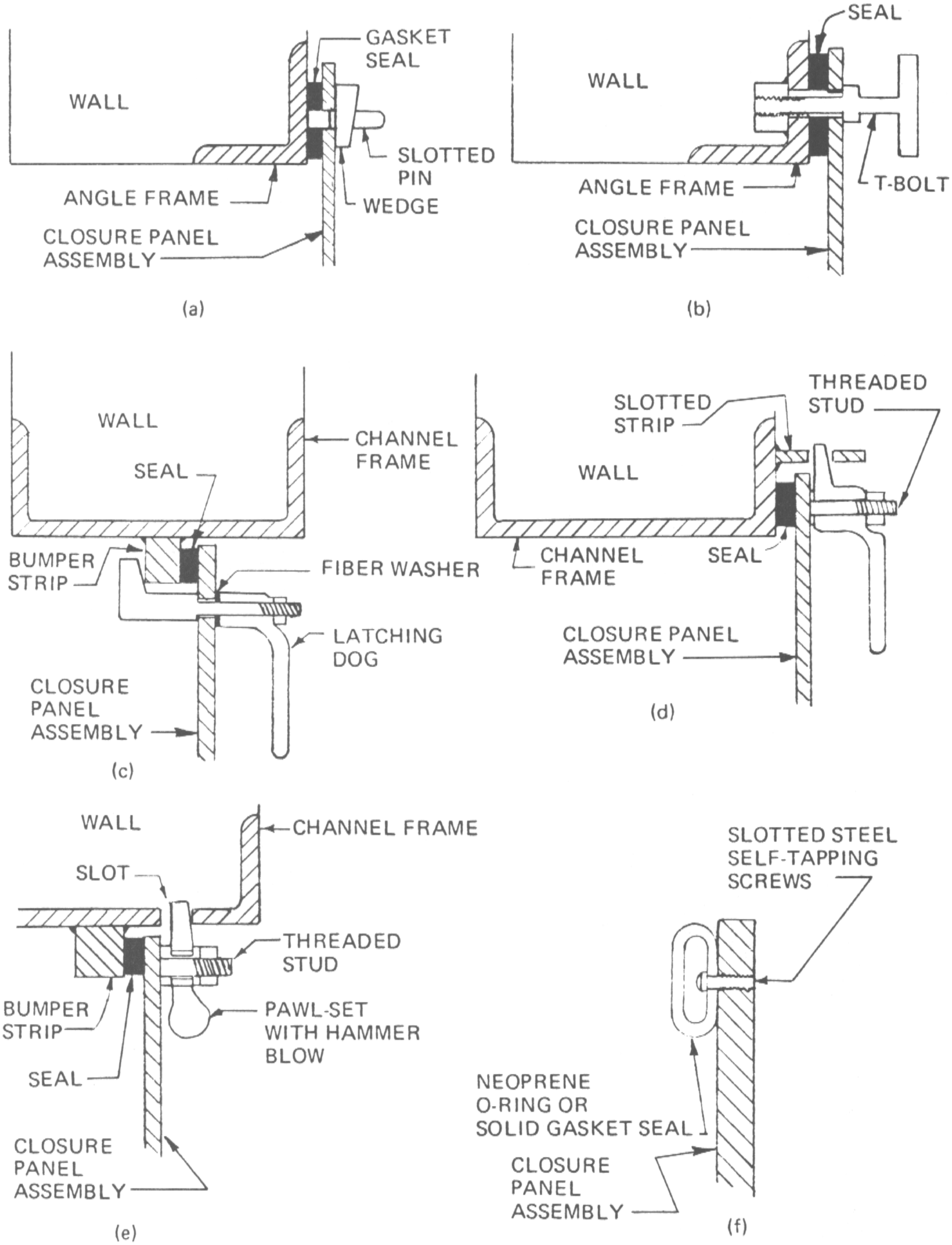


Figure III-31. Typical Flood Shield Fastening Methods
Source: Flood Proofing Regulations

A variety of flood barriers and watertight doors are available commercially. Doors are closed by sliding, hand dogs or wheels and can be pneumatically sealed. Doors and barriers are constructed of structural steel or aluminum plate. Figures III-32 through III-37 illustrate some of the available doors and barriers.

FIGURE	TECHNIQUE DESCRIBED
III-32	Watertight hinged double doors
III-33	Watertight quick acting hinged doors
III-34	Watertight sliding door
III-35	Bottom hinged flood barrier
III-36	Manually installed flood barrier
III-37	Fork lift installed flood barrier

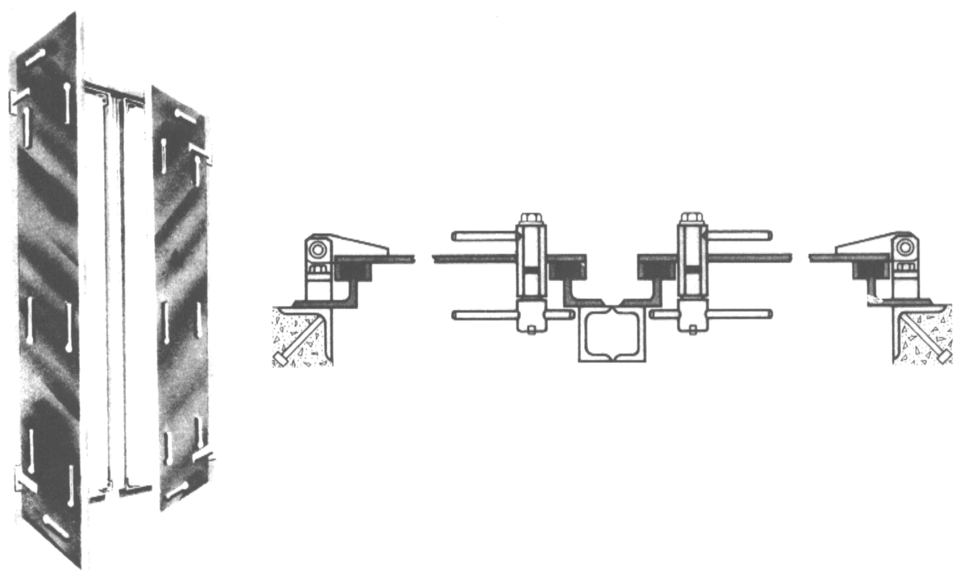


Figure III-32. Watertight Hinged Double Doors

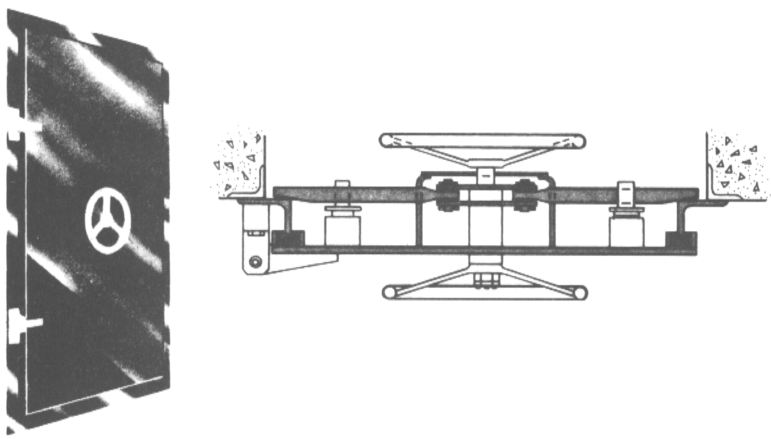


Figure III-33. Watertight Quick Action Hinged Doors

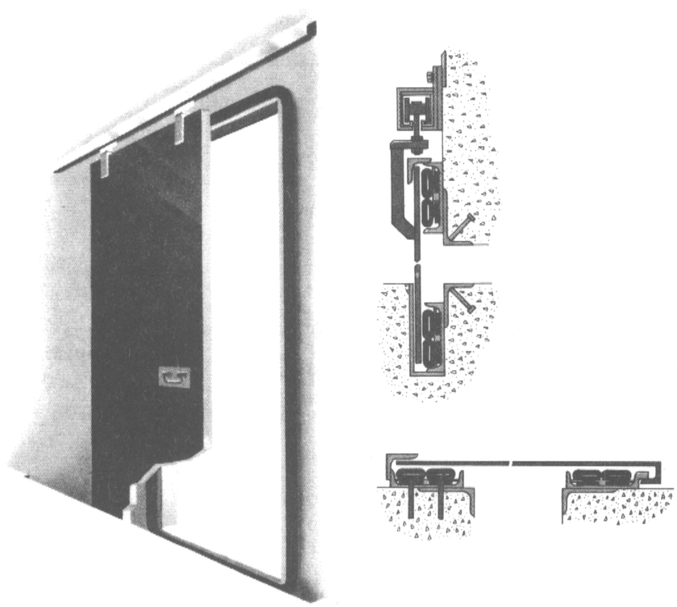


Figure III-34. Watertight Sliding Door

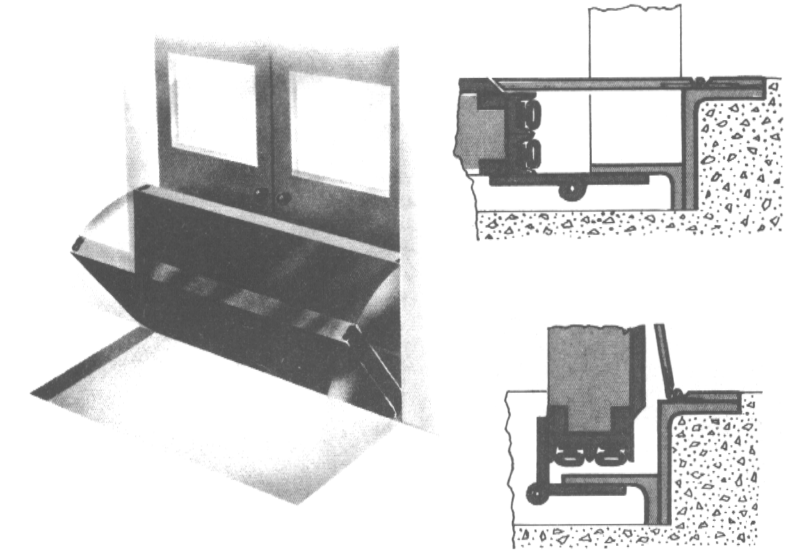


Figure III-35. Bottom Hinged Flood Barrier

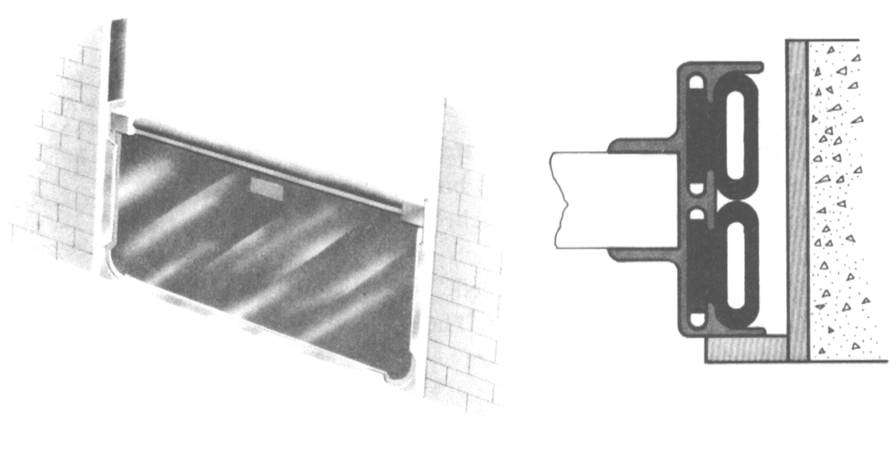


Figure III-36. Manually Installed Flood Barrier

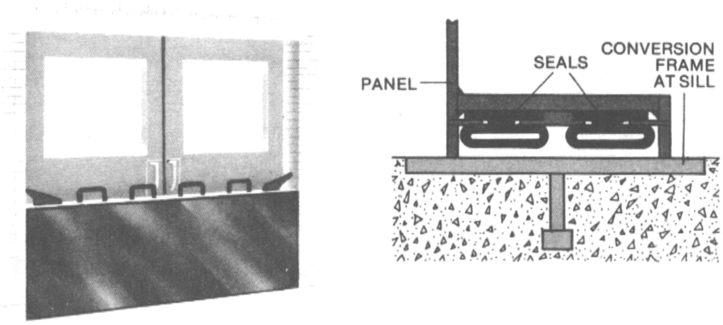


Figure III-37. Fork Lift Installed Flood Barrier

8. TESTING, STORAGE AND MAINTENANCE. It is recommended that new flood shields should be installed and tested before they are used. Testing may be performed by constructing a concrete block wall or plywood bin around the outside of the installed shield and filling it with water to at least the Design Flood elevation. A plywood bin may be constructed of 3/4″exterior plywood attached to 2″ x 4″ studs and 2″ x 4″ braces at 16 or 24 inch spacing. For very large openings, mortar reinforced concrete blocks may be used to construct the bin. The bins should be lined with polyethylene to minimize water loss. The test depth should be monitored as frequently as necessary to ensure full hydrostatic loadline throughout the test period. The length of the test period should always be greater than that which would be expected in actual flooding, but never less than 24 hours. During the test the interior of the shield should be monitored frequently to determine the location and extent of any leakage that may occur. (See Figure III-38).

