SECTION 5
ABUTMENTS, PIERS AND WALLS

TABLE OF CONTENTS

SECTION 5
ABUTMENTS, PIERS AND WALLS

5.1 GENERAL

5.1.1 Abutments

An abutment supports the end of a bridge span, provides lateral support for approach roadway fill and supports the approach roadway and approach slab. Abutments may be described by their location relative to the approach embankments.

A stub (embankment) abutment is located at or near the top of the approach fill. A partial depth abutment is located approximately mid-depth of the front slope of the approach embankment. A full depth (shoulder) abutment is located at the approximate toe of the approach embankments.

Abutment types shall be selected considering structure aesthetics, foundation recommendations, structure location, and the loads it must transmit to the foundation. For structures over waterways, the abutment type and location should also be specified with consideration to hydraulic conditions at the site. Wherever possible, use stub (embankment) abutments for structures over waterways.

The acceptable abutments types include non-proprietary systems such as gravity walls, cantilever walls, counterfort walls and integral abutments. Preference shall be given to integral abutments. Abutments shall not be placed on fill supported by mechanically stabilized earth walls or prefabricated modular walls, except for Geosynthetic Reinforced Soil-Integrated Bridge Systems. Generally, for abutments and wingwalls founded on rock, where the footings are exposed, the abutment and wingwalls shall be designed without a toe.

5.1.2 Piers

A pier provides intermediate support between the superstructure and the foundation. Pier types shall be selected considering structure aesthetics, foundation recommendations, structure location, and the loads it must transmit to the foundation. If possible, on large projects with many piers, the type of pier shall be consistent throughout the entire project for reasons of economy. The acceptable concrete pier types include wall piers, open column bents, multiple column piers, and single column piers. The use of permanent steel pier bents is discouraged due to future maintenance.

5.1.3 Walls

Walls should be used where the construction of a roadway or facility cannot be accomplished with slopes. Walls can be classified as either retaining walls, or wingwalls. Wingwalls are used to provide lateral support for the bridge approach roadway embankment. For bridges with long wingwalls that are parallel to the roadway, the wingwall shall be referred to as a retaining wall and may receive a wall number.

Wingwalls shall preferably be U-type (parallel to the roadway). Flared wingwalls are permitted where conditions warrant such as for hydraulic performance of waterway crossings. The acceptable wingwall types include non-proprietary systems such as gravity walls, cantilever walls and counterfort walls. Proprietary systems, such as mechanically stabilized earth and prefabricated modular walls, shall only be used for retaining walls.

Retaining walls may be non-proprietary systems such as gravity walls, cantilever walls, counterfort walls or tie-back walls, or may be proprietary systems such as mechanically stabilized earth walls or prefabricated modular walls.

The tops of retaining walls shall not be determined by the exact fill slope but shall follow a smooth unbroken line for a more pleasing appearance. This may require the use of vertical curves, in which case elevations shall be given at 5 foot intervals.

5.1.4 Foundations

A foundation serves to transmit the forces acting on the abutments, piers, or walls into the ground. Foundations are classified as either shallow or deep. A shallow foundation derives its support by transferring load directly to soil or rock at a shallow depth. Spread footings are shallow foundations. A deep foundation derives its support by transferring loads to soil or rock at some depth below the structure by end bearing, adhesion or friction or both. Driven piles, micropiles and drilled shafts are deep foundations.

Foundation type is generally based on the anticipated (structure) loads, underlying soil conditions, scour potential, and site constraints along with the ease and cost of construction.

5.2 IDENTIFICATION NUMBERS

All abutments and piers shall be identified by numbers which start with the number "one" and progress consecutively but separately in the direction of stationing of the roadway, such as, Abutment 1, Pier 1, Abutment 2, etc.

All wingwalls shall be identified by a combination of a number and a letter (alphanumerical), such as 1A or 1B. The number used must correspond to the abutment to which the wingwall is attached. Looking up station, the letter "A" indicates the wingwall is on the left and the letter "B" indicates the wingwall is on the right.

Retaining walls shall be identified by three numbers that start at 101 and progress consecutively in the direction of stationing of the roadway, such as Retaining Wall 101, Retaining Wall 102. Parallel walls along both edges of roadway beginning at the same station are to follow wingwall rules. These numbers may designate a proprietary wall, a proprietary embankment wall, a cast-in-place wall or a soil nail wall. A table shall be provided in the contract identifying the relationship between the wall number, type and site number of the wall, and location as in the following:

RETAINING WALL NUMBER	DESCRIPTION	LOCATION
101	Embankment Wall – Site 1	Station 10+00 to 12+50
102	Retaining Wall – Site 2	Station 25+50 to 32+50
103	Cast-in-place – Site 3	Station 70+00 to 72+50
104	Retaining Wall – Site 4	Station 80+00 to 82+50

5.3 EXCAVATION

5.3.1 General

Contract items for structure excavation, unless the work is included under other items, are required for the removal of all material of whatever nature necessary for the construction of foundations for bridges, box culverts, retaining walls and other structures. The items specified in the contract depends on the type of material removed, earth or rock, and whether or not separate payment will be made for the work related to cofferdams and dewatering.

On any project where only some of the structures and/or their components require cofferdams and some do not, a combination of structure excavation items shall be shown in the contract. The contract for the structures and components requiring “Cofferdam and Dewatering” shall clearly delineate the pay limits and the limits of the cofferdam.

5.3.2 Construction Requiring Cofferdam and Dewatering

A cofferdam is a structure that retains water and soil that allows the enclosed area to be pumped out and excavated dry to permit construction.

At water crossings, where structures or their components are located partially or wholly in the water and the bottom of the footing is below water level, or where a considerable flow or concentration of water is present that cannot be diverted, partly or wholly, from the site, the contract shall include the following item:

ITEM NAME	PAY UNIT
Cofferdam and Dewatering	L.F.

The hydraulic design of the cofferdam should be done in accordance with the **Drainage Manual.**

The contract shall also include either one or both of the following items, as required for the type of material removed:

ITEM NAME	PAY UNIT
Structure Excavation – Earth (Excluding Cofferdam and Dewatering)	C.Y.
Structure Excavation – Rock (Excluding Cofferdam and Dewatering)	C.Y.

Where underwater (tremie) concrete may be used to seal the bottom of a cofferdam to allow dewatering, the weight of the tremie concrete, adjusted for buoyancy, shall be added to pile and foundation loads for design purposes.

5.3.3 Construction in the Dry

Where structures or their components are to be constructed in the dry or where water may be temporarily directed away from an excavation, eliminating the need for a cofferdam, the contract shall also include either one or both of the following items, as required for the type of material removed:

ITEM NAME	PAY UNIT
Structure Excavation – Earth (Complete)	C.Y.
Structure Excavation – Rock (Complete)	C.Y.

The temporary redirection of water or water courses, either partially or wholly, from an excavation or site, must be coordinated with hydraulic studies and DEEP, Federal and State permit submittals. At the sites where water is directed away from an excavation, the following item shall be included in the contract.

