Section 11
Substructures

11.1 Foundations

11.1.1 General

There are two basic types of substructure foundations, shallow and deep. Shallow foundations typically utilize spread footings to transfer structure loads to the soil at a relatively shallow level below the ground surface. Deep foundations utilize some type of driven pile, drilled shaft or caisson to transfer the structure load to some lower stratum of soil or rock.

The choice of whether to use a shallow or deep foundation depends on soil conditions and the potential for scour at the site. Foundation recommendations are made to the bridge designer in the Foundation Design Report (FDR).

11.1.2 Spread Footings on Soil

Spread footings transfer the load directly from the bridge substructure to the subsurface. In the case of abutments, the footings must withstand dead and live loads associated with the bridge, horizontal and overturning forces from the retained earth, construction loads and any live load surcharges that might occur.

Spread footings need to be designed to keep the bearing pressures and sliding forces within the allowable soil parameters. At the same time, they must be designed structurally to resist bending moments and shear forces.

Abutment and retaining wall spread footings that are founded on soil should be designed according to Section 10 of the NYSDOT LRFD Bridge Design Specifications. Footing design needs to be optimized to minimize toe and heel projections of the footing. Designers are cautioned that some computer programs do not automatically produce an optimum or economical footing design and, therefore, it is necessary to vary toe and heel projections until an economical design is achieved.

11.1.3 Spread Footings on Rock

Rock lines should be shown on the plans only when the footings are on rock or when drilled shafts or caissons are to be placed to rock. When rock lines are shown on the plans, they shall be marked as “Assumed Rock Surface.” The elevations of the rock are not to be labeled.

When it is planned to place footings on or key footings into rock, the plans shall show the top of footing elevation and the minimum depth of footing. This will enable adjustments to be made in the depth of footing if the actual rock elevation differs from that assumed during design, while keeping the top of the footing elevation constant.
Rock removal shall be avoided whenever possible in the construction of footings. Footings shall not be detailed with keys or dowels into rock unless dictated by design requirements or other special circumstances. This will be noted in the FDR.

When a footing must be keyed into rock, usually the entire footing is keyed into rock to simplify construction.

When a footing is doweled into rock, the dowels shall be #9 reinforcing bars or larger and shall be embedded into the footing as well as into the rock to a depth noted in the FDR. The designer shall determine the required spacing between the rows of dowels, but in no case shall there be greater than 3 ft. between rows or less than two rows.

Doweling is generally preferred to keying except where the rock is shale or is scourable. The recommendation of whether to key or dowel is contained in the FDR.

11.1.4 Pile Foundations

11.1.4.1 Pile Types

Pile foundations are used when it is necessary to carry the structure load through a zone of weak or compressible material to firmer foundation material at a deeper level. Piles are also used to found a structure below the depth of potential scour. End bearing piles develop their load capacity through their tip by bearing on hard material. Friction piles develop their load capacity by skin friction between the pile and soil over their length.

Most piles used by NYSDOT are either steel H-piles or C.I.P. concrete piles. Other types of piles, such as prestressed concrete, timber, or micropiles, have also been used. Prestressed concrete piles are typically used in marine environments. Micropiles are used in areas where vibrations from pile driving are unacceptable, at pile installation locations where there is limited headroom or obstructions are present, and for retrofitting existing substructures.

Most piles function as a combination of friction and end bearing. Steel H-piles are the better choice where it is anticipated there will be hard driving conditions.

C.I.P. piles utilize a driven steel pipe that is later filled with concrete. Steel reinforcement shall be placed in the top \( \frac{1}{3} \) of the pile or 10 ft. whichever is greater. C.I.P. piles are usually the choice when friction capacity is important.

Steel H-piles and C.I.P. piles shall have their tips reinforced to protect them during driving.

See the *Highway Design Manual*, Chapter 9, for a more detailed discussion of Deep Foundation Types.

11.1.4.2 Pile Spacing and Placement Details

Timber piles shall be spaced not less than 2’-6” center-to-center. All other types of piles shall be spaced a minimum of three pile diameters or three pile widths center-to-center. Maximum pile spacing shall be 9 ft.
The minimum distance from the center of a pile to the nearest footing edge should be 1'-6". The minimum distance from the edge of a pile to the nearest footing edge shall be 9". The minimum distance from the center of a pile to the nearest edge of the capbeam shall be 1'-6". The minimum distance from the edge of a pile to the nearest edge of the capbeam shall be 1'-0".

The tops of C.I.P. piles shall be embedded 6" into the footing. The tops of all other piles shall be embedded 1'-0" into the footing. Additional embedment requirements apply to integral abutments (see Section 11.6.1.6).

When a pier is composed of steel H-piles extending above the ground surface and embedded directly into a concrete capbeam, the piles shall be embedded a minimum of 2'-0" into the capbeam. The same embedment applies to C.I.P. piles unless the pile reinforcement projects into the cap. In this case, the embedment shall be 1'-0".

The front row of piles (at the toe) of all abutment and wall footings shall be battered. The outside rows of piles of all pier footings shall be battered. Minimum batter is 6 on 1; however, analysis may indicate that a greater batter is required. The maximum batter shall be 3 on 1. If a critical clearance problem exists (e.g., underground utilities), it may be possible to place some piles vertical that would ordinarily be battered. In this case, the Office of Structures' Foundations and Construction Unit should be consulted. Horizontal forces must be resisted through a combination of the horizontal component of a battered pile and the lateral resistance of the soil to the pile. Lateral resistance of piles is specified in the Foundation Section of the latest NYSDOT LRFD Bridge Design Specifications unless modified in the FDR.

### 11.1.4.3 Numbering and Tabulation of Piles

All piles used in a structure shall be numbered on the plans. The pile numbering shall begin with the number one (1) and proceed continuously through all piles in that substructure unit. The pile numbers shall restart with the number one (1) for each different substructure unit encountered.

In order to record the actual driven length, a table shall be placed on the plans for each different substructure unit. The table shall include a column titled “PILE NO.” and a column titled “LENGTH BELOW CUT-OFF.” The length below cut-off of each pile shall be filled in by the E.I.C.

### 11.1.4.4 Pile Splices

Piles requiring splices shall be spliced by using either complete penetration groove welds or mechanical splices.

Complete penetration groove welds are allowed on splices for all conditions.

Mechanical splices are not allowed in the following situations:

- Any type of pile subject to uplift loads.
- CIP piles in integral abutments due to bending.
- CIP piles in any substructure subject to scour, when the depth of scour from either the Q100 or Q500 flood is below the bottom of footing.