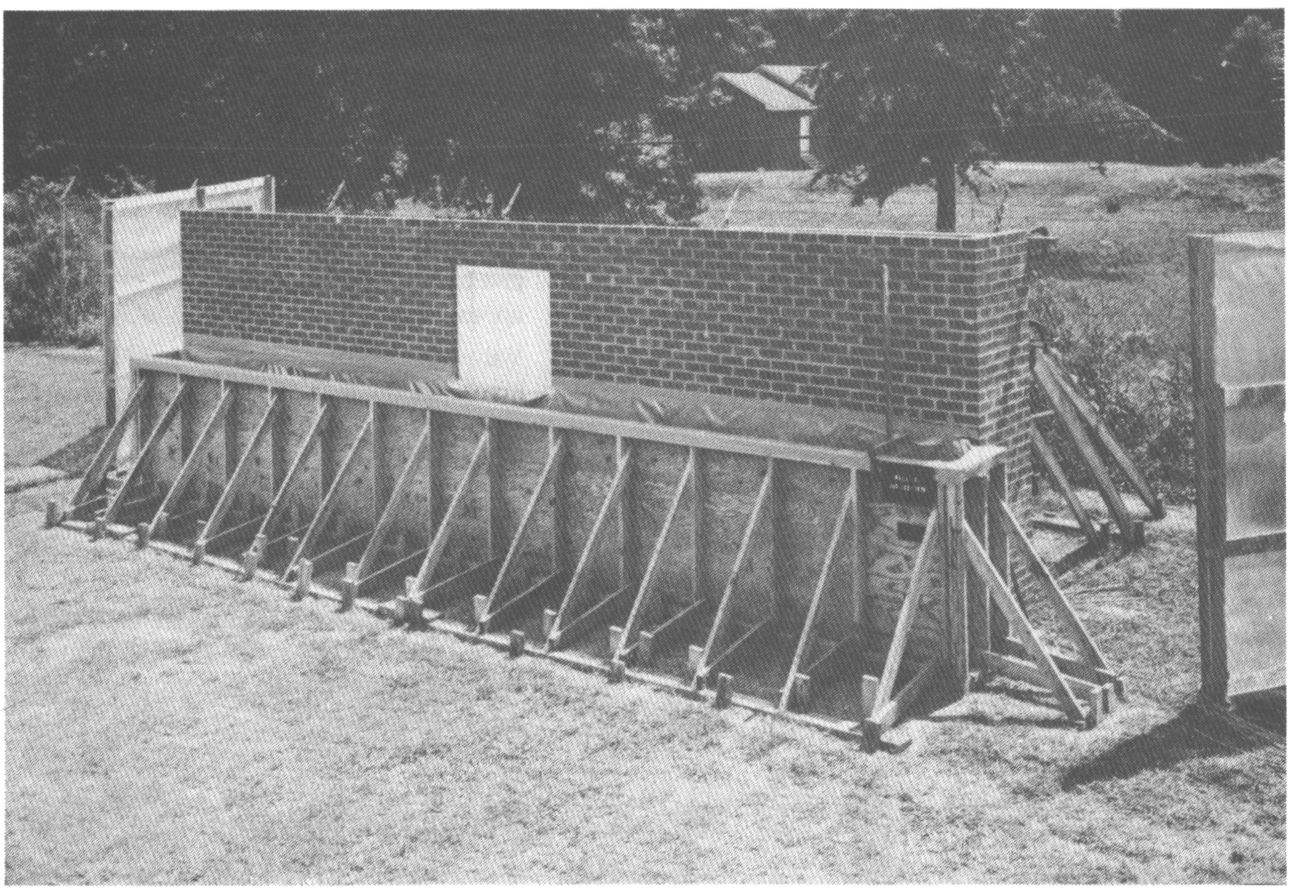


Figure III-38. Testing Bin

Provisions must be made for storing flood shields when they are not in use. Storage areas must be carefully planned and maintained to ensure that the shields can be located and installed with a minimum of effort and time. The storage area should be as close to the openings to be sealed as possible. In addition, any tools, hardware, or equipment that is needed to attach the shields should be conveniently located at the storage area or installation site.

For complex flood protection systems, a master checklist for the installation of shields, pump operation, and valve closures should be prepared. Pump and valve locations and all shields should be numbered and color-coded based on installation and operation priority. For example, low levels of flooding might have a white color code, with intermediate levels up to the design flood. Figure III-39 illustrates the format of such a checklist.

For the most part, permanent closures, doors, and barriers require little or no special maintenance. If the closures use gaskets or sealants, these items will have to be inspected annually and perhaps changed every ten years.

Flood shields require more attention. Flood shields should be inspected and function tested at least once a year to assure serviceability. All of these temporary systems require gaskets and sealants which must be checked and replaced as necessary.

All systems, such as sump pumps, special utility protections, and backflow preventor or check valves in sewers require annual testing.

CHECKLIST

Priority - WHITE

Item Number	*Bldg.*	*Item*	*Notes*	*Remarks*
1	#3	Shield	2	
2		Shield	1,2	
3		Door		
4	#1	Double Door		

Priority - YELLOW

5	#2	Shield	2	
6		Double Door	1	
7	#3	Valve	2, 3	Main Drain Valve

Priority - ORANGE

8	#1	Valve	2, 4	Main Drain Valve
9		Shield	2	
10	#2	Valve	2, 5	Main Drain Valve

Priority - RED

11	#2	Rolling Door		
12		Hatch	6	

Notes:

1. Normally closed.
2. Tools required.
3. Valve closes a main drain. Pumps should be prepared to start pumping when water appears in sumps at this location.
4. Valve closes down main drains. If system is not surcharging, it may be left open, but pumps should be ready for continuous operation.
5. Valve closes overflow from sump. Can be shut at any time after water recycling system is shut down.
6. For hatch, apply a layer of polyethelene and hold down with sandbags. Overlap the frame by 2 feet and apply the sandbags three deep along the edge.

Figure III-39. Sample Flood Protection Installation Master Checklist

E. FLOODWALLS AND LEVEES

1. GENERAL. As described in Chapter II, floodwalls and levees may be used to prevent floodwaters from reaching an individual structure and adjacent functional land areas. Floodwalls and levees may be used to protect a structure on all sides, or to protect the low side of a structure that is located on the edge of the floodplain.

Experience has shown that floodwalls and levees can be used to effectively protect individual structures from flooding depths up to 7 feet. The feasibility of floodwall protection for depths that exceed 7 feet are often limited by the cost of design and construction; while the height of a levee is generally limited by the amount of construction space that is required to accomodate embankment side-slopes.

The design requirements for a particular floodwall or levee are generally variable and complex. However, the information presented in this section can be used to evaluate the initial feasibility of a floodwall or levee at a particular site and to develop conceptual design plans. This section begins with a presentation of site survey (Part 2) and internal drainage (Part 3) requirements that are applicable to floodwall and levee projects. Part 4 and 5 present guidelines that are unique to the design of floodwalls and levees, respectively.

2. SITE SURVEY. Floodwall and levee design analysis should begin with a careful review of the site-specific factors that govern the feasibility of these measures. As an initial step, hydrologic data should be gathered and reviewed (as discussed in Chapter II) to determine the Design Flood elevation, anticipated flood water velocities, the duration of flooding and the potential impact of floodwall or levee construction on existing channel capacity. All regulatory restrictions associated with floodwall or levee construction shall be investigated to determine the feasibility of obtaining any required construction permits (see Chapter II).

Once this information is known, topographic maps can be used to identify the most logical location and alignment of the floodwall or levee. The structure grade or design elevation must be established to protect against the Design Flood plus allowances for residual settlement and/or freeboard. (Freeboard is the vertical distance between the top of the floodwall or levee grade and the Design Flood elevation.) The freeboard allowance provides a margin of safety against wave and scour action, overtopping, and the inherent uncertainties of estimating techniques used in establishing the Design Flood elevation. Freeboard allowances for floodwalls and levees have not been strictly standardized, but as a general rule, a minimum value of 3 feet is often used. However, freeboard of less than 3 feet, even as low as 1 foot, may be acceptable, depending upon applicable construction regulations, provided that protection against the Design Flood can still be achieved. The latter conditions would more likely be attained for the construction of floodwalls, due to their comparatively greater structural integrity.

Topographic maps may also be used to evaluate potential problems of surface drainage accumulation on the 'dry' side of the floodwall or levee, and in the identification of appropriate access points through, across, or above the proposed structure.

After the floodwall or levee alignment has been established, the designer must assemble geotechnical information to determine the properties of foundation soils that will support the floodwall or levee. For levee design, it will also be necessary to identify the physical properties of available construction material. Initial geotechnical studies must determine soil bearing capacity, permeability, and depth to an impervious stratum. For small floodwalls and levees (less than 10 feet high, 1000 feet long) a limited number (depending on the homogeneity of underlying conditions) of soil test borings supplemented by a thorough field reconnaissance will generally provide adequate design information. Foundation materials have been classified as:

a) Ledge Rocks. Ledge rocks present a potential permeability hazard and frequently need grouting.

b) Fine Uniform Sands. If below 'critical density' (void ratio at which a soil can undergo deformation without change of volume) fine uniform sands must be consolidated to prevent flow when saturated under load.

c) Coarse Sands and Gravel. From a stability standpoint they will consolidate under load. A streamside impervious blanket may be required to prevent seepage.

d) Plastic Clays. They require careful analysis to assure that shear stress imposed by the weight of the levee or floodwall is less than the shear strength of the foundation material; flattened levee side slopes may be required to reduce shear stress.

If the preliminary investigations identify specific problems, more detailed geotechnical studies may be required.

3. INTERIOR DRAINAGE SYSTEM. Floodwall and levee systems must be designed to reduce or eliminate the accumulation of seepage and/or internal surface runoff on the dry side of the structure. If adequate space is not available to temporarily store all seepage and runoff that is likely to occur at the site, excess water must be drained to low lying sump areas and pumped to the wet side of the floodwall or levee. The pump discharge level should be located above the Design Flood level.