ITEM NAME	PAY UNIT
Handling Water (Site No.)	L.S.

5.4 EXPANSION, CONTRACTION AND CONSTRUCTION JOINTS

Expansion and contraction joints in concrete abutment and wall stems shall be provided in accordance with **LRFD**. Construction joints shall be placed as conditions warrant. Construction joints other than those shown in the contract require prior approval from the Engineer. Expansion or contraction joints should not be provided in footings. Footings for abutments and walls should be continuous including any steps provided.

No reinforcement shall pass through expansion and contraction joints. Reinforcement shall pass through construction joints.

5.5 DAMPPROOFING

The rear face of cast-in-place and precast abutments and wall stems shall be damp- proofed.

5.6 BACKFILL REQUIREMENTS

5.6.1 General

Unless otherwise directed, all abutments, wingwalls and retaining walls shall be backfilled with Pervious Structure Backfill to the limits described below. Pervious Structure Backfill is a clean, granular soil. For design purposes, the effective angle of internal friction shall be taken as equal to 35 degrees.

5.6.2 Backfill Limits

Indicate a wedge of Pervious Structure Backfill above a slope line starting at the top of the heel and extending upward at slope of 1:1½ (rise to run) to the bottom of the subbase. In cut situations, the following note, with a leader pointing to the slope line, shall be placed in the contract:

Slope line except where undisturbed material obtrudes within this area.

5.7 SUBSURFACE DRAINAGE *(Rev. 04/19)*

5.7.1 General

Subsurface drainage shall be accomplished with the use of weepholes extending through the wall stems, or underdrains placed along the wall stems. Subsurface drainage for proprietary retaining walls shall conform to the owned special provisions governing their design and construction.

5.7.2 Weepholes and Bagged Stone

Except for structures placed on embankments, 4 inch diameter weepholes, sloped 1:8 (rise to run), shall be placed approximately 1 foot above the finished grade at the front face of the wall stem. For structures placed on embankments, the weepholes shall be extended through the slope with an outlet. Weepholes shall not drain onto adjacent sidewalks. Weepholes should be spaced at approximately 8 to 10 foot intervals unless conditions warrant a closer spacing. The spacing and invert elevations of the weepholes shall be shown in an elevation view.

The cost of furnishing and installing weepholes is included in the cost of the concrete. Bagged Stone is paid under “Pervious Structure Backfill”.

5.7.3 Underdrains and Outlets *(Rev. 04/19)*

Underdrains shall have a 6 inch nominal diameter, perforated and placed at the base of the stem and sloped a minimum of 1%. Underdrains shall be either connected to the roadway drainage or to a free outlet. The location and limits of the underdrain shall be shown in plan view. The invert elevations shall be shown in an elevation view. Outlets for underdrains shall consist of pipe laid in a trench and refilled with earth. The size and type of outlet pipe shall be the same as that of the underdrain to which it is connected, except that it shall not be pervious to water.

Underdrains shall be paid for under the item “6 inch Structure Underdrain”. Outlets shall be paid under “6 inch Outlets for Underdrain”. When an underdrain is connected to the roadway drainage, the pipe beyond the face of the wall stem or the end of the wall shall be shown in the contract to be included in the roadway items, and should be coordinated with the roadway designer.

5.7.4 Subsurface Drainage Selection Criteria

5.7.4.1 Full Height Abutments

At abutments in cut situations, either an underdrain or weepholes may be used with the latter being preferred. Weepholes should be used at abutments located on fills. When there is a sidewalk in front of any abutment, an underdrain should be used. Where this type of abutment is used at water crossings, drainage shall be provided by weepholes.

5.7.4.2 Perched Abutments

At abutments in wet cuts, an underdrain should be used. At abutments in dry cuts and fills, extended weepholes should be used. If the total length of the extended weepholes exceeds what’s required for underdrains, the latter is preferred.

For walls with a fully exposed face adjacent to a sidewalk, an underdrain should be used. For walls with a fully exposed face not adjacent to a sidewalk, weepholes should be used. For walls with a partially exposed face, an underdrain should be used.

5.8 APPROACH SLABS *(Rev. 04/19)*

Approach slabs shall be provided on all bridges carrying State highways. Approach slabs shall be strongly considered on all bridges undergoing superstructure replacement and local road bridges.

Approach slabs should extend the full width of the roadway (including shoulders), have a standard length of 16 feet and be 1.25 feet thick. Generally, approach slabs should follow the skew of the bridge for skew angles up to 35 degrees. For skew angles greater than 35 degrees, the ends of the approach slabs should be square to the roadway with a minimum length of 15 feet. Acute corners of approach slabs and approach pavement should be squared off for a distance of five feet from the gutter line. Approach slabs shall be anchored to the bridge abutment.

Approach slabs shall be constructed in accordance with **BDM** [6]. Approach slabs shall be covered with a waterproofing membrane and a bituminous concrete overlay. All the material items used in the construction of the approach slabs, including the overlay, shall be included in the structure items and quantities.

All elevations necessary for construction of the approach slabs shall be shown in the contract. These elevations shall include the elevations at the point of application of grade line, the gutter lines and at shoulder break lines at both ends of the approach slabs.

5.9 SLOPE PROTECTION

Provisions shall be made for protection of earth slopes in front of abutments on bridges over State highways, local roads, railroads and waterways. The slope of the embankment in front of the abutment shall be no steeper than 1:2 (rise to run).

5.9.1 Selection Criteria

The type of slope protection shall generally conform to the following criteria:

- Crushed Stone for Slope Protection shall be used under structures overpassing Interstate highways, railroads and waterways. Protection between the edge of the shoulder and the toe of the slope should be founded on a 6 inch granular fill base or geotextile. The limits of this base should be shown in the contract and shall be include in the estimated structure quantities.

- Concrete Block Slope Protection shall be used under structures overpassing State highways and local roads. The block shall be anchored or mortared into place to prevent vandalism. The use of granite block is not permitted due to its higher cost. Cast-in-place concrete is not permitted due to cracking and settlement of existing installations.

- Abutment slope protection for bridges over waterways should be designed in accordance with the procedures outlined in **HEC-18** or successor documents as well as documents referenced therein.

5.9.2 Limits of Slope Protection

The limits of slope protection shall cover the complete area, exclusive of sidewalks, from the edge of the shoulder to the face of the abutment stem and transversely within lines parallel to and 2 feet outside of the bridge rails.

5.9.3 Inspection Shelf

Provisions for inspection access (for bridge inspectors) shall be provided on all slopes. On stems with exposed heights less than or equal to 5 feet, access shall be provided by a shelf at the top of the slopes. On stems with exposed heights greater than 5 feet, access may be by a shelf at the top of the slopes or ladder stops on the slope itself. The contract shall include details of the intersection of the shelf and the slope along the wingwalls.