The FDR will provide notes to be included in the contract plans when Mechanical splices are not allowed. When mechanical splices are allowed on CIP piles, a seal weld will still be required.

When the estimated length of pile exceeds 30 ft., the designer's estimate shall allow for at least one-half the total number of piles to be spliced. This is a contingency to cover the situation where the actual length of driven pile exceeds the estimated length by more than 10 ft.

Details of pile splices and reinforced tips are shown on the current BD sheets. These details shall be included in the contract plans.

11.1.5 Drilled Shafts

Drilled shafts are typically used as an alternative to piles. They are capable of carrying very large loads. Drilled shafts are usually advanced with a steel casing, although a slurry solution is sometimes used to keep the excavation open. The FDR may require that the shaft be socketed a minimum distance into bedrock to develop the necessary skin friction to support the applied loads. When the shaft is excavated, reinforcement is placed and the shaft is filled with concrete.

11.1.6 Pilasters

Pilasters are typically square concrete columns that are used when rock is located too near the surface to drive piles. They are capable of handling very large loads. Pilasters are usually constructed in an open excavation down to bedrock and may be socketed into bedrock a minimum distance.

11.1.7 Design Footing Pressures and Pile Capacities

Notes that specify either the maximum foundation pressure for spread footings or the maximum allowable pile load and ultimate pile capacity for pile foundations shall be shown on the contract plans. The wording and format of these notes are given in the FDR. For spread footings on rock, the actual design bearing pressure shown on the plans should be rounded to the nearest one-half ton/ft².

11.1.8 Footing Depth

The depth to which footings are carried below the ground surface is usually determined by three factors: frost depth, scour action, and foundation type.

Frost heaves in soil can cause displacement of the footing and damage to the structure. Spread footings founded on soil shall have their bottom of footing a minimum of 4 ft. below finished ground to assure that the bottom of the footing is below the maximum frost penetration. Spread footings on rock are not susceptible to frost heaves and, therefore, do not require the minimum 4 ft. depth. Spread footings on soil are not ordinarily used near water because of their
vulnerability to scour action. If they are used near water, however, their bottom of footing needs to be well below any potential scour depth and special protective measures may be needed.

Although footings supported on piles or drilled shafts are not normally susceptible to frost action, they are often subject to erosion or scour action. Footings on piles, drilled shafts, or pilasters should be set at least 4 ft. below the (stream bed, river bed, lake bottom, etc.) or finished slope. The top of the footing should be at least 1 ft. below the finished ground surface, therefore, thicker footings may require more than a 4 ft. depth.

If a stone apron is to be used for bank protection, sufficient room must be left to place the stone over the top of the footing.

11.1.9 Stepped Footings

Stepped footings introduce construction difficulties and, in the case of spread footings on soil, an increased risk of differential settlement. They are, therefore, very seldom used. The use of stepped footings may be warranted in some cases, such as a variable rock elevation or a long wall where the required bottom of footing elevation changes considerably.

The most common reason for stepping footings is to accommodate spread footings on a sloping rock surface. Stepped footings on rock shall have steps at least 8 ft. in length and at least a 2 ft. change in height. Footing continuity is not required.

Stepping spread footings on soil or pile foundations should only be done under wingwalls and retaining walls longer than 25 ft. The minimum length of each step section should be 12 ft and the change in height of each step should be at least 2 ft. Footing continuity is preferred for all steps up to 3 ft., but is not mandatory. Steps more than 3 ft. will require a construction or contraction joint to facilitate construction. Any joint introduced shall be continued up through the stem or walls above.

Stepping of the leveling pad for a Mechanically Stabilized Earth System (M.S.E.S.) on embankments is permitted. The minimum length of a step section is the width of one panel. The minimum height of a step for this type of wall system is one half the panel height. The manufacturer of the mechanically stabilized earth system shall set the final configuration of the leveling pad as part of the panel layout.

Any proposed steps in footings should be included in the Preliminary Structure Plan and approved by the Office of Technical Services’ Geotechnical Engineering Bureau.

11.1.10 Tremie Seals

A tremie seal is concrete placed under water through the use of a tremie placement tube. As the concrete is placed, water is displaced and the tube is gradually raised keeping the outlet below the level of the placed concrete. Tremie seals are usually used where piers need to be constructed in fairly deep water and it is difficult to dewater the excavation. A sheet piling cofferdam is usually placed to surround the excavation. Piles, if required, are driven inside the cell with water still inside. The tremie seal is then placed to a level where its submerged weight will exceed the hydrostatic pressure of the dewatered excavation. The water can then be
pumped out of the excavation and the footing constructed on top of the tremie seal in the dry. The piles should be sufficiently long to project above the tremie seal and into the footing. Tremie concrete uses Class G concrete which has a higher cement content and slump range than Class A concrete.

In the design of a tremie seal, the designer must remember to use the buoyant weight of the concrete in balancing the hydrostatic pressure. In calculation, the dry weight of the tremie seal should be conservatively taken as 140 lb/ft$^3$. Tremie seals are normally designed to resist the hydrostatic pressure at ordinary high water. The excavation should be designed to flood when the water level exceeds ordinary high water to prevent unequal hydrostatic pressure from “floating out” the tremie seal during construction. A minimum safety factor of 1.25 is recommended in tremie design. See Section 4 of this manual for further information on cofferdams and tremie seals.

11.1.11 Footing Thickness

The minimum footing thickness for spread footings shall be 2 ft. The minimum footing thickness for pile supported footings shall be 2'-2” for C.I.P. piles and 2'-6” for steel H-piles.

11.2 Forming Considerations

In heavily reinforced concrete structures, the labor and material costs for formwork often average between 30% to 50% of the total in-place cost of the structure. Within that total formwork cost, the labor cost to build and place the forms is generally two to three times the formwork material cost. In other words, an efficient structure is one that not only conserves cubic yards of concrete, but also reduces the labor involved in formwork. The shape should be such that large flat forms and large placements may be employed.

Simplicity and repetition are the keys to achieving economy in forming. Configurations that lend themselves to commercially built forming systems will generally be less expensive than those requiring custom built forms. If special forms are required, the high initial cost of those forms may be offset if those forms can be used several times.

Generally, in normal column construction the circular shape is the most economical to form because commercially prefabricated forms are available in many standard diameters. These forms are easy to set up, strip and require no form ties.