The drainage system for the interior area enclosed by a levee or floodwall must accommodate the precipitation runoff from the interior area and the anticipated seepage through the levee or floodwall during flooding conditions. A means of positive drainage for the interior of the floodwall or levee area is needed to discharge the accumulated water outside the enclosed area.

First, a collection system composed of pervious trenches or underground tiles must be designed to transport the accumulating water to a sump area. In the levee application, these drains should be incorporated into the collection system. The anticipated seepage from under and through levees and floodwalls must also be taken into consideration. To determine the amount of precipitation that can collect in the enclosure, the rainfall intensity must be determined for a particular location.

Using Figure III-40, a value is obtained in inches per hour. This value should be multiplied by both the area in square feet and a conversion factor of 0.01. The product will be in gallons per minute. In some cases, a levee or floodwall extends only partially around a property and ties into higher ground. For these cases, the amount of precipitation that can flow downhill as runoff into the enclosure must be included. To calculate this value, the area of land in acres that can discharge water into the enclosure should be estimated. This value is then multiplied by the previously determined rainfall intensity and by the most suitable terrain coefficient provided in Table III-2. The product of these three values is the rate of flow in gallons per minute into the enclosure.

TABLE III-2
TERRAIN COEFFICIENTS

Roof	.85
Street, parking lot	.85
Urban area, paved areas	.80
Industrial area	.70
Residental area (homes or apartments)	.60
Unimproved vegetated areas	.20
Grass Area grade is 7 percent or more	.25
Grade is 2 percent to 7 percent	.15
Grade is flat to 2 percent	.10

Seepage flow rates from the levee must also be estimated. In general, unless this seepage rate is calculated by a qualified soils engineer, a value of one gallon per minute for every 100 square feet of levee or floodwall enclosed area should be assumed. The values for precipitation within the enclosed area, runoff areas uphill draining into the enclosure, and seepage through the levee or floodwall should be added together, and the sum multiplied by a safety factor of 1.5.

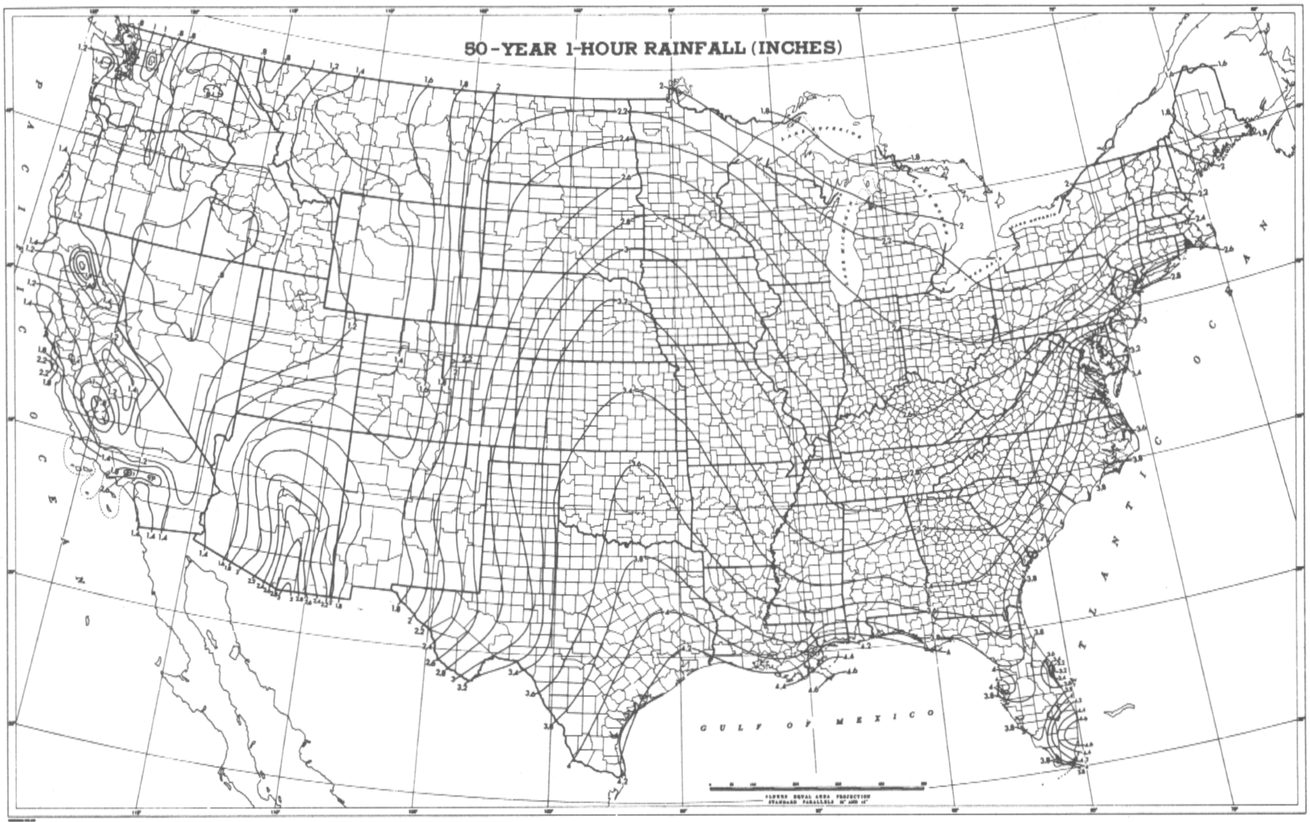


Figure III-40. Rainfall Intensity for 50-year, 60-Minute Duration For United States.

The result is the minimum discharge size in gallons per minute (gpm) of the sump pump. The pump to discharge the collected water from the interior of the area should be a submersible-type model mounted in the sump basin with a backup electrical generator. The backup electrical generator should be available during power outage, which is often the case during flooding conditions.

Under normal circumstances, the electrical service from the structure can operate the pump. The pump controls should consist of three float-type mercury tube switches to activate the pump, turn it off, and to signal high water levels. The pump motor should be fully submerged in an oil-filled chamber providing efficient heat dissipation, permanent lubrication, and sealing for complete protection from the environment. The pump should have a semi-open, non-clog type impeller capable of passing a 2-inch solid sphere without damage. The housing should be cast iron with corrosion resistant fasteners and a mechanical seal between the pump and motor. A check and gate valve should be installed on the discharge piping.

An alternative might be a suction-type pump powered by a gasoline engine. A control system should consist of water level switches automatically operating an electric starter for the gasoline engine. The pump performance should match that of the submersible pump described above. The major disadvantages of this system are the need for constant monitoring of fuel levels, and the additional cost of control and starter implementation.

During non-flood situations, surface runoff within the protected area may be discharged through drainage pipes or culverts that extend through the floodwall or levee. These outlets must be equipped with an automatic check valve to prevent backflow during a flood. Backflow prevention valves will also be required on all sewer and other underground utility lines that extend into the floodproofed building (see Chapter IV, part C).

4. SEEPAGE. If a floodwall or levee is constructed on impervious soils that extend riverward for a considerable distance, seepage beneath the structure may not represent a problem. However, underseepage through pervious foundation materials can cause hydrostatic pressures at the dry side base of a floodwall or 'toe' of a levee. This pressure may result in piping beneath the structure and heaving and rupturing of adjacent soils.

There are a variety of techniques that can be used alone or in combination with each other to control underseepage. These techniques include landside berms, impervious cut-offs, pervious trenches and pressure relief wells as described below.

a) Landside Berm. Landside uplift pressure can become greater than the effective weight of a levee structure. The construction of a landside berm (where space is available) can eliminate this hazard by providing additional weight to counteract uplift pressures at the toe of the levee. A landside berm may be used to reinforce an existing impervious or semipervious top stratum; or, if none exists, the berm may be placed directly on pervious deposits.

b) Impervious Cut-off. Where foundation and/or levee construction material is relatively permeable, an impervious cut-off should be installed to reduce seepage. Impervious cut-offs for levees include sheet piling or cement curtain cut-offs (Figure III-41), compacted impervious fill that extends to an impervious stratum (see Figure III-42), or an impervious blanket (Figure III-43). Sheet piling and cement curtains may also be used to prevent seepage beneath the key of a floodwall (Figure III-44). For cases where pervious foundation materials are deep, initial consideration should be given to steel sheet piling because of the relative ease of installing this type of cut-off.

c) Pervious Trench. If properly installed, the impervious cut-off system described above will eliminate major piping of water under a floodwall or levee. Where a cut-off is not provided or where it is probable that the cut-off will not eliminate all underseepage, it may be necessary to collect remaining seepage in a pervious trench. The trench may be installed with or without a drain pipe.

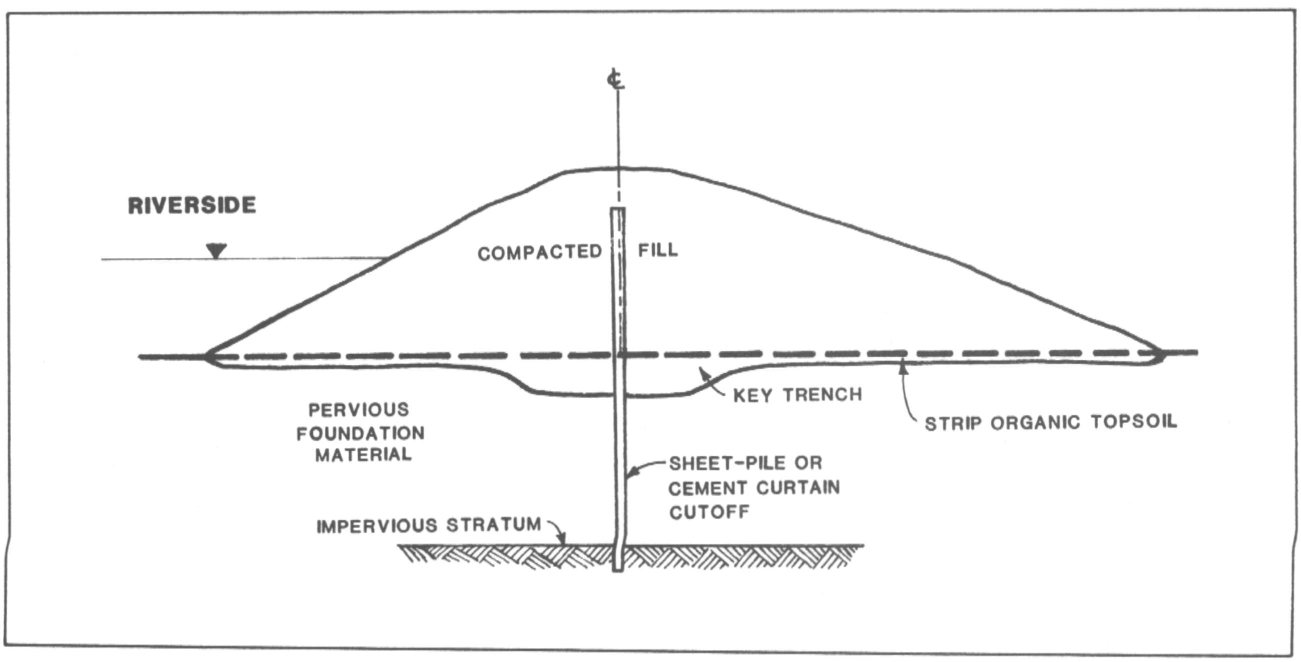


Figure III-41. Levee Underseepage Controlled By Sheet-Pile or Cement Curtain Cut-Off