5.10 SURFACE TREATMENTS

5.10.1 General

In general, abutments, piers and walls shall be faced with standard formed concrete. Surface treatments other than standard formed concrete should only be considered in the following situations:

- When the structure has been determined by the **CTDOT** to be architecturally or historically significant.
- If there is a desire expressed for special surface treatments during the public involvement process of the project. The basis for surface treatments should involve the character of the area in which the wall is to be built. The use of special surface treatments should be coordinated with the town or city administration.
- Where the structure is to be built on a designated State scenic highway.
- The structure is part of the Merritt Parkway. For these structures, every attempt should be made to replicate the appearance and structure type that was originally built.
- The treatment of the structure is part of a right-of-way settlement with a property owner. For instance, if a property has an existing stone wall that is to be removed and relocated, the owner may request that the replacement wall also have a stone surface.

If special surface treatments are desired for a particular structure, every attempt should be made to achieve architectural aesthetics by means of shape and form, not through surface treatments alone. Surface treatments should generally be used in conjunction with the shape of the structure.

5.10.2 Form Liners

When the use of surface treatments has been determined to be appropriate, the preferred method is the use of concrete form liners. Form liners offer a lower cost alternative to stone veneer. There is a wide variety of form liners available for different architectural treatments. Linear corrugated form liners should be avoided since it is difficult to hide joint lines and form tie holes. Form liners that replicate stone are preferred since the random nature of the surface makes it easy to hide form tie holes.

5.10.3 Simulated Stone Masonry

In more sensitive areas, where the look of real stone is required, the use of simulated stone masonry may be considered. Simulated stone masonry utilizes a flexible form liner system and color stains or dry-colorant admixtures to provide the aesthetic appeal of natural stone with the durability of reinforced concrete.

5.10.4 Stone Veneer

The use of stone veneer on concrete should only be considered in very sensitive areas where the increased cost can be justified. Stone veneer shall only be used with approval from the **CTDOT**.

5.11 REQUIREMENTS FOR ABUTMENTS

5.11.1 General

The abutments shall be designed, unless otherwise noted, in accordance with the **LRFD.**

Generally, abutments shall be constructed of reinforced concrete. Cast-in- place footings and stems shall be constructed in accordance with **BDM** [6].

5.11.2 Gravity and Counterfort Abutments

5.11.2.1 Steel Girder and Concrete Bulb Tee and Box Girder Bridges

Gravity, cantilever, and counterfort walls, with bridge seats, may be used for abutments.

Bridge seats shall be sloped with a minimum 2 inch draw from the front face of the backwall and closed at the ends. When determining bridge seat widths, consideration shall be given to superstructure jacking requirements as given in **BDM** [7.2.13] and the clear distance requirements between the end bearing diaphragms and the front face of the backwall as given in **BDM** [7.3.3.7]. On bridges constructed with box girders, the clear distance from the end of the box girder to the face of the backwall should be no less than two feet.

At the elevation of the bridge seat, the minimum dimension from the front face of the abutment stem to the centerline of the bearings shall be 1.25 feet. The minimum backwall thickness shall be 1.25 feet. Stem thicknesses may be less than the combined dimensions of the bridge seat and backwall.

5.11.2.2 Butted Deck Unit and Box Beam

Gravity, cantilever and counterfort walls, with bridge seats, may be used for abutments.

Bridge seats shall be sloped to match the grade of beams. Provisions should be provided on the contract plans to provide drainage at the low end of span.

At the elevation of the bridge seat, the minimum dimension from the front face of the abutment stem to the centerline of the bearings shall be 9 inches. The minimum backwall thickness shall be 1.25 feet. Stem thickness may be less than the combined dimensions of the bridge seat and backwall.

5.11.3 Integral Abutments

Integral abutments are defined as abutments that are cast integrally with the superstructure. Integral abutments should be considered on all bridges, and especially where pile supported foundations are required, since the use of integral abutments will greatly reduce the number of piles and simplify the abutment details.

All integral abutment bridges shall be designed with full width approach slabs in order to minimize surcharge loads and hydrostatic pressures. Integral abutments shall be designed with U-Type wingwalls.

5.11.3.1 Fully Integral Abutments

Fully integral abutments are defined as abutments that are integral from the superstructure through to the piles. In order to control the effects of the soil mass on the abutment, the maximum height of the cast-in-place abutment shall be 8 feet.

The piles shall be placed in a single line and typically are oriented such that the weak axis of the pile is parallel to the abutment face. For design purposes, the connection of the superstructure to the substructure shall be modeled as a pinned connection. The piles shall be designed for vertical forces only and adhere to the guidelines in **HEC-18** and **HEC-22**. The effects of thermal expansion, end rotation of the superstructure, and soil forces should be neglected.

5.11.3.2 Semi-Integral Abutments

Semi-integral abutments are defined as abutments that are integral from the superstructure through a portion of the abutment stem. Typically, a joint will be detailed in the abutment stem. In order to control the effects of the soil mass on the abutment, the maximum height of the integral portion of the cast-in-place abutment shall be 8 feet.

For design purposes, the connection of the superstructure to the substructure shall be modeled as a pinned connection. The lower portion of a semi-integral abutment shall be designed as a standard cantilever abutment with all vertical forces from the superstructure transmitted to lower portion of the abutment.

5.12 REQUIREMENTS FOR PIERS

5.12.1 General

The piers shall be designed, unless otherwise noted, in accordance with the **LRFD**. Generally, piers shall be constructed of reinforced concrete. While the design of steel pier caps is allowed, they are discouraged. For additional information, see **BDM** [7]. Piers may be made integral with the superstructure.

Footings, concrete pier stems, columns, and pier caps shall be constructed in accordance with **BDM** [6]. Post-tensioned concrete pier caps may require concrete with greater compressive strengths.

All reinforcement in piers shall conform to **BDM** [6]. The concrete cover over the reinforcement in pier footings, stems, columns, and pier caps shall be 3 inches.

Circular concrete columns are preferred over rectangular concrete columns. With circular columns, spiral reinforcement is preferred over ties.

Cantilever concrete pier caps shall be post tensioned in order to eliminate cracking. The design shall be based on zero tension in the top of the cap after all losses have occurred under all loads.

The top surfaces of concrete piers and concrete pier caps shall have a transverse slope of 1:10 (rise to run). The slope shall be in both directions from the centerline to the face of the pier with a minimum draw of 2 inches.

Drilling holes for anchor bolts will not be permitted in concrete pier caps for new structures. Anchor bolts installed before the concrete is placed shall be set and held accurately by a template. Anchor bolts to be set after the concrete is poured shall be set in forms that shall be placed before the concrete is poured. The designer shall indicate in the contract which method of setting anchor bolts is to be used.

For structures over waterways, the following criteria applies:

- Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is likelihood that the stream channel will shift its location over the life of the bridge.

- Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.