Battered forms are more expensive than vertical forms and should be avoided whenever possible, especially on short wingwalls. If a thicker wall section is required at the base of a wall, the designer should consider using the thicker section for the full height of the wall or to a construction joint and stepping the thickness. If battered forms are used, the batter should remain constant. Battering only one side is the least expensive battering system. Battering on three or four sides always requires special forming and should only be considered when the hydraulic flow characteristics require special pier geometry.
11.3 Substructure Joints

At locations where a waterstop is to be installed, the walls shall be laid out such that the rear faces of the two adjoining walls are flush at the joint in order to accommodate the waterstop. All joints required in conformance with this section shall be shown on the contract plans.

11.3.1 Contraction Joints

Contraction joints are defined as interruptions provided in the concrete placement to control the location of cracks forming in the concrete due to shrinkage of the concrete during curing. Reinforcement shall not extend through a contraction joint. All contraction joints shall be provided with a shear key and a Type "D" PVC waterstop, except where leakage through the joint is unlikely or where staining due to leakage would not be objectionable.

Vertical contraction joints are required at 30 ft. maximum spacing in all retaining walls and wingwalls greater than 60 ft. long. In this case, contraction joints shall not extend through the footing.

11.3.2 Construction Joints

Construction joints are defined as interruptions in the concrete placement provided to facilitate construction. Vertical construction joints are sometimes detailed in abutment stems and backwalls to control the location of cracks forming due to shrinkage of the concrete during curing, thus performing a function similar to a contraction joint.

Reinforcement shall always extend through construction joints. All construction joints shall be provided with shear keys, unless otherwise specified, and sealed with Type "D" PVC waterstops, except where leakage through the joint is unlikely or where staining due to leakage would not be objectionable.

Vertical construction joints are required in long abutment stems and backwalls. The recommended maximum spacing for construction joints in abutments is 30 ft. When an abutment reaches 60 ft. in length construction joints should be considered. Placement of construction joints should be midway between pedestals at a fairly uniform spacing. Construction joints shall be placed between the abutment and flared wingwalls longer than 6 ft. Flared wingwalls less than 6 ft. long, shall not have a joint.

Construction joints should not extend through the footing. Special attention is required for construction joints in stepped footings, see Section 11.1.9.
11.3.3 Expansion Joints

Expansion joints shall be defined as interruptions in the concrete placement provided to allow for movements of the wall and footing due to thermal expansion.

Reinforcement shall never extend through an expansion joint. Expansion joints in walls shall be provided with a shear key, a Type "E" PVC waterstop, and a layer of joint material separating the concrete surfaces. The requirements for expansion joints in footings shall be the same except that the waterstops will not be required.

The maximum interval for expansion joints shall be 90 ft. in all retaining walls and wingwalls. When an abutment reaches 165 ft. in length expansion joints should be considered. Expansion joints shall extend through the footing.

The footings on each side of an expansion joint shall be independently designed. For footings on piles, the pile spacing and edge distance, including distance to the joint, shall meet all current pile layout and design requirements.

11.4 Concrete for Substructures

Class HP concrete was developed to provide increased durability by limiting the absorption of salt-laden water into the concrete. Therefore, any substructure that might be exposed to chlorides through splashing, runoff, or leaking through joints, should be designated as Class HP concrete on the plans. Class A concrete should be limited to placements that will not be exposed to chlorides. Also, it is inefficient to use Class A concrete for small placements when Class HP concrete is the concrete primarily used on the job. If only a small amount of concrete might be designated Class A, then make all of the concrete Class HP.

11.5 Retaining Walls

A retaining wall is a structure that provides lateral support for a mass of soil. A properly designed retaining wall ensures the structure will not fail by overturning, sliding, excessive settlement, excessive bearing pressures or pile capacities and the structure itself possesses adequate strength to resist the applied earth and live loadings and surcharges.

A retaining wall adjacent or abutting a bridge abutment is commonly referred to in bridge plans as a wingwall. The Highway Design Manual (Chapter 9) refers to cantilever walls supporting a highway embankment as a retaining wall.
11.5.1 Retaining Wall Types

Fig. 11.1
Typical Retaining Wall Types
11.5.1.1 Cantilevered Retaining Wall

Cantilevered retaining walls consist of a spread or pile supported footing that supports a relatively thin concrete stem that is structurally reinforced vertically along the back face. This is the most commonly used type of retaining wall for new bridges in NYSDOT. Cantilevered retaining walls remain stable due to their own weight and the weight of the soil located over the heel of the footing. The efficient height range of walls of this type is 5 ft.-30 ft.

11.5.1.2 Counterfort Retaining Wall

Counterfort retaining walls consist of a relatively thin vertical concrete face wall supported at intervals on the earth side by vertical walls (counterforts) that are perpendicular to the face wall. Both the face wall and the counterforts are connected to a footing slab that can be either spread or pile supported. The base is backfilled with soil between the counterforts. The vertical face wall is structurally reinforced horizontally to span between the counterforts. The counterforts are structurally reinforced to resist the tensile forces along their back. This type of wall is only used where economically justified, such as when the height of the wall is in excess of 30 ft., since forming is relatively expensive. The efficient height range of walls of this type is 30 ft.-60 ft.

11.5.1.3 Buttressed Retaining Wall

This is very similar to a counterfort retaining wall, except the counterforts are placed on the exposed front face of the wall due to limited construction access to the rear of the wall.

11.5.1.4 Crib Wall

Crib walls consist of individual structural units that are assembled at the site into a series of hollow cells called a crib. The cribs are backfilled with soil and/or rock and their stability depends on both the weight and the strength of the fill material to hold the units together. The structural units are usually constructed of precast reinforced concrete, although units with fabricated metal members and units with timber members have been used. Care should be taken to select the best structural unit type for the site conditions and desired service life. Crib walls are relatively inexpensive. For guidance in selecting the proper type of crib wall, see Highway Standard Sheets 632-1,-2,-3.

11.5.1.5 Gabions

A gabion is a special type of gravity wall. Gabions use wire-mesh baskets as the crib and are filled with rocks to provide the necessary weight and stability. The wire mesh can be vinyl coated or galvanized. These units are usually stacked on top of each other to create the retaining wall. Gabions are susceptible to damage from debris or ice flows in high water conditions and to corrosion of the wire mesh. The efficient height range of walls of this type is 5 ft.-25 ft.
11.5.1.6 Gravity Retaining Wall

Gravity retaining walls are large masses of concrete or masonry that have nominal to no structural reinforcement in the back face of the stem. This type of retaining wall depends on its own large self weight to provide the lateral support and resist overturning forces. A large plan area at the base provides bearing on the soil or it may be pile supported. This type of retaining wall is usually no longer used for new bridges by NYSDOT.