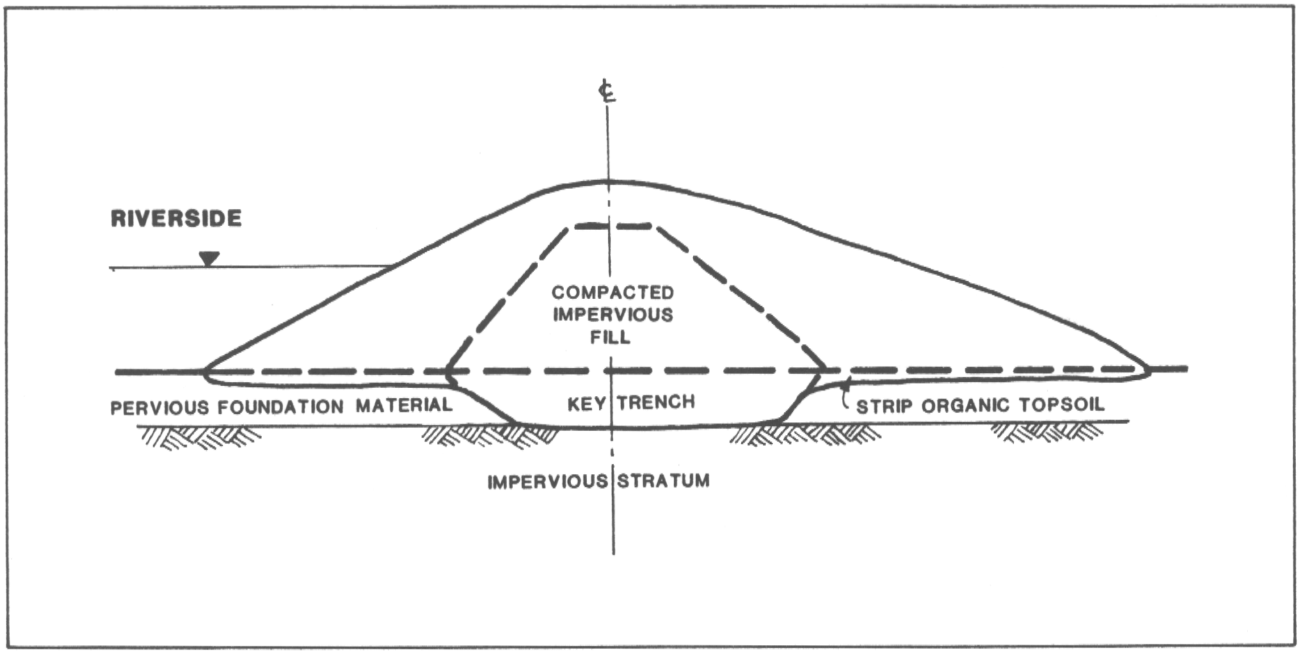


Figure III-42. Levee Underseepage Controlled By Compacted Impervious Core

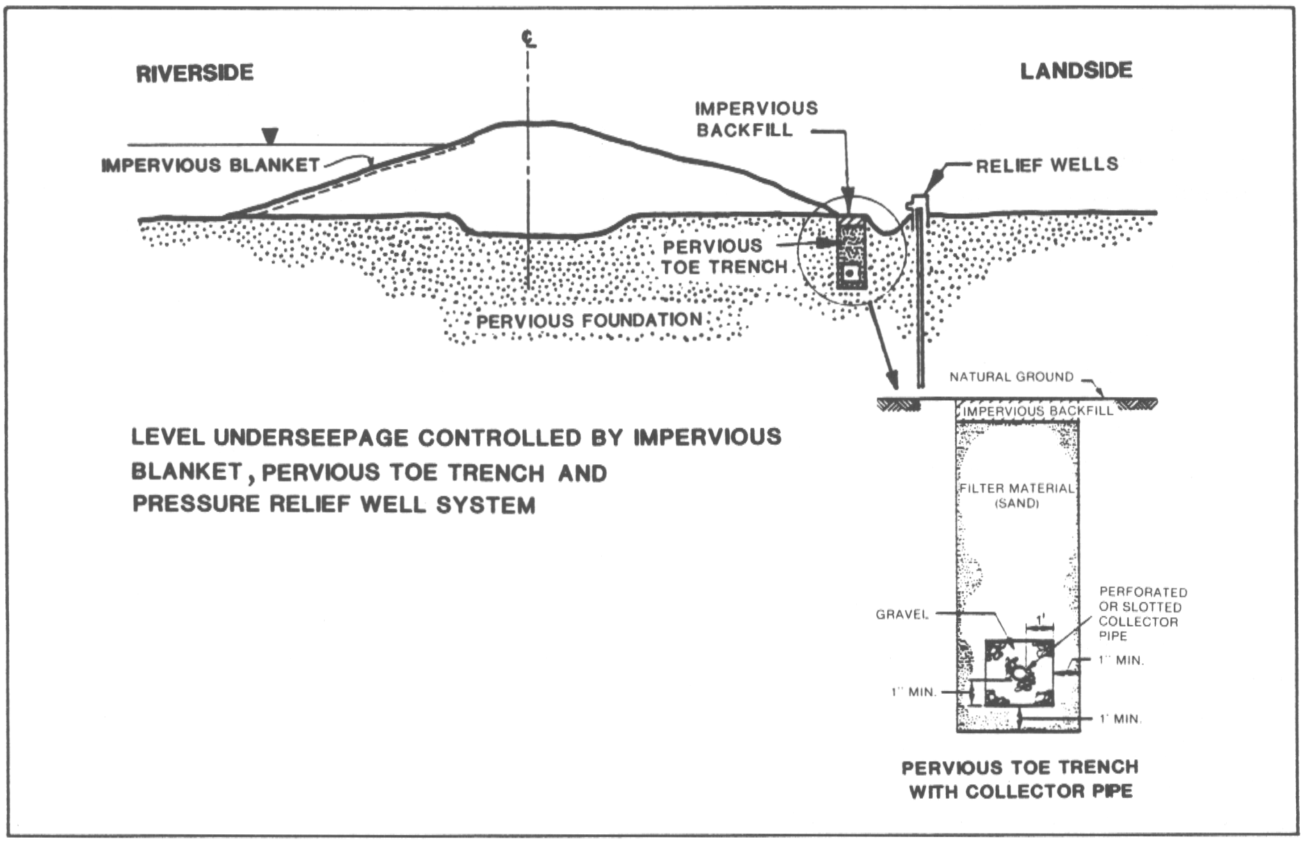


Figure III-43. Level Underseepage Controlled by Impervious Blanket, Pervious Toe Trench and Pressure Relief Well System

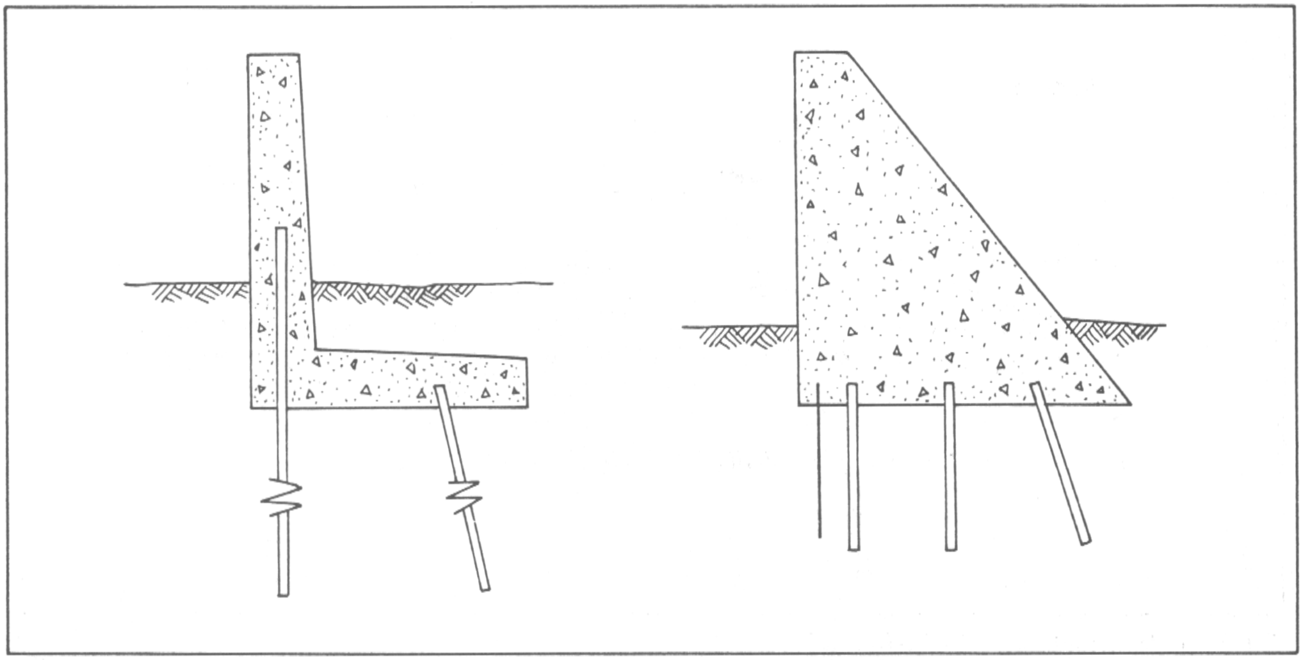


Figure III-44. Floodwall Underseepage Controlled by Sheet Pile or Cement Curtain Cut-Off

A pervious toe trench (Figure III-43) is effective for collecting underseepage where an underlying pervious stratum is thin (or where a cut-off has been used) and the trench can, therefore, intercept a large percentage of the seepage. For the case of thick underlying pervious strata, a blanket/toe drain system will be more effective in collecting deep seepage. Occasionally, it may be advantageous to locate the pervious trench towards the center of the levee system (Figure III-45) and to discharge intercepted seepage through a horizontal blanket drainage layer. There is some advantage to a location under the levee in that the trench can also serve as an inspection trench and because the blanket drain can help to control seepage that may occur through the levee embankment.

d) Pressure Relief Wells. Pressure relief wells may also be installed along the landside toe of a levee or floodwall system to reduce uplift pressure. These wells are designed to intercept and control seepage and associated hydrostatic pressures. They are particularly effective where pervious foundation strata are too deep to be penetrated by cut-offs. A relief well system can be expanded if the initial installation does not provide adequate control. Wells require periodic maintenance and generally suffer loss in efficiency with time. This efficiency loss is caused by muddy surface waters, bacterial growth, or carbonate incrustation that tend to clog the well screens. Figure III-46 illustrates a typical pressure relief well.