- Streamline piers to decrease scour and minimize the potential for the buildup of ice and debris. Use ice and debris deflectors where appropriate.

5.12.2 Wall Piers

A wall pier consists of a solid wall that extends up from its foundation. Generally, wall piers or wall piers combined with open bents should be considered at water crossings. Wall piers offer minimal resistance to water and ice flows.

5.12.3 Open Column Bents

An open column bent consists of a pier cap beam and supporting columns in a frame-type structure. Open column bents should be considered for wide overpasses at low skews.

Open column bents founded on rock shall generally be designed with isolated footings while open column bents founded on soil shall generally be designed with combined footings. When these piers are founded on piles, they may be designed with either isolated or combined footings.

5.12.4 Multiple Column Piers

A multiple column pier consists of an individual column supporting each beam or girder. Multiple column piers should be considered for wide overpasses at low skews.

5.12.5 Single Column Piers

Single column piers are simple, easy to construct, require minimum space, and provide open appearance to traffic. Single column piers may have a hammer head pier cap. Hammer head piers should be considered for overpasses at high skews with tight alignment constraints. This type of pier provides open appearance when supporting structures with long spans.

5.12.6 Protection from Adjacent Traffic

To limit damage to piers by vehicular traffic, crash walls shall be provided. The minimum height of the wall shall be 42 inches, and shall be placed a minimum of 6 inches from the face of the pier.

To limit damage to piers by railroad equipment, crash walls shall be provided in accordance with **AREMA**. Extensions to crash walls may be required to satisfy site conditions. The top surface of the crash wall shall have a transverse slope of 12:1.

5.13 REQUIREMENTS FOR WALLS

5.13.1 General

The following is a list of appropriate retaining wall types that may be considered:

1. Non-Proprietary: Precast and Cast-In-Place Reinforced Concrete

2. Proprietary: Prefabricated Modular Wall Systems
   Mechanically Stabilized Earth Walls (precast concrete)
   Mechanically Stabilized Earth Embankment Walls (dry-cast block)

A design is required for the non-proprietary walls only. The Contractor shall be responsible for the structural/internal design of the proprietary walls. For projects where proprietary retaining walls are included, the walls will be bid as a lump sum for each site. The designer shall clearly define the horizontal, vertical, and transverse pay limits in the contract.

The **CTDOT** maintains a list of approved proprietary retaining walls for each category listed above. No other proprietary retaining walls will be allowed.

5.13.2 Wall Selection Criteria

The designer shall select the appropriate retaining walls for each site. The designer may need to contact wall manufacturers to ensure that each wall will be suitable at each site, and fit within the available right-of-way. The following general criteria should be followed for the selection of appropriate retaining walls:

5.13.2.1 Walls < 8 Feet (Measured from Front Grade to Back Grade)

5.13.2.1.1 Embankment Walls *(Rev. 12/19)*

Embankment walls are defined as mechanically stabilized earth structures faced with dry cast concrete block that are less than 8 feet high and support an embankment. Embankment walls are typically used to support earth only, not roadways or where there is a potential for future underground utilities or drainage structures. The mechanical strength of the wall comes from soil reinforcements comprised of either geogrids or welded wire mesh.

Embankment walls are proprietary wall systems, and there are several approved manufacturers of these types of walls. It is not necessary to design a cast-in-place retaining wall as an alternate; however, the designer shall lay out the embankment wall in the contract with at least the following information:

- Retaining wall plan view with all required dimensions, contours, property lines, utilities, etc.
- Retaining wall elevation view showing top and bottom elevations, approximate step locations, existing and finished grade, etc. Where required, the designer shall also show the location of railings or fences required to be attached to the top of the wall.
- Typical sections (schematic) of the wall showing pay limits and minimum drainage requirements.
- Borings and soils information including the maximum factored bearing resistance.
- Temporary Sheeting required for excavation.

5.13.2.1.2 Cast-in-Place Walls

For locations where embankment walls are not appropriate (in accordance with the criteria listed above), a cast-in-place wall should be designed and detailed.

At the discretion of the **CTDOT**, proprietary walls may also be allowed if the wall is very long resulting in a large overall area. The designer shall provide the same information for proprietary walls as required in **BDM** [5.13.2.2.1].

If the appearance of stone is desired, architectural form liners should be used. These liners are significantly less costly than stone veneer. If there are multiple walls on a project, the surface treatment shall be similar for each wall.

5.13.2.2 Walls > 8 Feet (Measured from Front Slope to Back Slope)

5.13.2.2.1 Walls < Than 5,000 ft^2 of Vertical Face Area (Measured to Bottom of Footing)

For this situation, a cast-in-place wall should be designed to be bid against the proprietary walls. The Contractor may be able to build the cast-in-place wall with his own forces at a lower cost. For these situations, the designer shall completely design and detail the cast-in- place wall. For the proprietary retaining wall, schematic typical cross sections combined with the cast-in-place details should be enough for the proprietary wall manufacturers to design their walls.

The designer shall provide a list of the specific wall types allowed for each site. For instance, the designer may limit the selection based on the available right of way at a site.

For mechanically stabilized earth walls with metallic soil reinforcements that are to be built in areas of potential stray currents within 200 feet of the structure (for example: an electrified railroad), a corrosion expert shall evaluate the potential need for corrosion control requirements.

If the wall is required to be designed for seismic loads, it shall be stated in the notes for the wall.

5.13.2.2.2 Walls > 5,000 ft^2 of Vertical Face Area *(Rev. 12/19)*

For this situation, proprietary retaining walls will most likely be more economical; therefore, a cast-in-place wall design should generally not be done except where site conditions or soil constraints may require a cast-in-place wall. The designer shall determine which proprietary retaining walls are appropriate for each site. The designer shall also lay out the proprietary retaining walls in the contract with at least the following information:

- A list of the specific walls allowed for each site. For instance, the designer may limit the selections based on the available right of way at a site.

- Retaining wall plan view with all required dimensions, offsets, contours, property lines, utilities, etc.

- Retaining wall elevation view showing top and bottom elevations, approximate footing step locations, existing and finished grade, etc. Where required, the designer shall also show the location of railings or fences, light standard and/or sign support anchorage locations, rigid metal conduit and junction boxes.

- Typical Sections (schematic) of the wall showing pay limits and minimum drainage requirements. Specific details are not required for each wall manufacturer, only for each wall type.

- All soils information normally used for the design of a cast-in-place wall shall be shown in the contract, including but not limited to borings and maximum factored bearing resistance.

- Temporary Sheeting required for excavation.

- If the wall is required to be designed for seismic loads, it shall be stated in the notes for the wall.

- For mechanically stabilized earth walls with metallic soil reinforcements that are to be built in areas of potential stray currents within 200 feet of the structure (for example: an electrified railroad), a corrosion expert shall evaluate the potential need for corrosion control requirements.

5.13.2.2.3 Inverted Wall Systems *(Rev. 12/19)*

Inverted wall systems are modular block walls with a modified design methodology where smaller modular units are at the bottom of the wall and larger units at the top.