11.5.1.7 Semigravity Retaining Wall

A semigravity retaining wall resembles a gravity retaining wall except the stem is somewhat thinner and vertical structural reinforcement is provided in the back face of the stem. The foundation uses either spread or pile supported footings. This type of retaining wall is no longer used for new bridges by NYSDOT.

11.5.1.8 Internally Stabilized Fill Type Retaining Walls

Mechanically Stabilized Earth System (M.S.E.S.) retaining walls consist of interlocking concrete shapes that create a wall face. Each of the concrete shapes has a soil anchoring system that mechanically reinforces the retained embankment and uses the weight of the fill as the stabilizing force to hold the panels in place. The efficient height range of walls of this type is 10 ft.-65 ft.

Geosynthetic Reinforced Soil Structures (G.R.S.S.) are similar to M.S.E.S walls, in that layers of geotextile membrane are covered with soil. In place of the interlocking concrete shapes in M.S.E.S. walls, the exposed face of the embankment is formed by folding the lower layer of reinforcing geo-grid over the top of the layer of fill that covers it. Subsequent layers of geo-grid and soil are placed and compacted until the desired height of the embankment is reached. The efficient height range of walls of this type is 5 ft.-35 ft. See Section 11.6.1.4 for more information on M.S.E.S. systems.

Mechanically Stabilized Wall Systems are a combination of M.S.E.S. and G.R.S.S. In this case, layers of geotextile membrane are covered with soil and anchored between prefabricated modular blocks that make up the exposed face of the embankment. Subsequent layers of block and reinforcing geo-grid are placed and compacted until the desired height of the embankment is reached. The efficient height range of walls of this type is 5 ft.-35 ft.

11.5.1.9 Cantilevered Sheet Pile Retaining Wall

Cantilevered sheet pile walls consist of a series of interlocking structural shapes that are set into the ground to a sufficient depth to mobilize enough passive earth pressure to withstand the active pressure from the retained soil. The structural shapes are most commonly made of steel and driven into the ground. Concrete shapes have also been used and jetted in place. Cantilevered sheet pile retaining walls are commonly used by NYSDOT for both temporary and permanent conditions. The efficient height range of walls of this type is 5 ft.-15 ft.
11.5.1.10 Tied Back Sheet Pile Retaining Wall

Tied back sheet pile walls consist of a series of interlocking structural steel shapes that are driven into the ground to a sufficient depth so as to mobilize enough passive earth pressure to withstand the active pressure from the retained soil at the bottom, and utilize a tie-back or anchored bulkhead system to support the top of the sheet piling. This type of retaining wall is commonly used by NYSDOT for both temporary and permanent conditions. The efficient height range of walls of this type is 15 ft.-65 ft.

11.5.1.11 Soldier Pile and Lagging Retaining Wall

This retaining wall consists of two main structural parts, the piles and the lagging. The piles are driven into the ground or set into augured holes at regular spacings and to a sufficient depth so as to mobilize enough passive earth pressure to withstand the lateral load from the retained fill. That lateral backfill load is transferred to the piles through the lagging which spans horizontally between the piles and behaves like a simple beam between two supports. The piles are commonly steel H-piles and the lagging could be heavy wood timbers or precast concrete panels. This type of retaining wall is commonly used by NYSDOT for both temporary and permanent conditions. The efficient height range of walls of this type is 5 ft.-15 ft.

11.5.1.12 Tied Back Soldier Pile and Lagging Retaining Wall

Similar to a normal soldier pile and lagging wall with the addition of a tie back system. The piles are driven into the ground or set into augured holes at regular spacings and to a sufficient depth so as to mobilize enough passive earth pressure to withstand the lateral load from the retained fill at the base of the excavation. The tie back system supports the top of the retaining wall. This type of retaining wall is commonly used by NYSDOT for both temporary and permanent conditions. The efficient height range of walls of this type is 15 ft.-65 ft.

11.5.2 Proportioning of Cantilevered Retaining Walls

Since the cantilevered retaining wall is by far the most common type of retaining wall used, it is important to achieve as much efficiency in its design as possible. In general, the width (B) of the footing should range from 0.40 to 0.60 times the height (H) of the wall above the top of the footing. The B/H ratio is closer to 0.40 when the bearing soil is firm or when the footing is on piles. The B/H ratio increases as the quality of the bearing soil and coefficient of friction decreases, and the slope of the fill and any other surcharge behind the wall increases. The distance from the centerline of the wall stem to the front edge of the footing (D) should be approximately 0.30 to 0.50 times the width of the footing. The footing thickness (T) is generally between 0.10 and 0.15 times the height of the stem but should always meet the minimum footing thickness requirement for the type of foundation selected. The stem thickness (t) should be at least 0.10 times the height for an economically reinforced section.
Further information on retaining wall design is found in Chapter 9 of the *Highway Design Manual*.

### 11.5.3 Wingwall Type and Considerations

Wingwalls are simply retaining walls placed adjacent to the abutment stem to retain the fill behind the abutments. The orientation of the wall in relation to the centerline of bearings or centerline of the roadway determines the wingwall type.

When the wingwalls are parallel to the roadway, they are called **U-wingwalls**. U-wingwalls are used primarily in fill situations where there are obstructions or limited right of way on either side of the roadway to build a wide embankment. The length of the U-wingwall is determined by equating the point where the embankment slope meets the shoulder break elevation from the roadway. The intersection shall occur at the inside corner of the top of the wingwall. The elevation of the end of the U-wingwall shall be at this intersection and stated on the plans.
When the wingwalls are parallel to the centerline of bearings, they are called in-line wingwalls. These wingwalls are used when the abutment is relatively short and there are no obstructions or right of way limitations on either side of the highway. The end of an in-line wingwall is located where the slope from the shoulder break meets the underbridge embankment slope. The intersection shall occur at the rear corner of the wingwall. The elevation of the top of the wingwall shall be 8” higher than this intersection and stated on the plans.

When the wingwalls are turned back toward the retained fill but not parallel to the roadway, they are called flared wingwalls. These wingwalls are used when the abutment fill would spill out too far for in-line wingwalls, but there are not enough restrictions to justify U-wingwalls. The end of a flared wingwall is located where the shoulder break from the roadway meets the underbridge embankment slope. The intersection shall occur at the rear corner of the wingwall. The elevation of the top of the wingwall shall be 8” higher than this intersection and stated on the plans.

Curved wingwalls should be avoided whenever possible. If it is absolutely necessary to provide a curved wingwall, it is best to place a widened footing on a chord and only curve the top portion of the wall. Curved wingwalls should never be battered since the forming is extremely difficult.

Wingwall foundations shall match the abutment foundation requirements (e.g., a pile supported abutment will always have pile supported wingwalls) except for integral abutments.