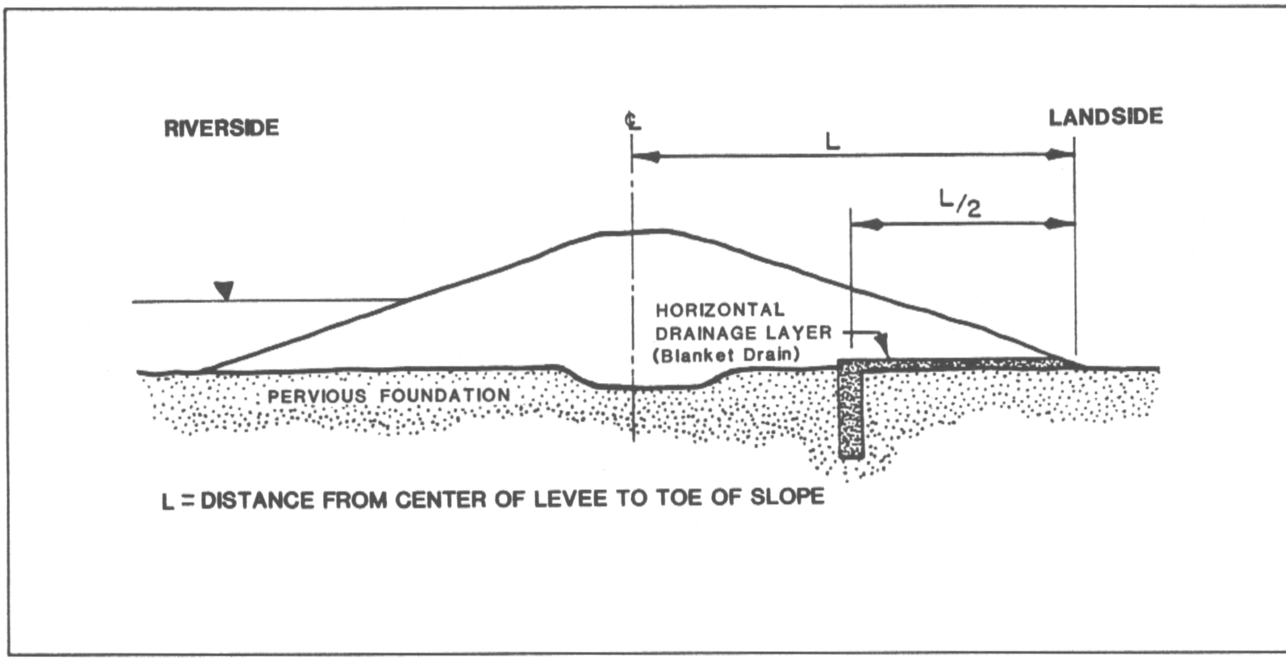


Figure III-45. Blanket Drain Beneath Levee

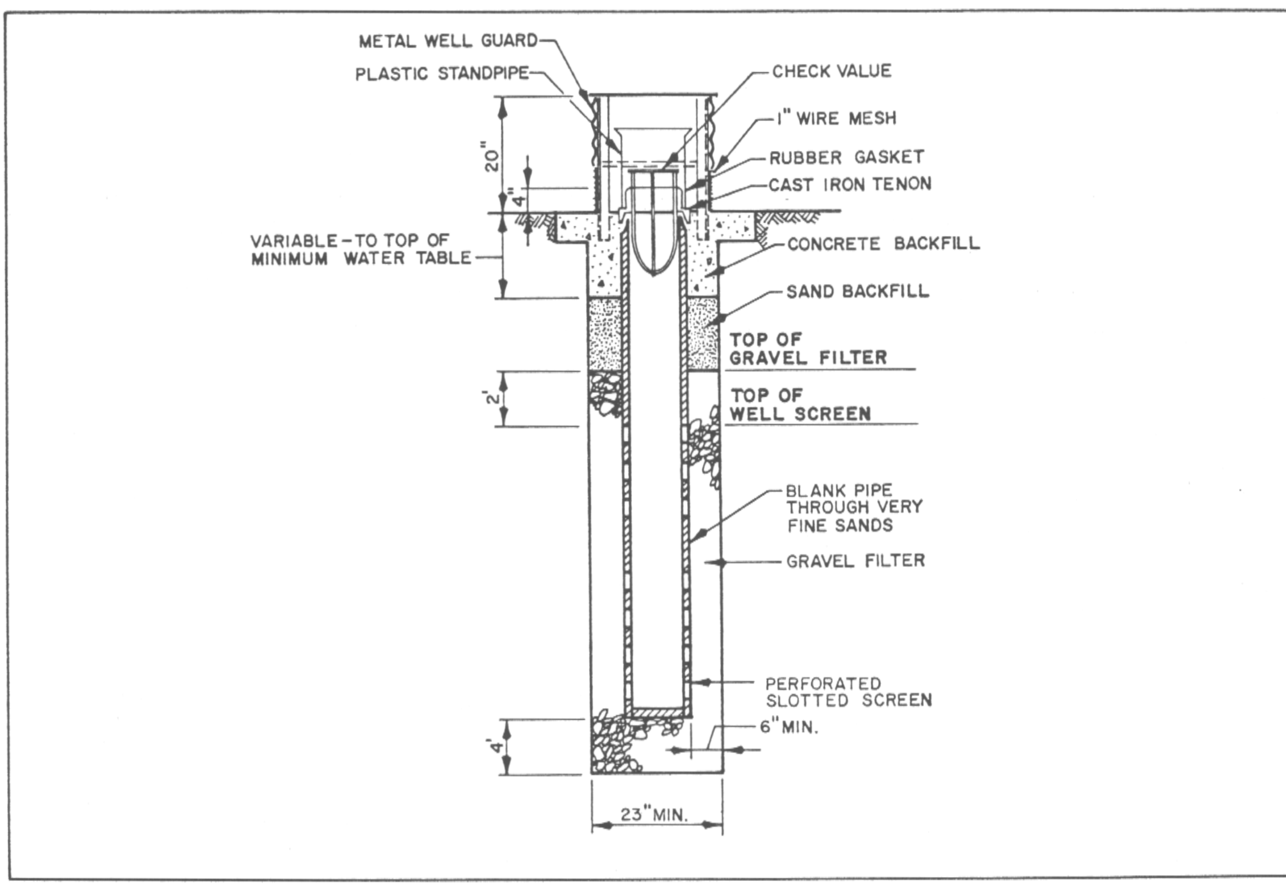


Figure III-46. Pressure Relief Well

Source: U.S. Army Corps of Engineers, Design and Construction of Levees, EM 1110-2-1913

5. FLOODWALL DESIGN. If it has been determined that an area would best be protected by the construction of a floodwall, a wide range of configurations, construction materials, and other variations are available. The design of any type floodwall, whether fixed or movable, must address two broad concerns: the overall stability of the wall as related to external loads, and the design of all wall features for sufficient strength as related to calculated internal stresses.(see Figure III-47).

a. Structural Design of Permanent Floodwalls. The stability of a floodwall (or any structure) can be defined as the ability to develop sufficient reactions to prevent gross movement under load. A structure may be strong enough to maintain its shape under load, but be unstable due to geometry or support conditions. A stability analysis of a proposed floodwall design includes consideration of overturning due to unbalanced moment, sliding due to unbalanced lateral load, and failure of the underlying soil due to high lateral and vertical loads. These three concerns are illustrated in Figure III-48.

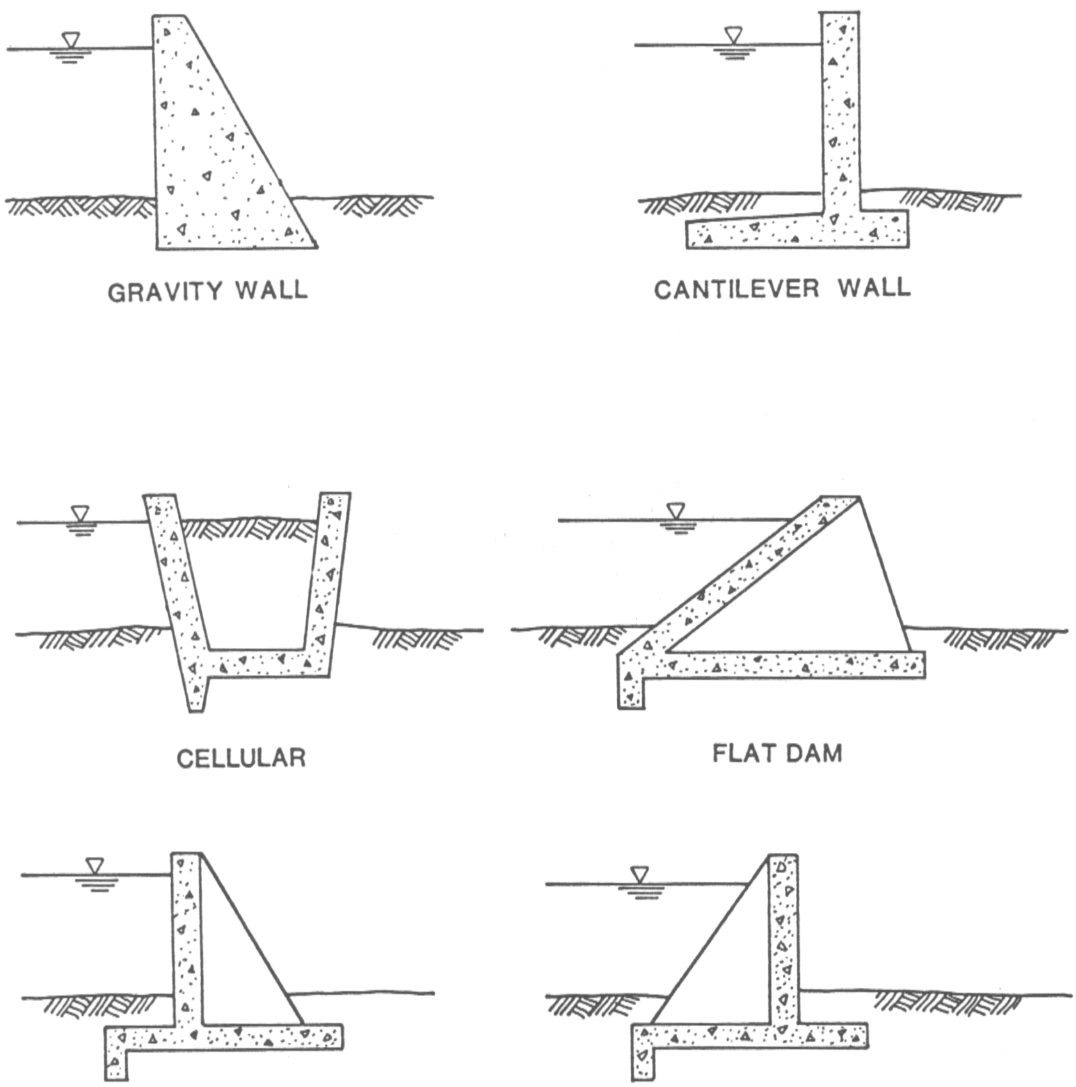


Figure III-47. Various Floodwall Types

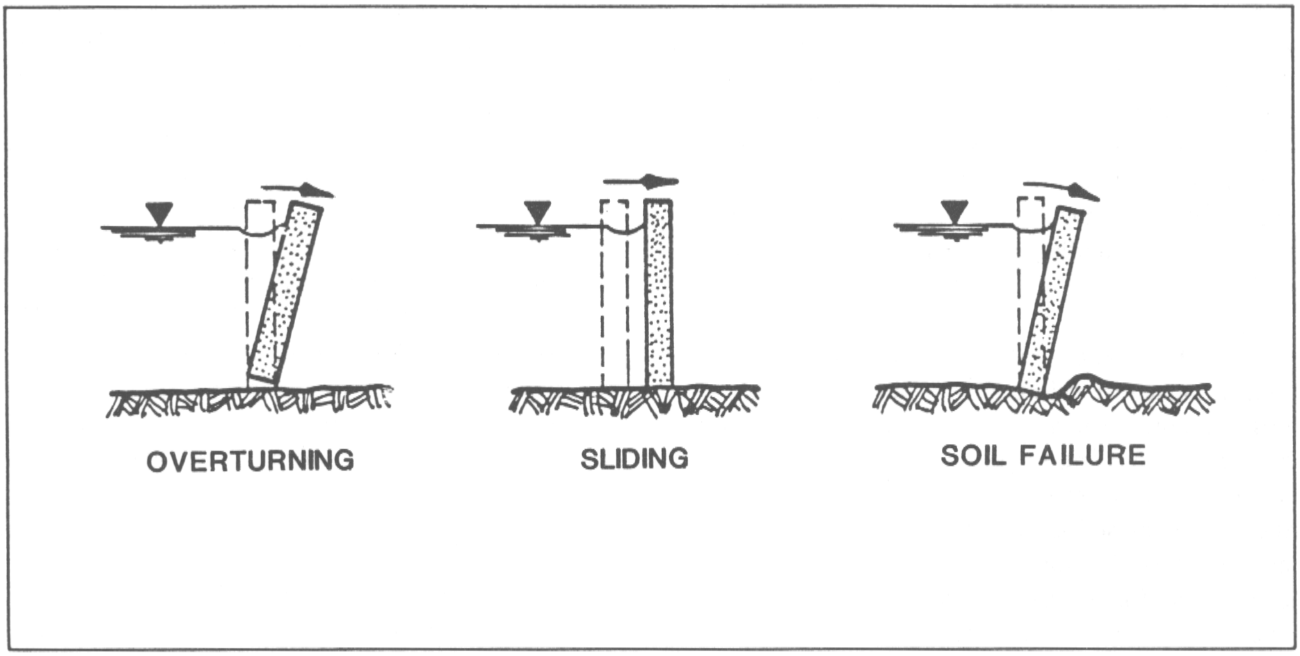


Figure III-48. Floodwall Stability

b. Gravity Walls. In designing the gravity wall, the stability of the structure and its supporting foundation materials represents the major design consideration. The structural stability of a gravity wall is attained through effective positioning of the mass of the wall rather than by depending on the weight of the retained materials.