Due to the current sole source requirement, inverted wall systems can only be used where site conditions restrict the use of all other retaining wall systems. Inverted wall systems are well-suited for the specific scenario in which ground conditions restrict the use of temporary earth retaining systems (such as where ledge prohibits driven or drilled piles; adjacent structures may be damaged due to vibrations) and open excavation is restricted (e.g. – undermining of adjacent structures, utilities, etc.; Rights-of-Way constraints).

5.13.2.3 Architectural Treatments

If the appearance of stone is desired, architectural form liners should be used. These liners are significantly less costly than stone veneer. Several of the proprietary retaining walls can be built with form liners resembling stone. The designer should contact the approved wall manufacturers for specifics about available form liners. Every effort should be made to keep the surface treatment similar for all the wall types specified.

5.13.2.4 Large Anticipated Settlements and Liquefaction

If large settlements or liquefaction are anticipated that require a wall supported on piles, in general, proprietary retaining walls should not be used. Even though these walls can accommodate some settlement, the opening and closing of the joints would produce an undesirable appearance. For these situations, a cast-in-place wall should be designed supported on piles, or the proprietary retaining walls shall be detailed with pile supported full width footings.

5.13.2.5 Walls Supporting Roadways

If the wall supports a roadway where there is a possibility of future underground utilities and drainage structures, mechanically stabilized earth walls should not be used. This would not be the case for walls supporting limited access highways. If the utilities are extensive or deep, it may not be possible to use the modular wall options either.

5.13.2.6 Multiple Walls in Same Project

If there are several retaining walls within the same project, the designer may wish to require that all walls selected by the Contractor for the project be manufactured by the same wall supplier. This is especially true for walls that are close together.

5.13.2.7 Pre-Construction Procedures

The designer should contact the wall companies for tall walls or walls with unusual geometry to be sure that the proprietary walls will function at each site. This should be done during the preliminary design phase of the project.

Prior to construction advertising, the designer should inform in writing each proprietary wall company that they are listed as acceptable alternates in the contract. This will allow them to obtain the contract in order to accomplish preliminary design during advertising for the project. Part of this submission should include the anticipated advertising date.

5.13.3 Requirements for Cast-in-Place Non-Proprietary Walls

5.13.3.1 Flared Type Wingwalls and Retaining Walls

The stems of flared type wingwalls shall be 1.33 feet wide at the top, with the rear face battered. The minimum batter shall be 10:1.

5.13.3.2 U-Type Wingwalls with Sidewalks

The top of the wingwall section shall conform to the parapet width for the full length. If a batter is required, the rear face shall be vertical to approximately 12 inches below the sidewalk.

5.13.3.3 U-Type Wingwalls with Sloped Curb

The top of the wingwall section shall conform to the parapet width for the full length. If a batter is required, the rear face shall be vertical to approximately 12 inches below the bottom of subbase.

5.14 REQUIREMENTS FOR FOUNDATIONS

5.14.1 Structures over Waterways *(Rev. 09/23)*

Foundations for structures adjacent to or within waterways shall meet the requirements of Article 2.6 amended as follows:

This BDM section is presented in a format like that used in the LRFD. In the LRFD, the code is in the left column and the commentary is in the right column. Below the BDM practice is in the left column and the BDM commentary is in the right column.

The BDM practice and commentary amend the LRFD by supplementing, revising, or deleting the LRFD code and commentary.

Headings, table or figures, such as x.x.x.x.x-General, refer to the LRFD. Headings, table or figures, such as x.x.x.x.xCT or Table x.x.x.x.xCT are new requirements of the CTDOT.

References in the BDM practice to other articles in the BDM are shown as BDM [x.x] or BDM [Table x.x]. References to the LRFD are shown as Article x.x or Table x.x (which is consistent with the LRFD convention). Non-structural references in the practice are only made to other CTDOT engineering discipline manuals, such as the CTDOT Drainage Manual.

References in the BDM commentary to the BDM and LRFD match the format used in the BDM practice. The commentary may include references to documents from any source.

2.6— HYDROLOGY AND HYDRAULICS

2.6.1— Hydraulic Analysis

2.6.4.4—Bridge Foundations

2.6.4.4.1 General

This section shall be supplemented with the following:

Foundations, for both new and existing structures, adjacent to or within waterways subjected to scour shall be designed for changes in foundation conditions resulting from the scour design flood and the scour check flood. Structure foundations shall include bridge foundations, supporting intermediate piers and abutments, foundations for 3-sided frames supporting pedestals and frame legs, and foundations for walls retaining transportation facilities. The design of foundations for scour encompasses both the placement of the foundations and the evaluation of the foundations for changes in conditions due to the scour. The scour design flood, scour check flood, and the changes in foundation conditions resulting from scour shall be determined in accordance with the CTDOT Drainage Manual.

C.2.6.4.4.1

This section shall be supplemented with the following:

The terms “scour design flood” and “scour check flood” used in this practice are consistent with *Evaluating Scour at Bridges*, 5th Edition, dated April 2012, FHWA-HIF-12-003, Hydraulic Engineering Circular No. 18 (HEC-18). The terms “scour design flood” and “scour check flood” are equivalent to the LRFD terms “design flood for bridge scour” and “check flood for bridge scour” respectively.

Since the terminology used by disciplines and documents varies, for clarification refer to *Technical Brief (TechBrief)* FHWA-HIF-19-060. The *TechBrief* defines terms, differentiates between hydraulic design and scour design and describes the interaction of limits states and scour depths in foundation design within the context of the LRFD.

This practice separates the design of foundations for scour into 2 parts, foundation support and placement, and foundation evaluation for limit states, to differentiate and clarify requirements.

Scour is determined in accordance with the CTDOT Drainage Manual. The manual refers to the use of HEC-18 for scour design.

Per CTDOT Drainage Manual, Section 9.5, scour information is presented in a scour evaluation report (see Section 9, Appendix C). The report may include, but not be limited to, the following: scour depths and limits, flow elevations and flood velocities for each flood event; recommended foundation placement elevations; limits and details for scour countermeasures; and hydraulic analysis assumptions for existing substructure components that remain in place.

Designers should understand that designing structures for scour requires coordination and collaboration between hydraulics, geotechnical and structural engineers. Multiple design iterations may be required before a final solution is developed that meets each discipline's requirements.

Regarding scour at bridges, the bridge abutment shall be defined to include the structure(s) fully or partially supported by the abutment foundation, such as the bridge end, abutment wall, curtain and wing walls, and approach slab, and walls critical to bridge stability. Retaining walls supporting the approach fill side slopes that are not critical to the bridge stability are not considered part of the bridge abutment for scour considerations.