11.6 Abutments

Abutments serve two principal functions. They vertically support the bridge superstructure and horizontally support the retained earth of the roadway approach immediately adjacent to the bridge. Therefore, a bridge abutment combines the functions of a pier and a retaining wall.
11.6.1 Abutment Types and Considerations

Figure 11.3
Typical Abutment Types
(a) Cantilevered  (b) Isolated Pedestal Stub  (c) Spill Through  (d) Gravity
11.6.1.1 Cantilevered Abutment

Cantilevered abutments consist of a central stem supporting the bridge seat and pedestals. A backwall on top of the stem and wingwalls on either side of the stem retains the fill. The stem and wingwalls rest upon a continuous footing that can be either soil or pile supported.

The structural reinforcing steel in a cantilevered abutment is designed to withstand the overturning forces that cause tension in the back of the stem and backwall. Also, design footing reinforcement is required and depends on the type of foundation selected. The large thickness of the abutment stem and backwall prevent horizontal bending from being a major concern in their design.

Cantilevered abutments have no limit on the skew angle, however, bridges with less skew perform significantly better than highly skewed bridges.

The superstructure length used with cantilevered abutments is not limited. The abutment shall be designed to support the applied superstructure loads. The thermal expansion of the superstructure shall be accounted for by the use of an expansion joint or appropriate jointless detail.

There are three different forms of the cantilevered abutment. When the abutment is placed so that the abutment has as little reveal above the ground surface as allowed, it is called a stub cantilevered abutment. When the abutment has the largest possible reveal with respect to the clearances required for the feature crossed, it is called a cantilevered high abutment. An abutment that falls in between these two extremes is called a cantilevered semihigh abutment.

11.6.1.2 Isolated Pedestal Stub Abutment

Isolated pedestal stub abutments have tall pedestals that rest directly on the footing and have no bridge seat. They have a backwall between the pedestals and wingwalls on each side to retain the fill. The footing may be either soil or pile supported.

The structural reinforcing steel in a stub abutment is designed to withstand forces that cause tension in the front of the backwall as it spans between the tall pedestals, and to withstand the forces that cause tension in the back of the backwall as it cantilevers above the footing. Also, design footing reinforcement is required and depends on the type of foundation selected.

Isolated pedestal stub abutments are no longer used for new structures by NYSDOT, however, existing isolated pedestal stub abutments may be encountered on bridge rehabilitation projects.

11.6.1.3 Spill Through Abutment

The spill through, or open, abutment consists of two or more vertical columns carrying a beam that supports the bridge seat and pedestals. The fill extends on its natural slope from the bottom of the beam through the openings in the columns. In an extreme form, the spill through abutment is no more than a row of alternating vertical and battered piles driven through the fill and supporting a bridge seat and pedestals. The stem is usually provided with small wingwalls.
to keep the bridge seat free of soil. Although no longer commonly used, spill through abutments may be encountered on rehabilitation projects.

### 11.6.1.4 Mechanically Stabilized Earth Systems (M.S.E.S.) Abutments

M.S.E.S. abutments consist of a mechanically stabilized earth wall embankment supporting a short or stub abutment on top of the retained soil. Further information on M.S.E Systems is contained in Section 11.5.1.8. M.S.E. Systems are to be installed and paid for according to the provisions of the Fill Type Retaining Wall items. M.S.E. Systems are the only kind of Fill Type Retaining Wall currently approved to support an abutment as shown in the BD sheets. Concerns about the response of this system to a seismic event have been satisfied by additional experience and AASHTO design specifications. Designers may consider the use of this system where site conditions are appropriate.

**Guidelines for Use:**

This type of abutment system is most efficient when the height of the wall supporting the bridge abutment is 15 ft. or greater. When the use of this system includes wingwalls and/or retaining walls the average height of the entire system should be 10 ft. or greater.

- The project site should be predominately a fill area. If extensive excavation is required, this type of system would be inappropriate.
- Utilities of any nature shall not be placed within or underneath the reinforced zone.
- If the project site involves a railroad, the railroad must approve the use of this type of system. A copy of the railroad’s acceptance letter of this type of construction should accompany the Structure Justification Report submitted to the Office of Structures.
- In waterway areas where the anticipated depth of scour falls below the concrete leveling pad, the use of this type system within the affected waterway area will not be approved. If the concrete leveling pad is founded on sound rock or the M.S.E.S can be located a substantial distance from the affected area of scour, the use of this system could be considered.
- Additional guidance for the use of M.S.E.S. can be found in Article 11.10 of the NYSDOT LRFD Bridge Design Specifications.
- When considering the use of M.S.E.S. for abutment support, the designer should consider the height of the M.S.E.S as well as the sensitivity of the proposed structure to differential settlement or differential movement. The designer should be aware that M.S.E.S. fills are not rigid structures, and are designed to deflect slightly vertically and laterally under load. Movement of the wall may be due to the soils beneath the wall, and/or the compacted backfill. The risk of settlement and differential movement typically increases with height. Due to the potential of larger vertical and lateral movements with taller walls, it is not desirable to place abutments behind M.S.E.S. walls when the M.S.E.S. wall height exceeds 25 – 30 feet.
- The bottom of footing shall be as shown on BD-EE13E. It cannot be placed 4 feet below grade because of interference with the wall straps. This is acceptable because the gradation of the backfill will not allow development of frost heaves.
Design Guidelines:

In addition to design requirements outlined in Article 11.10 of the NYSDOT LRFD Bridge Design Specifications, the following criteria have been adopted by NYSDOT.

- As a preliminary starting point for determining the span length, the centerline of bearings should be assumed as 7’-6” behind the front face of the M.S.E.S.
- A minimum distance of 2 ft. shall be provided between the back of the M.S.E.S. panels and the front face of the abutment footing.
- The top of the M.S.E.S. panel in front of the abutment footing shall be set 1 ft. above the berm elevation.
- A minimum vertical clearance of 4 ft. shall be provided between the bottom of the superstructure and the berm in front of the abutment footing.

Review and Approval:

The M.S.E.S. should be considered as an option for all bridge substructures and developed as a part of the Structure Study Plan. Use of this system should be compared with other abutment types to determine which option best meets project objectives, i.e., structure cost, functionality, construction time, aesthetics and other project specific parameters. The selected option shall then be progressed in the Structure Justification Report through the normal review and approval procedure as described in Section 3.