The gravity wall resists overturning primarily by the dead weight of the concrete construction; it is simply too heavy to be overturned by the lateral flood load. To overturn the gravity wall illustrated in Figure III-49, the applied loading must cause the concrete to rotate about the lowest point of its axis on the side away from the load, and this movement is resisted by the concrete mass which tends to rotate the wall in the opposite direction (counterclockwise in the figure) about the same point. For a given wall height, more overturning resistance is added by increasing its top width (C) and/or its bottom width (L), which will increase the volume and weight of the concrete, or the distance from the center of mass of concrete to the point of rotation, or both.

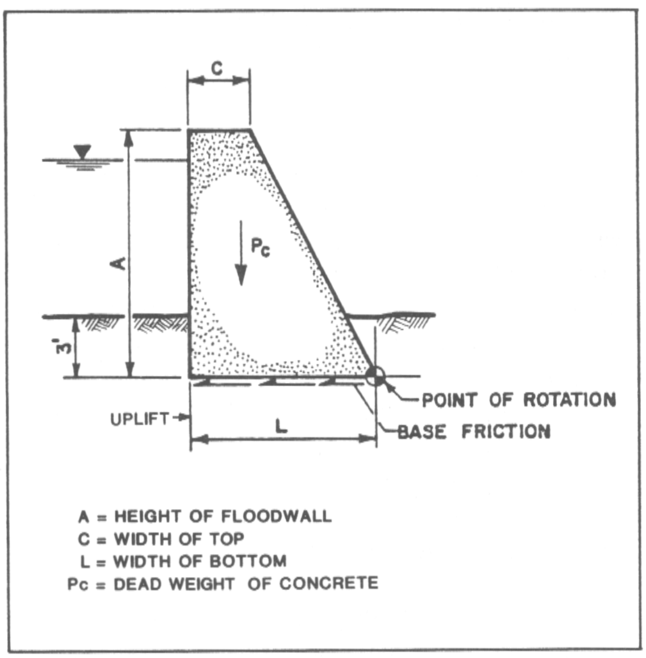


Figure III-49. Stability of Gravity Floodwalls

Sliding is generally resisted by frictional forces between the concrete base and the soil foundation.

The magnitude of this force depends on the vertical pressure between concrete and soil due to the weight of the wall, and on the size of the base area over which the friction acts. Sliding is further resisted by passive resistance, or resistance to displacement, or the soil mass behind the floodwall on the land side. Resistance to sliding in the gravity wall can be increased by increasing the volume and weight of the structure or by adding a shear key to the base of the wall.

Soil foundation stability is achieved by ensuring that the structure neither moves nor fails along possible failure surfaces including the surface bounded by the structure and the supporting foundation. Vertical contact pressure along the base of the wall on the underlying soil is caused by the wall dead load and any overlying soil or water, and also from overturning forces related to lateral loads. The overturning forces tend to cause higher contact pressures at points further from the wetted face of the wall. Two methods of controlling the resulting contact pressure are to increase the size of the base to spread the loads over a greater contact area, or to rearrange the geometry to minimize the effect of the overturning forces. This must be accomplished with due regard to satisfying the requirements of overturning and sliding. In areas where the floodwall must be founded on weak soil, the requirement for maintaining low contact pressure often governs the design of the wall.

In summary, gravity walls are appropriate for low walls or lightly loaded walls. They are relatively easy to design and construct. The internal stresses in gravity walls are low. Therefore, they may be constructed with minimal reinforcing if they are properly jointed. The primary disadvantage of gravity walls is that a large volume of concrete is required. At some point, it becomes more cost effective to use a cantilever wall. The cantilever wall (as discussed below) is more complex, but considerably less concrete is required. Therefore, cantilever walls are more cost effective for most floodwall applications.

c. Cantilever Walls. For gravity walls, the resistance to potential overturning can be increased not only by increasing the wall weight, but also by increasing the distance from the center of mass to the point of rotation of the wall. The mechanism of the cantilever wall is an extension of this method of resisting rotation by reducing weight and extending the lever arm. In addition, the cantilever wall utilizes the potentially stabilizing dead weight of both soil and floodwater as these materials exert overturning forces on the structure.

For the cantilever wall shown in Figure III-50, a significant portion of the weight that contributes to stability is the weight of the water above the toe 'T' of the base. This effect is offset to some extent by uplift pressure caused by water seeping under the foundation. To effectively increase the resistance to overturning for a given height, the values for 'T' and 'H' must be adjusted to yield the desired stability while still satisfying soil pressure constraints. Soil pressure and the factors of safety against sliding and overturning are calculated in the same manner as described in the gravity wall discussion.

As mentioned above, the internal stresses in a gravity wall are low, due to the massive nature of the structure. Therefore, design for internal stress is not generally required for a gravity wall. The elements of a cantilever wall, however, are slender, and careful consideration of reinforcing and detailing is necessary. The wall stem section acts as a cantilever fixed at the base, and therefore, its depth is normally controlled by the bending force at this location and must be sized to safely carry all applied loads. Shear is another important consideration, particularly near the connection of the wall stem at its base, where a construction joint is usually located. Again, the wall stem must safely carry the full lateral load and be capable of safely transferring this load to the wall base.

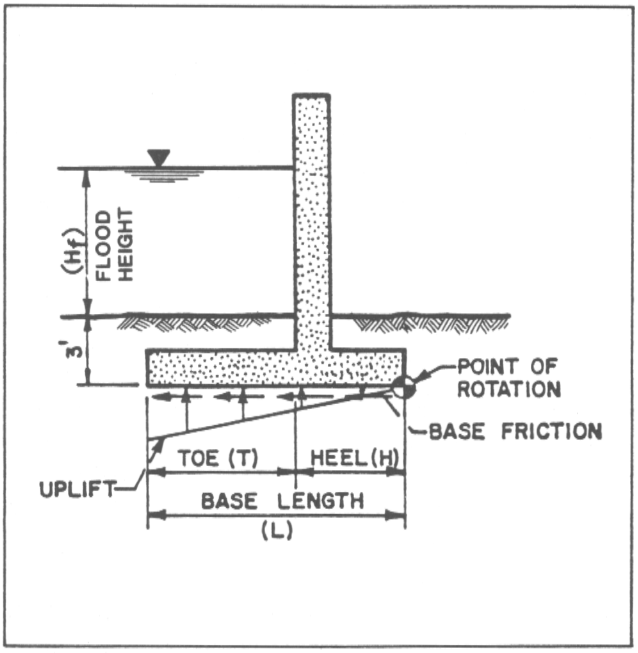


Figure III-50. Stability-Cantilever Floodwalls

To provide bending resistance against the applied loads, reinforcing bars must be placed toward the wetted face of the cantilevered stem. The shear is generally capable of being carried by the concrete cross section provided it has been properly proportioned according to ACI 318, Section II (American Concrete Institute, *Building Code Requirements for Reinforced Concrete).* When there is a construction joint between the base and the stem, as is usually the case to facilitate construction, a shear key or additional reinforcing bars must be provided to transfer the applied shear forces.

Prevention of water leakage through the hairline crack at the joint is generally provided by a waterstop. The arrangement of bars, etc., at the critical stem-to-base joint is shown in Figure III-51. Horizontal bars are designed for the wallstem section not to resist forces from floodwaters, but to control cracking of the concrete due to shrinkage, changes in volume due to temperature variations, and to provide integrity to the concrete.

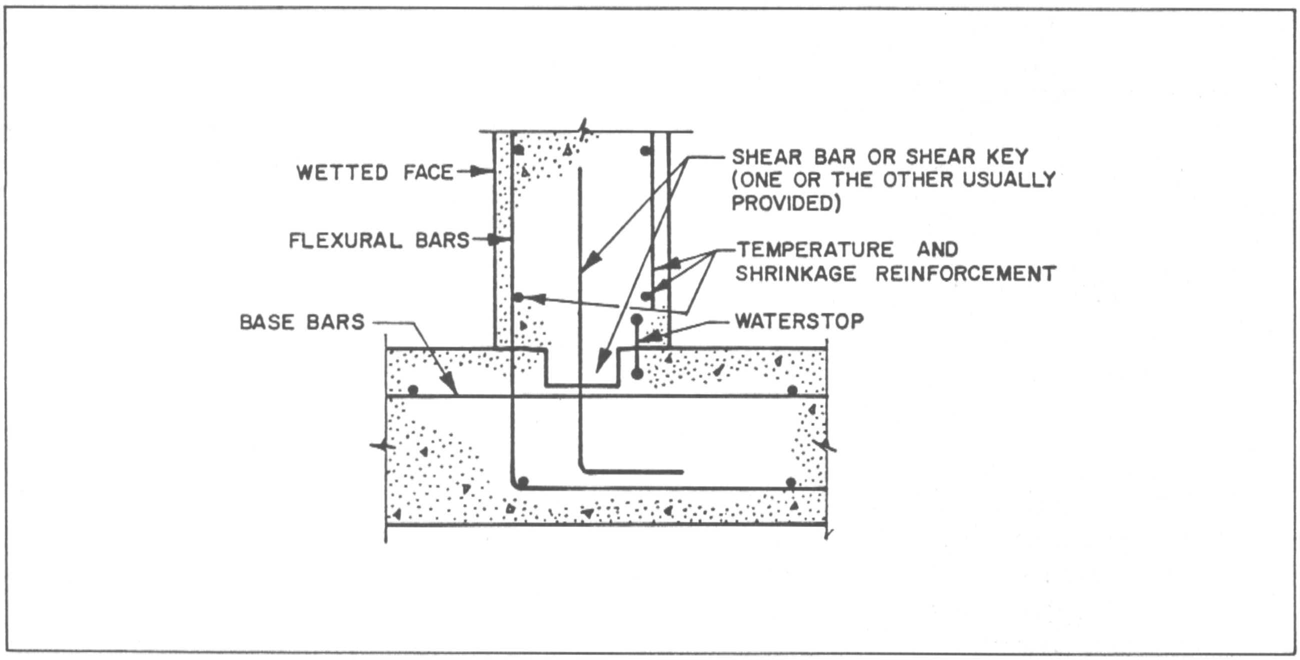


Figure III-51. Cantilever Wall Base

Shear and flexure in the concrete base are caused by the net effect of upward contact pressure and the downward weight of soil, water, and concrete. Although the relative effect of all forces varies from case to case, the usual situation is that concrete, submerged soil, and floodwaters tend to bend the toe downward while the contact pressure along the heel tends to bend this portion upward. Reinforcing steel must therefore be placed at both the top and bottom in the base. Shear forces are also considered and must be carried by a properly proportioned concrete section. In extreme cases, shear reinforcement can be provided in the base, but usually it is more cost-effective to simply thicken the base.