2.6.4.4.2—Bridge Scour

This section, including the title, shall be deleted and replaced with the following:

2.6.4.4.2—Foundation Scour

The greatest scour depth may not occur at the least frequent flood (i.e., greatest discharge) event selected for the scour design flood frequency and scour check flood frequency. Flood events associated with low tail water, occurrence of ice or debris dams, overtopping conditions, waterway confluences, changes in the angle of approach flow due to movement of the channel or any other flood events that can adversely affect the scour depth shall be considered when determining the governing the flood event for the scour design flood and the scour check flood. The term scour design flood is used to designate a flood event, with a magnitude that is less than or equal to the discharge selected for the scour design flood frequency, that will cause the worst-case scour. Similarly, the term scour check flood is used to designate a flood event, with a magnitude that is less than or equal to the discharge selected for the scour check flood frequency, that will cause the worst-case scour.

C2.6.4.4.2

This section shall be supplemented with the following:

For additional information for determining scour depth, refer to the CTDOT Drainage Manual, HEC-18 and *TechBrief* FHWA-HIF-19-060.

Low tail water conditions, occurrence of ice or debris dams, overtopping flood conditions, or changes in the angle of approach flow due to movement of the channel tend to be the most problematic with regard to scour depth.

To eliminate the potential errors, the terms scour design flood and scour check flood should be used to designate the governing flood events that will cause the worst-case scour.

Designing foundations scour does not preclude potential damage to highway approaches from flood events. For additional information, refer to HEC-18, Section 2.1, Item 4.

The potential for stream migration and its effect on scour depth outside of the main channel shall be considered when determining the potential depth of scour at the bridge abutment and interior piers outside of the main channel.

The potential for scour to undermine the channel slope, resulting in a slope failure shall be considered when assessing the potential scour depth, including the potential of the scour-induced slope failure to cause lateral loading on the bridge foundations for the abutment as well as the nearby intermediate pier foundations.

If scour induced slope failure of the channel bank and approach fill slope is possible, refer to the CTDOT Drainage Manual. The manual refers to the use of the document *Stream Stability at Highway Structures,* 4th edition*, dated 2012,* Hydraulic Engineering Circular (HEC-20), FHWA-HIF-12-004, for scour design.

For deep foundations, the effect of the foundation components, such as footing/pile caps, piles, etc., above the total scour depth shall be included in determining the total scour depth.

For new structures, changes in foundation conditions resulting from scour shall be determined without the benefits provided by scour countermeasures.

For additional information on scour countermeasures, see Article 2.6.4.4.5CT.

The provisions for tsunami-induced scour are contained in the AASHTO Guide Specifications for Bridges Subject to Tsunami Effects.

2.6.4.4.3CT—Foundation Support and Placement

Foundations adjacent to or within waterways subjected to scour shall be supported on piles or drilled shafts, scour resistant rock, or on spread footings founded below (outside) the scour limits. Supporting foundations subjected to scour on piles, drilled shafts or on scour resistant rock is preferred. The use of structural tremies (concrete placed under water) to directly support a foundation without piles is not permitted.

The use of scour countermeasures shall meet Article 2.6.4.4.5CT.

The placement of foundations subject to scour shall consider the type of foundation support, the location of the foundation within the channel, the migration of the thalweg, and the scour due to the scour design flood and scour check flood.

HEC-18, Article 2.3.2, notes that the thalweg of channels can migrate within the bridge opening. Foundation placement shall consider this condition.

The potential for lateral channel migration shall also be considered in determining foundation placement. For abutment and pier foundations located outside the main channel, where there is potential for lateral channel migration and the foundation could end up in the migrated channel, the foundations shall be placed using the scour depth determined for the main channel.

Foundation placement shall meet the requirements of BDM [5.14.2], [5.14.3] and [5.14.4].

Foundation placement shall be adjusted to avoid effects of potential channel slope failure due to scour and to ensure proper embedment of scour countermeasures.

Recommendations for the placement of foundations subject to scour are provided in the scour evaluation report described in the CTDOT Drainage Manual.

If the potential effects of lateral channel migration are not specifically addressed by the scour evaluation report, the designer should confirm with the hydraulics engineer that potential lateral channel migration will not affect the placement of the foundations.

For deep foundations, such as foundations on piles or drilled shafts with and without projecting footings/piles caps, subjected to scour, the bottom of abutment/pier stems (top of footing/pile cap, as applicable) shall be placed below (outside) the scour due

The placement of foundations on piles or drilled shafts (deep foundations) with footings/piles caps is consistent with Section 2.2, Page 2.6, Item 6 entitled "For Deep Foundations (Drilled Shaft and Driven Piles) With Footings or Caps" of HEC-18.

to the scour design flood. The scour, in this case, shall be no less than the summation of the long-term degradation and the contraction scour, and include the effect of lateral channel migration, as applicable.

For shallow foundations on spread footings, supported on soil or erodible rock, subjected to scour, the top of the footing shall be placed below the total scour due to the scour design flood and the scour check flood.

Shallow foundations supported on scour resistant rock shall be designed, detailed, and constructed to maintain the integrity of the supporting rock. The bottom of the foundation shall be at or below the top of scour resistant material.

For bridge abutments supported on a spread footings that are reliant for support on a retaining wall subject to scour, the bottom of retaining wall stem (top of footing/pile cap, as applicable) shall be placed below (outside) the total scour due to the scour design flood and the scour check flood regardless of whether the foundation is a deep foundation or a shallow foundation.

For recommendations on the placement of foundations for abutments and piers entirely outside the total scour limits, coordinate with hydraulic and geotechnical engineers.

The final foundation placement and details shall be determined in coordination with a multi-discipline group of structural, hydraulic, and geotechnical engineers. The foundation placement and details shall be clearly specified in the contract documents and shall meet the recommendations included the final scour evaluation report, the final hydraulics report and final geotechnical report.

Figures 2.6.4.4.3-1CT, 2.6.4.4.3-2CT, 2.6.4.4.3-3CT, and 2.6.4.4.3-4CT, provide guidance on the bottom of abutment/pier stems (top of footing/pile cap, as applicable) placement.

The practice also addresses foundations on piles or drilled shafts (deep foundations) without projecting footings/piles caps, such as integral abutments. This foundation type is not specifically addressed by HEC-18. The placement of the bottom of the stem is consistent with foundations on piles or drilled shafts with footings/piles caps.

The placement of shallow foundations on spread footings is more conservative than the requirements in Section 2.2, Page 2.4, Item 1 entitled "Spread Footings on Soils – Piers" and Item 2 "Spread Footings on Soils – Abutments" in HEC-18.

For additional information for foundations supported on scour resistant rock, refer to HEC-18, Section 2.2, Page 2.5, Item 3 entitled "Spread Footings on Rock Highly Resistant to Scour".

Placing the bottom of retaining wall stem (top of footing/pile cap, as applicable) below (outside) the total scour ensures that the soil behind the stem will not be scoured and unavailable to support the abutment.