11.6.1.5 Gravity Abutments

Gravity abutments are large masses of concrete or masonry that have nominal to no design steel in the back face of the stem. This type of abutment uses its own large self weight to provide lateral support and resist overturning forces. A large plan area at the base provides bearing on the soil. This type of abutment is no longer usually used for new bridges by NYSDOT, but they may be encountered on rehabilitation projects.

11.6.1.6 Integral Abutments

In an integral abutment structure, a rigid connection is made between the primary support members of the superstructure and a pile supported substructure by encapsulating the support members into the abutment concrete. Unlike cantilevered abutments, integral abutments do not require a joint in the bridge deck or conventional bearings. An integral abutment does not have a footing, as the abutment is supported on a single row of piles extending out of the abutment stem. The piles are allowed to rotate and horizontally deflect as the abutment stem moves due to thermal expansion of the superstructure.

Integral abutment bridges offer many advantages over conventional cantilevered abutments. Joints at bridge abutments are prone to leak, allowing water containing road salts to drain onto the underlying superstructure beams, bearings, abutment backwalls and bridge seats. By doing away with these joints, future maintenance associated with joint leakage is eliminated, thereby greatly reducing the life cycle cost of the structure. Integral abutments also cost less to construct. Having no footing, no bearings, fewer piles, and relatively simple concrete forming requirements makes integral abutments a cost effective alternative to conventional abutments.
Another advantage of integral abutments is that they can be constructed in a much shorter time as compared to conventional abutments.

Integral abutments should always be considered as the first choice of abutment because of their lower construction cost and superior long-term performance.

Details of integral abutments for each type of superstructure can be found in the current BD sheets.

**Design Methodology**

There are two design methods for integral abutments: the “approximate method” and the “refined method”.

In the approximate design method, the superstructure support members are assumed to be simply supported at the abutment end for design purposes. For the design of the piles, the vertical reaction from the superstructure and the dead load of the abutment is assumed to be uniformly distributed to each pile. Also, bending stresses in the piles are ignored.

Horizontal reinforcement in the abutment stem of steel superstructure bridges is designed by considering the stem to be continuous between piles. The horizontal reinforcement in the front face of the stem is designed to withstand the positive moments between the beams due to full passive soil pressure. The horizontal reinforcement in the rear face is designed to withstand the negative moments at the beams caused by full passive soil pressure. Horizontal reinforcement in the abutment stem for prestressed concrete adjacent box beam superstructure bridges is usually nominal steel based on the prestressed beams fully supporting the abutment stem along its entire horizontal length. Vertical steel in the abutment stem is usually controlled by shear considerations. If the ratio of the abutment stem depth to spacing between the pile supports is 1:1 or greater, then deep beam considerations should be included in the design.

In order to use the approximate design method for integral abutments, each of the following criteria must be met:

- The expansion length used to calculate the movement at an integral abutment shall be less than 165 ft. (The expansion length of an integral abutment structure shall be measured as half the distance between abutments for both single span structures, and continuous structures with expansion piers.)

- The skew shall not be more than 45°.

- The reveal or dimension from the bottom of girder to the top of stone fill or finished grade shall not be less than 1 foot or more than 4 feet.

- For curved steel girder bridges, the horizontal geometry must be such that the NYSDOT LRFD Bridge Design Specifications, § 4.6.1.2, allows the girders to be designed as straight girders.

If any one of the above criteria is not met, then the refined design method must be used. Before using the refined method to design an integral abutment, the designer must obtain the approval
of the Deputy Chief Engineer (Structures). This should be done with submittal of the Structure Study Package for a Technical Quality Review (see Appendix 3D).

In the refined design method, the effects due to skew, curvature, thermal expansion of the superstructure, reveal, and grade are considered. It may be necessary to analyze the superstructure and abutment as a rigid frame system by using either a three dimensional finite element model or a two dimensional frame model. Piles are designed for both vertical loads and for bending. The interaction between the piles and the surrounding soil is considered. For abutments with a large reveal, it may not be possible to design the horizontal reinforcement in the front and rear face of the abutment stem for full passive pressure. The soil pressure resulting from the actual superstructure thermal movement may have to be used. For additional guidance on designing integral abutments using the refined method contact the Office of Structures.

**Approach Treatments**

Integral abutment bridges with:

- A length 100 ft. or less require no provision for expansion at the ends of approach slabs unless the highway pavement is rigid concrete.
- A length more than 100 ft. shall provide for expansion at the end of each approach slab. The span arrangement and interior bearing selection should be such that approximately equal movement will occur at each abutment.

**Pile Requirements**

Integral abutments have special foundation requirements. All integral abutments shall be supported on a single row of piles. C.I.P. or steel H-piles may be used for structures with lengths of 165 ft. or less. Only steel H-piles shall be used for structures with lengths more than 165 ft. When steel H-piles are used, the web of the piles shall be perpendicular to the centerline of the beams regardless of the skew, so that bending takes place about the weak axis of the pile. Orienting the piling for weak-axis bending offers the least resistance to thermal movement but increases the potential for flange buckling. For total bridge length of 245 ft. or greater, the designer shall investigate orienting the piles for strong-axis bending when the total lateral displacement causes buckling of the pile flanges.

The Office of Structures’ Foundation and Construction Unit, in coordination with the Office of Technical Services’ Geotechnical Engineering Bureau, will select a pile type for integral abutments on a case by case basis. If C.I.P. piles are used, pile casing requirements will be provided in the Foundation Design Report.

To accommodate expansion for bridge lengths of 100 ft. or more, each pile shall be inserted in a pre-excavated hole that extends 8 ft. below the bottom of the abutment. After driving the piles, the pre-excavated holes shall be filled with cushion sand. The cost of excavation, steel casings, and cushion sand shall be included in the unit price bid for the pile item. For bridges less than 100 ft., no special pre-excitation provisions are required for expansion purposes.

All piles placed in pre-excavated holes shall be driven to a minimum penetration of 20 ft. This will provide for scour protection and assure sufficient lateral support for the pile, particularly
when the top 8 ft. is excavated and backfilled with sand. If no pre-excavating for the piles is required, penetrations as low as 10 ft. can be used.

A pile bent configuration is to be used for the integral abutment detail. For steel and spread concrete girder bridges, a minimum of one pile per girder shall be used.

**Wingwalls**

Unlike other abutments, the wingwalls for integral abutments have special requirements. In-line wingwalls cantilevered from the abutment are the preferred arrangement. Flared walls cantilevered from the abutment may be considered by the designer on a case by case basis. The use of flared wingwalls should generally only be considered at stream crossings where the alignment and velocity of the stream would make in-line walls subject to scour. Piles shall never be placed under flared wingwalls that are integral with the abutment stem. Generally, the controlling design parameter is the horizontal bending in the wingwall at the fascia stringer caused by the large passive pressure behind the wingwalls. In-line or flared wingwalls connected to the abutment stem with lengths in excess of 13 ft. should be avoided.