In summary, cantilever walls are commonly used to resist flooding and other lateral loads and their mechanism is well understood by engineers. Their use in resisting floodwater is almost always appropriate, particularly where a fairly high wall is required. In areas where foundation soil conditions are poor, the cantilever wall is a good choice because contact pressures are more readily controlled than with the use of a gravity wall. In very poor soil conditions, the base may be supported on drilled piers or piles to provide additional resistance against soil failure. The cantilever wall is a more complex structure to construct than the gravity wall, which could be an important consideration, especially in areas where the number of experienced contractors and craftsmen is limited.

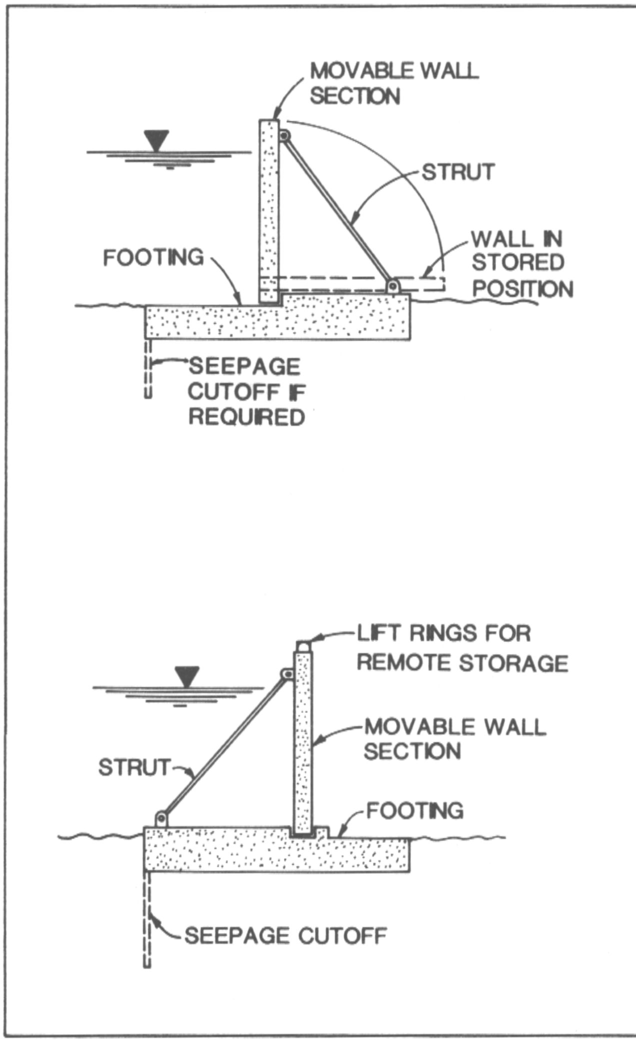


Figure III-52. Movable Floodwalls

d. Movable Walls. Figure III-52 illustrates two types of movable floodwalls. Wall sections may be either steel or concrete. The design of the footing requires the same considerations discussed above for permanent walls.

The movable wall is supported at both the top and the bottom. This is done to prevent rotation and allow easy assembly and removal. For walls of equal heights and loads, a simply supported removable wall panel will withstand a greater maximum shear and bending movement than will the fixed cantilever wall. Because of the support provided by the struts at the top, movable wall panels may be more lightly constructed.

Dry side strut construction is preferred because the struts are better protected and may be easily repaired or reinforced. If wet side struts are required, they should be of larger size to provide a safety margin against damage from floating debris.

6. LEVEE DESIGN. A levee is constructed of suitable fill material that is placed and compacted in layers to form a stable barrier to floodwater. Levees must be designed to have adequate strength and stability to resist all applied loads up to the designated protection level. For preliminary design purposes, the analysis of a levee may be divided into several critical components including (a) seepage and interior drainage control, (b) slope stability, (c) borrow area design, and (d) erosion protection. Seepage and interior drainage control have been covered in part 3 of this section. Therefore, the following presentation will be limited to items b, c, and d as listed above.

a) Slope Stability. Slope stability of an earth fill or levee embankment may be defined as the resistance of a given embankment to soil slippage or a tendency to move to a more stable (flatter) slope angle. Slope stability analysis techniques may be used to ensure that a given embankment will satisfy suitable safety factors. The 'safety factor' is generally defined as the ratio of all stabilizing (resisting) forces to the driving forces (the forces tending to cause movement). The slope on the verge of failure is considered to have a safety factor of 1.0. For normal loading cases, an acceptable safety factor would be between 1.3 and 1.5. For extreme loading cases, it may be as low as 1.1. The stability analysis should be performed for the worst case loading conditions that are expected to develop.

Two modes of shear failure must be investigated: the rotational slide (Figure III-53) approximated by circular arc, and the translatory slide (Figure III-54) that occurs along a definite plane of weakness near the base of the embankment.

Figure III-53 illustrates a cross section of a sliding soil mass along a curved surface (rotational failure surface). The sliding tendency is develped by the moment of the mass about the center of the arc as shown. This moment is opposed by the total shearing resistance developed along the assumed sliding surface. Of course, when all available resistance is overcome, a progressive failure occurs.

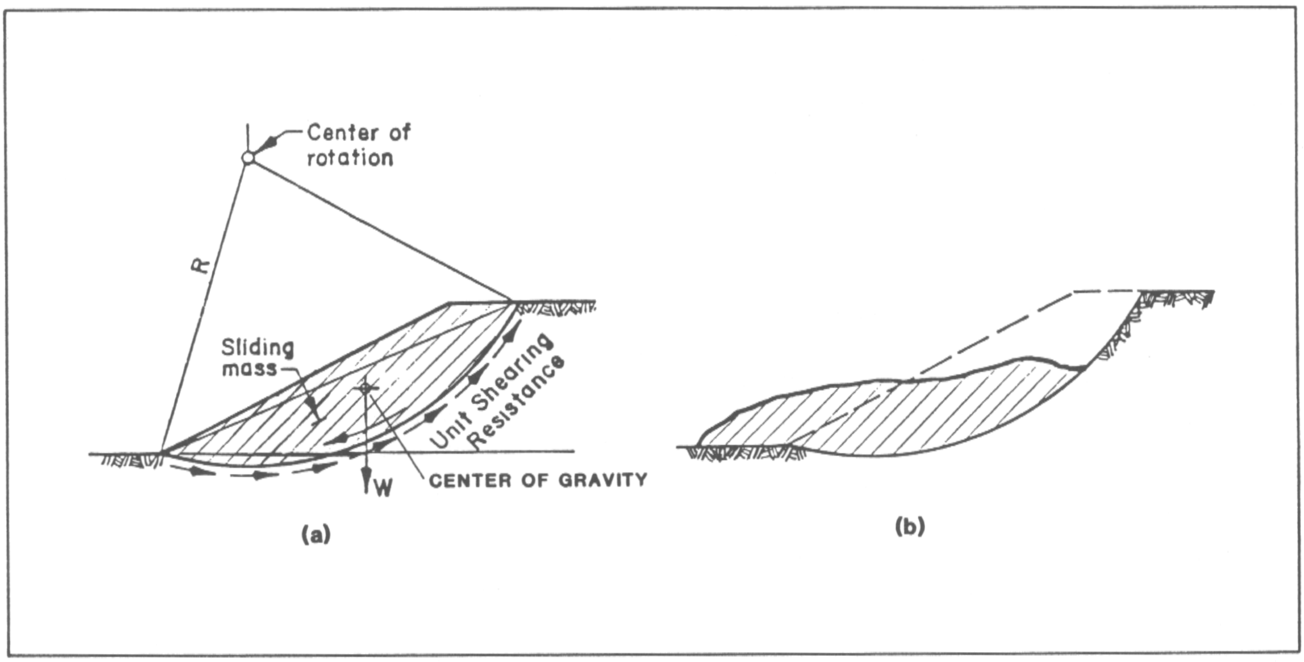


Figure III-53. Characteristics of a Rotational Slide
(Toe Slope Failure in Uniform Soil)

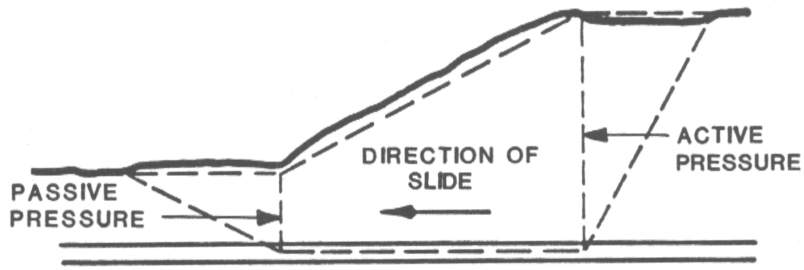


Figure III-54. Translatory Slide

Sophisticated numerical procedures, usually involving possible curved failure surfaces, have been proposed for evaluating the rotational slide. Due to the uncertainty in estimating the physical parameters of the material used in levee construction, however, use of these methods is rarely justified. The simple Swedish Slide Method or the Modified Swedish Method (method of slices), among others, provide acceptable analysis techniques. Most geotechnical engineering firms have access to computer programs that can quickly evaluate embankment stability if a detailed analysis is required.

For more detailed information relating to slope stability analysis and other embankment design considerations, the reader is encouraged to refer to *Design and Construction of Levees,* EM 1110-2-1913 as published by the U.S. Army Corps of Engineers. However, for preliminary design and cost estimating purposes a slope stability analysis is not generally required if standard slopes are maintained. The steepest slope that should be considered without detailed studies is a 1:2 slope ratio (1 vertical unit to 2 horizontal units). Where conventional mowing equipment is to be used to maintain the embankment, slopes should generally not exceed a 2:5 slope ratio.

Riverside slopes may be less steep than the ranges presented above if erosion damage from waves or high velocity floodwaters is anticipated.

b) Borrow Area. The selection of an appropriate borrow location often represents a critical factor in determining the applicability and economic feasibility of floodproofing with a levee. Factors that must be considered in the selection of a borrow area include the type and quantity of material available, distance from the levee site, land value, and environmental impacts. At sites where the borrow area is located in close proximity to the proposed levee, the designer must also evaluate any direct impacts that the borrow area may impose on the stability or impermeability of the levee. Because most soils are suitable for levee construction (except wet fine-grained or highly organic soils), accessibility and proximity usually represent the controlling factors in the selection of borrow areas.