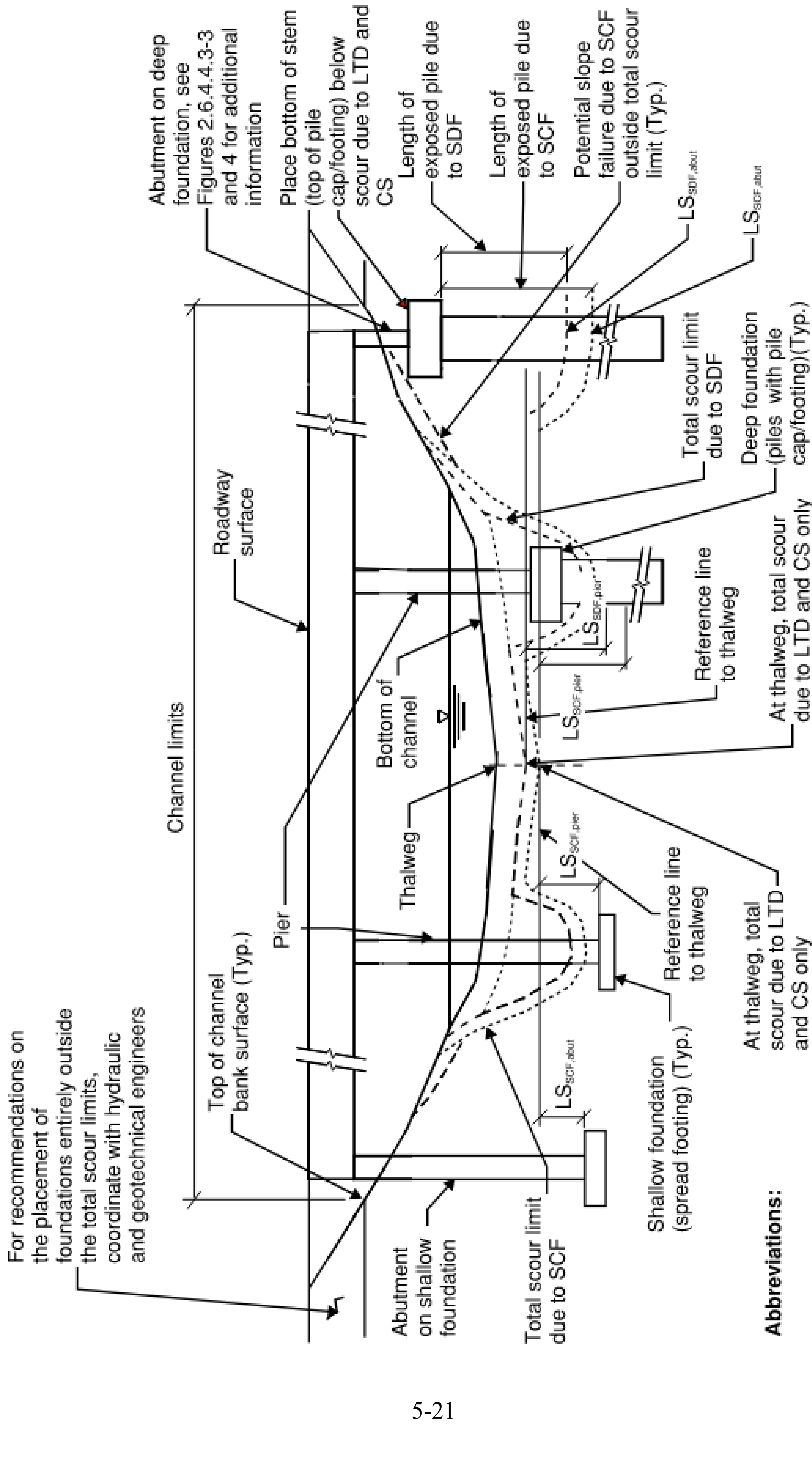
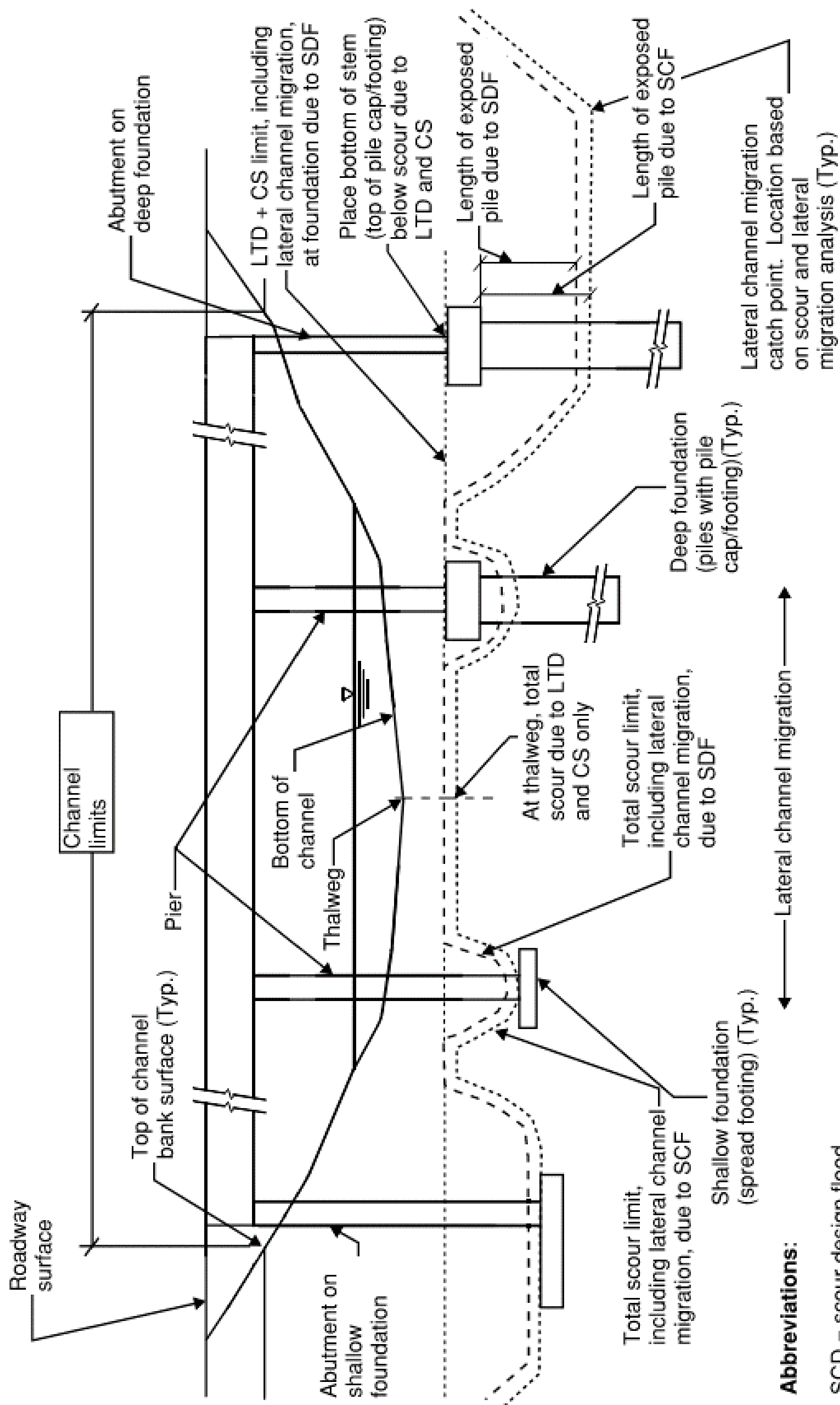