Because of high bending moments due to passive soil pressure, it may be necessary to support long wingwalls (13 ft. or more measured along the wall) on their own foundation, which is independent of the integral abutment system. In this case, a flexible joint must be provided between the wingwalls and the backwall. The joint between the abutment and the wingwalls shall be parallel to the centerline of the roadway to accommodate the longitudinal movement of the bridge. A joint that is not parallel to the direction of movement will likely lead to binding between the abutment stem and wingwall. Separate wingwalls may be designed as conventional walls with a footing or a stem with a single row of alternately battered piles. The choice will be governed by the site and loading conditions, but walls using a single row of piles should generally be limited to a height of 13 ft. Separate wingwalls for integral abutments on bridges over water shall be pile supported.

U-wingwalls cantilevered from the abutment stem shall be allowed only if in-line or flared walls cannot be used because of right-of-way or wetlands encroachment. The U-wingwalls shall not measure more than 6'-6" from the rear face of the abutment stem. No piles shall be placed under U-wingwalls physically connected to an integral abutment. This would inhibit the abutment's ability to translate and would cause internal stresses. The distance between the approach slab and the rear face of the U-wingwall should be a minimum of 4 ft. If the approach slab must extend to the U-wingwall, it shall be separated from the U-wingwall by a 2" joint filled with at least two sheets of Premoulded Resilient Joint Filler, Material Subsection 705-07.

**Utilities**

Rigid utility conduits, such as gas, water and sewer, are discouraged for use with integral abutments. If they are used, expansion joints in the conduits must be provided at each abutment. Sleeves through the abutment should provide at least 2" clearance all around the conduit. Flexible conduits for electrical or telephone utilities that are properly equipped with an expansion sleeve through the integral abutment are acceptable.
Stage Construction

When stage construction is used with integral abutments, the use of a closure placement between stages in the abutments shall be considered. The use of a closure placement can reduce the mismatch of the top of slab between stages caused by deflection from the superstructure. A closure placement in the abutment stem shall be required when the dead load deflection from the deck slab placement is calculated to be 3 inches or greater.

11.6.1.7 Semi-Integral Abutments

Description and Design Methodology

Semi-integral abutments use conventionally designed abutments where superstructure girders are supported by bearings and pedestals on a bridge seat. The girders extend over the bridge seat and are embedded in a backwall that hangs behind, but is not connected to, the abutment stem.

Full integral abutments have been used successfully by NYSDOT since the late 1970s. Their performance in terms of durability and first cost has been clearly superior to conventional abutments. This has mainly been due to the elimination of the deck expansion joint and the simple concrete forming required. Unfortunately, site condition criteria sometimes prevent their use. This is usually caused by rock being too close to the ground surface preventing the driving of piles or the necessity of using high abutments because of geometric constraints.

When site conditions have prevented the use of integral abutments, jointless decks at abutments have often been used. Jointless decks at abutments are conventionally designed but the deck slab extends and slides over the backwall. While jointless decks at abutments have performed better than conventional abutments with deck joints, there have been some problems with transverse deck cracking near the abutment backwall. Jointless decks at abutments are also limited to a maximum expansion length of 200 ft. Semi-integral abutments should be considered for use where site conditions prevent the construction of full integral abutments.

Semi-integral abutments are designed as conventional abutments with the following exceptions:

- Backwalls must be designed for full passive soil pressure.
- Wingwalls must be independent from the backwall to allow movement. Clearance details are shown on the applicable BD sheets.
- Adequate clearance to handle expected movements must be provided between the suspended backwall and the abutment stem.
- Provision for expansion at the ends of approach slabs should be provided in accordance with the details on the applicable BD sheet.
- The top reinforcement in the decks slab at the end of the span should be designed for the negative moment produced from the reaction of half the approach slab dead load and a live load reaction placed on the backwall. The dead load of the backwall should not be included because the backwall is constructed in a separate placement before the deck and will not contribute to tensile stress in the deck slab.
- Semi-integral abutments are not allowed for use on one end of a bridge opposite a conventional abutment. Each end of a bridge must have a semi-integral abutment to match passive pressure resistance.
Stage Construction

When stage construction is used with semi-integral abutments, the use of a closure placement between stages in the backwall shall be considered. The use of a closure placement can reduce the mismatch of the top of slab between stages caused by deflection from the superstructure. A closure placement in the backwall shall be required when dead load deflection from the deck slab placement is calculated to be 3 inches or greater.

Selection Criteria and Details

- Maximum skew = 30°.
- Maximum expansion length = 230 ft. (distance to nearest fixed bearing).
- No restriction on abutment height.
- No restriction on maximum grade.
- No restriction on footing type (spread or pile foundation).
- Utility restrictions are the same as integral abutments. See § 11.6.1.6 of the Bridge Manual.
- Single span bridges with two semi-integral abutments can have expansion bearings at each end as long as the grade between bearings is less than 2.5% and there is no stop sign or signal at either end of the bridge.
- Single-span bridges should have one of the abutment bearings fixed if the grade between bearings is ≥2.5%. Multiple-span, continuous bridges can have both abutments with expansion bearings as long as there is a fixed bearing at a pier.
- Fixed bearing designs should size the anchor pin in the bearing to handle passive pressure from the opposing backwall as well as the friction resistance of the approach slab.
- Curved girder structures are allowed if the curved girders are designed as straight as provided in NYSDOT LRFD Bridge Design Specifications, § 4.6.1.2.
- Backfill procedures are the same as for Integral Abutments.
- The hanging backwall may have its bottom surface cast on the ground or formed at the option of the Contractor.
- Polyethylene curing covers need not be placed under the hanging backwall.
- Bearings should be reset to their neutral position after the girder rotates due to deck dead loads.

11.6.2 Abutment and Wall Details

11.6.2.1 Stem Thickness

The stem thickness of cantilevered high abutments is almost always governed by the size of the bridge seat required for clearance between the superstructure and the backwall, the bearings and the backwall, and seismic criteria. For bridges with a pier, seismic criteria may dictate the support length at the ends of beams. The minimum support length (N) in the longitudinal direction should be measured perpendicular to the centerline of bearing. The minimum support length (N) in the transverse direction should be measured perpendicular to the centerline of the beam. The minimum support length shall meet the requirements of NYSDOT LRFD Bridge Design Specifications § 4.7.4.4. The minimum bridge seat width is 3 ft. for steel, bulb tee and AASHTO I-beam superstructures and 2 ft. for adjacent concrete beam superstructures.
The stem thickness of integral and semi-integral abutments shall be as shown in the current BD-ID series. The centerline of the piles and the centerline of bearings of the beams shall always line up.