Normally, long shallow borrow areas located some distance riverward of the proposed levee alignment present the optimum location for the borrow area (Figure III-55). However, landside pits are acceptable; and near urban areas, large centralized borrow areas are often established. It is generally

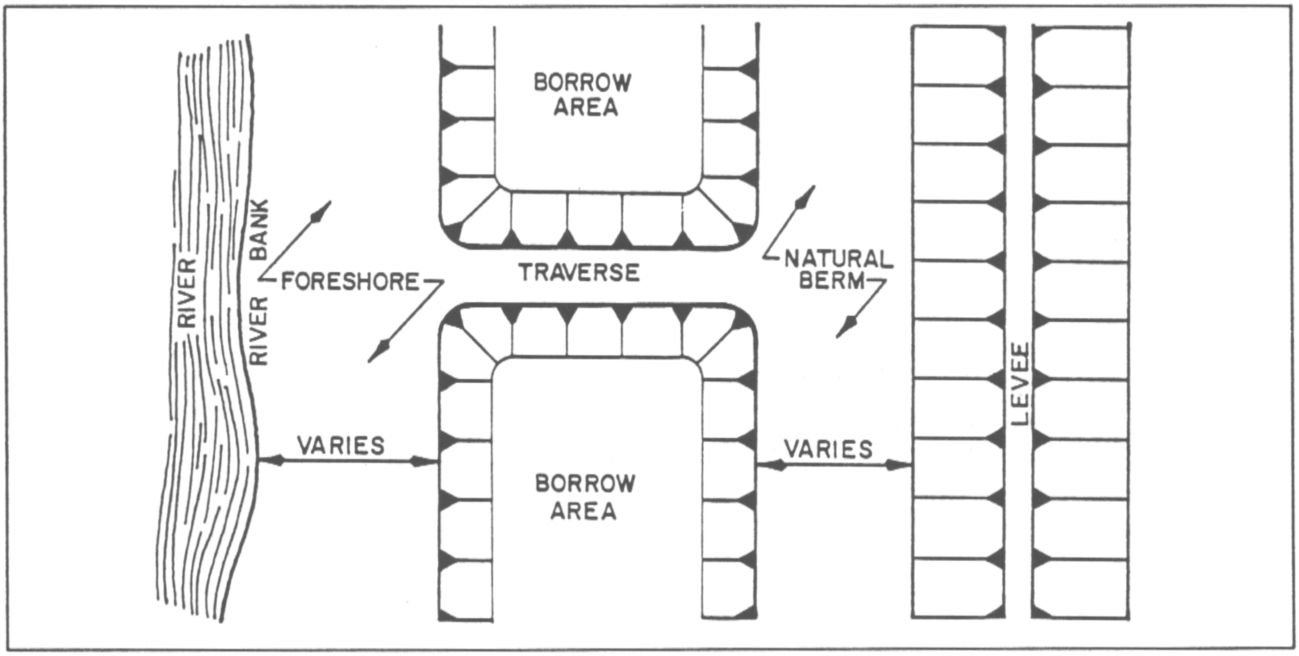


Figure III-55. Typical Levee and Borrow Areas

Source: U.S. Army Corps of Engineers, Design and Construction of Levees, EM 1110-1913

preferable to have 'wide and shallow' borrow pits as opposed to 'narrow and deep'. Side slopes should be relatively flat to avoid stability and erosion problems.

When using centralized borrow pits near the levee, an adequate thickness of impervious cover should be left over underlying pervious material. It is recommended that a minimum of two feet of impervious cover be left in place, and for landside pits the cover thickness should be adequate to prevent the formation of boils. The final borrow area should be graded for positive drainage and landscaped as required for aesthetic purposes and to protect against erosion.

c) Erosion Protection. Some form of erosion protection will be required on the riverside slope of an earth fill embankment or levee to withstand the scouring and impact forces of waves and stream currents. This protection can be provided by grass cover, gravel, asphalt paving, concrete mats, or riprap. Riprap is the most common protective cover when it is determined that vegetative cover will not be adequate. Factors that should be considered in selecting appropriate erosion protection material include:

- Velocity of Floodwaters. Riprap protection should be considered if stream velocities are expected to exceed 5 feet per second.
- Protective Barriers. An embankment may be protected from severe erosion by dense stands of vegetation or other features that reduce wave impact or stream velocity.
- Wind Velocity and Fetch. The severity of wave action is generally related to anticipated wind velocity and the length of the water body that is exposed to the wind (fetch). In general, riprap should be used if the 'fetch' is greater than 1,000 feet at the design flood level.
- Embankment Slope. The slope of the embankment has an influence on the susceptability of the structure to erosion. In general, flatter slopes are subject to less erosion damage than steep slopes.
- Levee Alignment and Materials. The characteristics of the embankment construction materials and the alignment of the embankment in relation to wave impacts, and moving floodwaters also have a significant effect on erosion potential.

For preliminary design and estimating purposes, a riprap layer that is 1 foot thick with a maximum stone size of 150 lbs. is considered to be adequate for most situations. The riprap should have a smooth size distribution with a median rock size of about 25 pounds (eight inch diameter), with 80% of the rocks larger than four inches in diameter and ranging down to gravels. With a distributed size range, the spaces formed by the larger stones are filled with smaller sizes which prevents the formation of open pockets. Angular stones are more suitable for riprap than rounded stones. The rock should be hard, dense, and durable to withstand long exposure to weathering. Rock should be dumped directly from trucks to minimize segregation of rock sizes. If further refinement is desired, the reader may refer to Figures III-56 to determine specific stone size requirements; and to Figure III-57 to determine the volume of riprap material that would be required for a particular embankment.

7. MAINTENANCE. Floodwalls and levees should be inspected annually for structural integrity. Following a flood, the structures should also be examined for scour and erosion damage. Depending upon the adequacy of the original levee protection, it may be necesary to replace riprap or increase the level of erosion protection. If excessive scour occurs, consideration should be given to landscaping features or the construction of flood flow diverters or barriers near the upstream side of the structure to reduce flood velocities and the associated impacts of scour and debris accumulation.

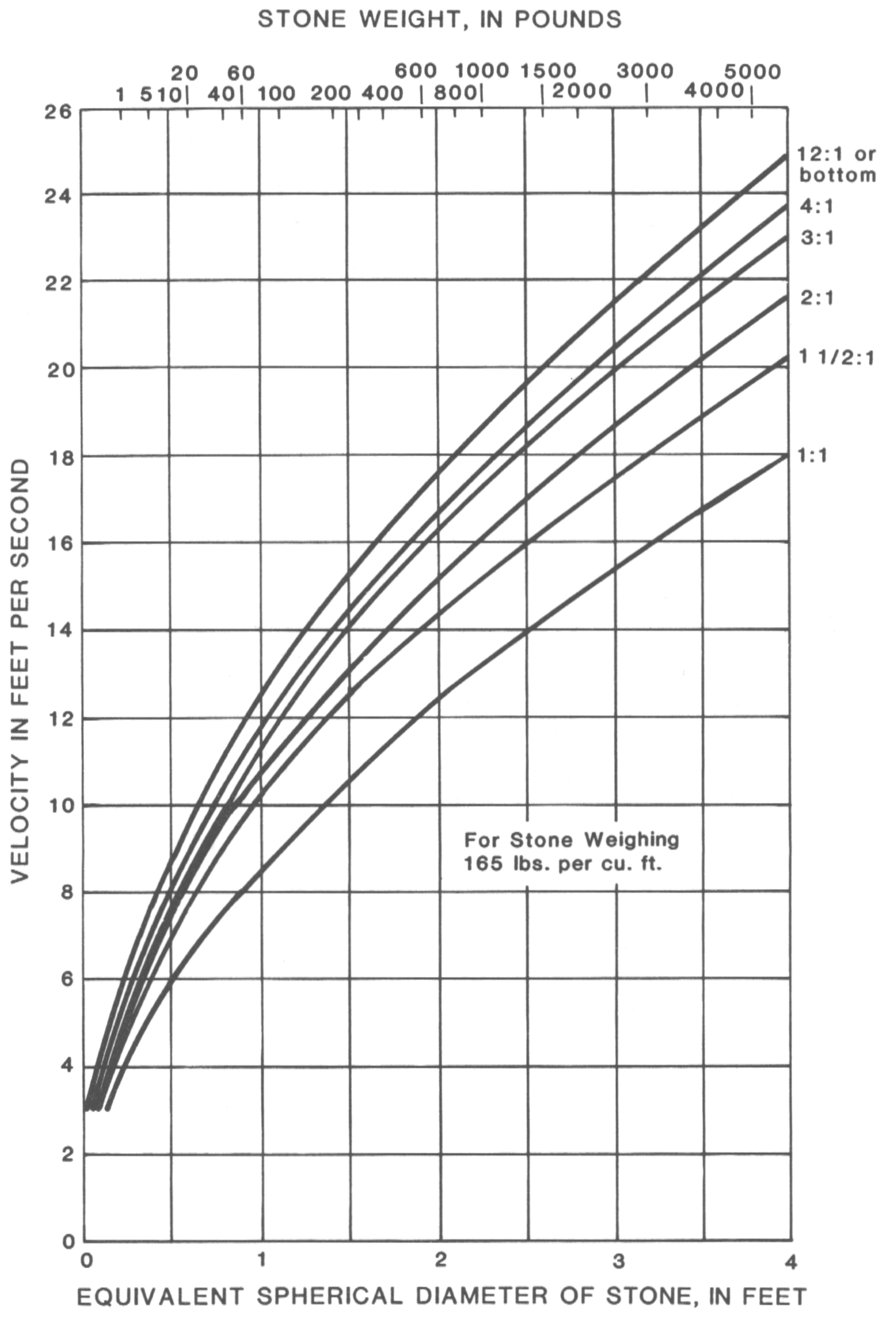


Figure III-56. Size of Stone That Will Resist Displacement for Various Velocities and Side Slopes

Source: Adapted from Subcommittee Report on Slope Protection, American Society of Civil Engineers Proceedings, June, 1948

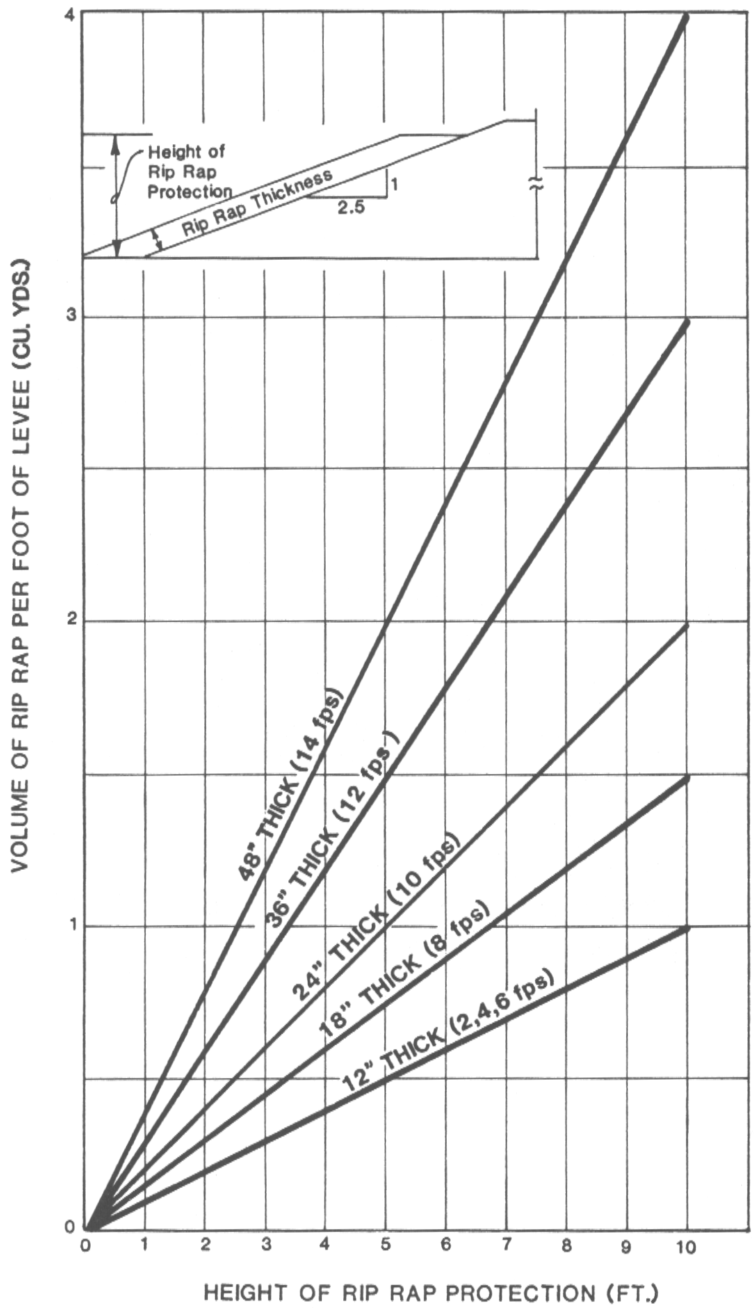


Figure III-57. Volume of Rip Rap Required Per Linear Foot of Embankment for Various Embankment Heights and Stream Velocities