### 11.6.2.2 Pedestal Dimensions

The minimum height of the shortest pedestal is 6" when used with elastomeric bearings. If multi-rotational bearings are used, then the minimum height shall be 8". The extra 2" is added for tolerance to allow the use of a taller multi-rotational bearing than the one used in the design and still provide a minimum pedestal height of 6". If the difference in height between fascia pedestals is more than 6" then a sloping bridge seat should be used with both fascia pedestals being set at the minimum height. Pedestals more than 1'-6" high should usually be avoided for aesthetic reasons. Pedestals greater than this height should be investigated for their strength acting as a column.

The minimum distance from the center of the bearing anchor bolt to any exposed vertical face of the pedestal shall be 8". In addition, the minimum distance from the edge of the masonry plate to any vertical face of the pedestal shall be 3" unless otherwise accounted for in the design. Masonry plate corners may be cropped to satisfy this requirement. The front face of all pedestals shall be flush with the front face of the abutment.

### 11.6.2.3 Drainage

The fill material behind all walls shall be effectively drained and weepholes shall be placed at a maximum spacing of 25 ft. In counterfort walls, there shall be at least one weephole for each pocket formed by the counterforts. Weepholes shall be located so that their invert is 6" above finished grade or low water in the case of stream bridges. Integral abutments generally do not require weepholes because of their minimal exposed height above finished ground. Weepholes
should only be placed in wingwalls over 40’ in length. Weepholes should not be placed so they drain onto sidewalks or shared-use paths, if possible.

11.7 Bridge Piers

For the purposes of this section, the term “pier” is defined as an intermediate support for a bridge superstructure, between the abutments, extending from below the ground surface to the bottom of the superstructure.

Piers may be required because of long spans, beam depth restrictions, or both. The pier may be a support point along a continuous superstructure, or it may be at the end of one simple span and the beginning of another. In either case, the pier must be designed to safely handle the dead, live, seismic and other loads introduced from the superstructure while at the same time handling any loads acting on the pier from flood water, ice flow, wind, and vehicular or ship impact. Suggested proportions of bridge piers can be found in Section 23.

11.7.1 Pier Types

![Fig. 11.5](image)
Typical Pier Types
(a) Solid  (b) Hammerhead  (c) Multi-column  (d) Pile Bent
11.7.1.1 Solid Pier

Solid piers consist of a solid mass of reinforced concrete, without overhangs, that is usually rectangular in plan. Solid piers are used primarily for river or stream crossings, low-clearance bridges, bridges over divided highways with narrow medians, and where short columns on wide bridges would have high stress due to shrinkage. Solid piers can also be used to meet crash protection requirements adjacent to railroads. This type of pier is currently used by NYSDOT for new bridges.

11.7.1.2 Hammerhead Pier

With increasing pier height and narrow superstructures, the hammerhead pier becomes more economical by reducing the required amounts of material and forming. Hammerhead piers consist of a single large column with a capbeam overhanging on either side. Both the column and cantilevered ends of the capbeam support the superstructure beams. When located in a waterway, pier protection may be required. The overhangs of hammerhead piers may need to be investigated for the bracket and corbel effect as described in Section 15.10. This type of pier is currently used by NYSDOT for new bridges.

11.7.1.3 Multi-Column Pier

When piers need to be tall and wide, a multiple-column pier is usually the best choice. This pier type consists of two or more columns that can be either rectangular or circular. The columns are usually connected by a capbeam that supports the superstructure at points between the columns. For some highly skewed bridges with large beam spacing, it may be necessary to place individual columns under each bearing and to connect the top of the columns with a simple tie strut. When there are only two columns with overhangs, this pier is called a $\pi$ (pi) pier. The overhangs may need to be investigated for bracket and corbel effects as described in Section 15.10. These types of piers are currently used in NYSDOT for new bridges.

A feature of most multi-column piers is the presence of the capbeam. This capbeam is subject to many design considerations that are not applicable to any other type of pier. The width of the capbeam is governed by the necessary width to support the bridge bearings with sufficient cover for the anchor bolts and the required support length for the beams. When the simply supported end of a beam rests on a pier, seismic criteria dictates the support length required. Support length (N) in the longitudinal direction should be measured perpendicular to the centerline of bearings. Support length (N) in the transverse direction should be measured perpendicular to the centerline of the beam. See Section 11.6.2.1. Round columns require that the capbeam be at least 2" wider than the columns on all sides.

For seismic response reasons, high concrete columns (slenderness >60) in multi-column piers shall have reinforced concrete struts between the columns in the middle half of the column height.
11.7.1.4 Pile Bents

Pile bents are the simplest and least expensive piers to construct. This pier consists of driven piles with a concrete cap beam cast over the top of the piles to support the superstructure. This type of pier is inexpensive because there are no footings or columns to form or cast. Pile bents are not frequently used by NYSDOT due to concerns about aesthetics, corrosion of the exposed steel piles or steel pile casings, and the closely spaced piles trapping debris during a flood and reducing the available hydraulic opening.

11.7.2 Pier Protection

Bridges in navigable waterways that are subjected to heavy commercial traffic may require additional protection according to AASHTO Guide Specification and Commentary for Vessel Collision Design of Highway Bridges. Additional information can be found in Section 2 of this manual.

For stream bridges, a recommendation shall be obtained from the Office of Structures’ Hydraulic Design Unit regarding the need for and type of ice breaker for pier noses. If required, the ice breaker shall consist of a steel angle or other device secured to the concrete by a suitable anchor system. For solid piers, this breaker may be attached to the pier stem. For hammerhead piers and multi-column piers, a plinth may be required to provide sufficient strength against the anticipated ice flows. A plinth is a solid mass of concrete that surrounds the pier to an elevation 2 ft. above the 100-year flood or flood of record, whichever is higher. In a navigable stream, the plinth should be carried to 3 ft. above design high water or maximum navigable pool elevation, whichever is higher.

For piers between opposing directions of traffic, appropriate care must be taken to ensure that minimum horizontal clearances and highway traffic barrier requirements are satisfied. For more information, refer to the Highway Design Manual and Standard Sheets.

For multi-column or hammerhead piers adjacent to railroad tracks, the need for crash walls must be investigated based on the proximity of the pier to the tracks in accordance with current AREMA Specifications. Additional information can be found in Section 2 of this manual.