Figure 2.6.4.4.3-1CT
FOUNDATION SUPPORT AND PLACEMENT
For scour without lateral migration

Abbreviations:

SCD = scour design flood
SCF = scour check flood

LTD = long-term degradation
CS = contraction scour
LS = local scour

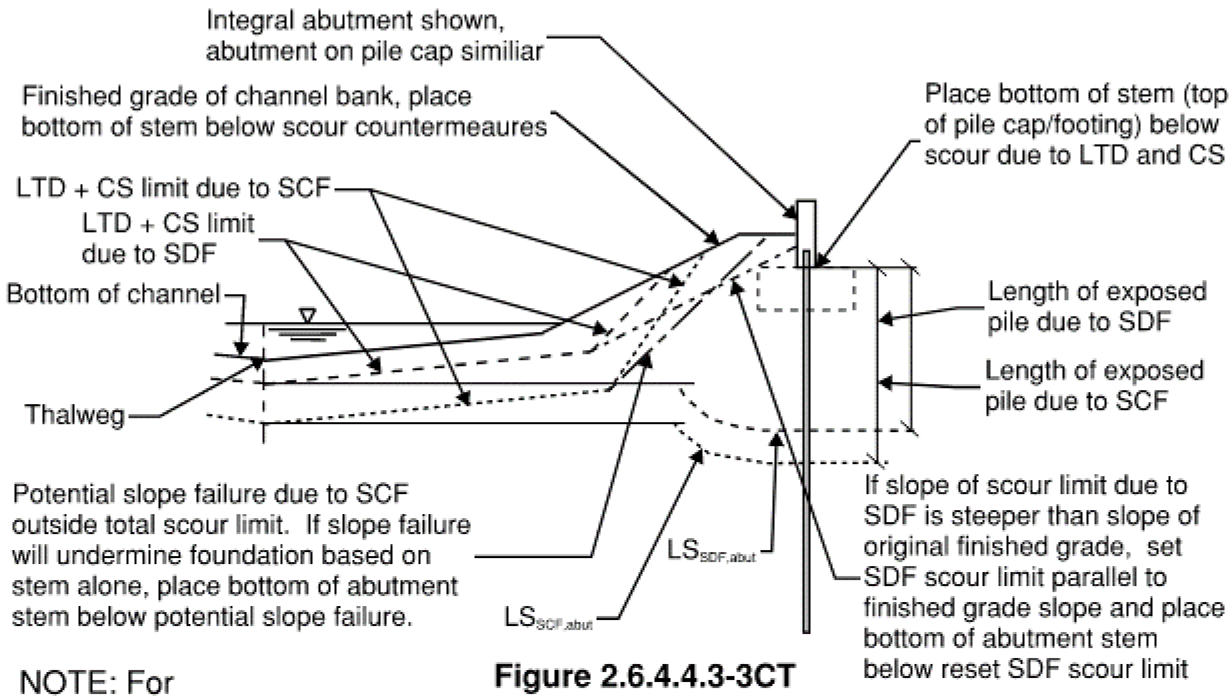


Abbreviations:

SCD = scour design flood
SCF = scour check flood

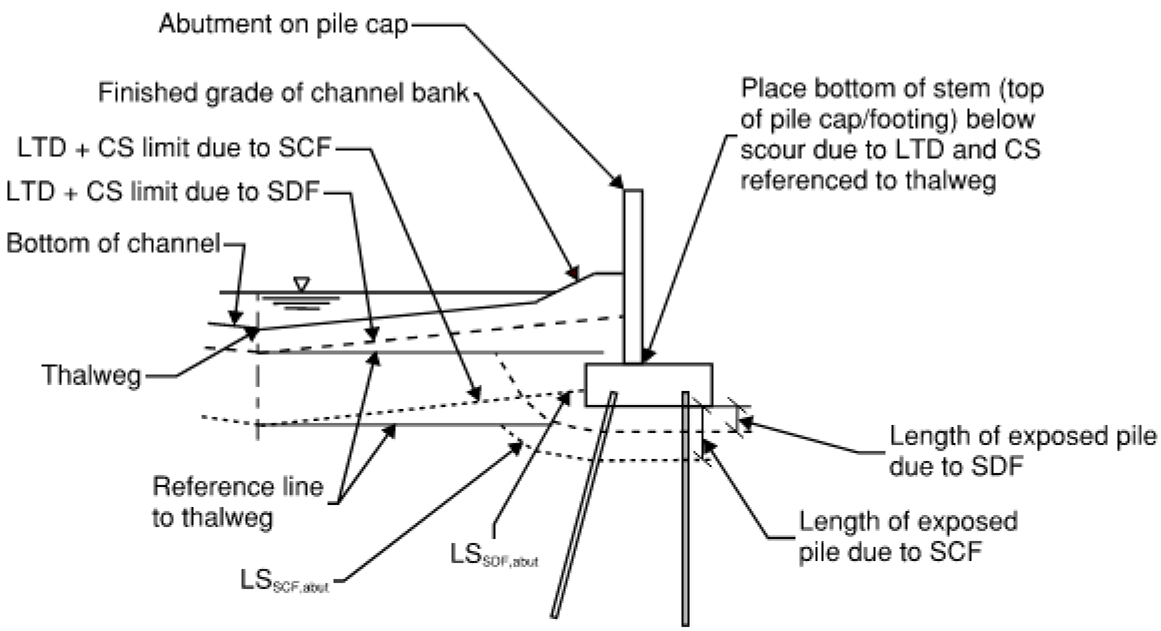
LTD = long-term degradation
CS = contraction scour
LS = local scour

Figure 2.6.4.4.3-2CT
FOUNDATION SUPPORT AND PLACEMENT
For scour with lateral migration



NOTE: For abbreviations, see Figure 2.6.4.4.3-1CT.

Figure 2.6.4.4.3-3CT
FOUNDATION SUPPORT AND PLACEMENT
For embankment abutment on deep foundation and scour without lateral migration



NOTE: For abbreviations, see Figure 2.6.4.4.3-1CT.

Figure 2.6.4.4.3-4CT
FOUNDATION SUPPORT AND PLACEMENT
For full height abutment on deep foundation and scour without lateral migration

2.6.4.4.4CT—Foundation Evaluation for Limit States

Foundations subject to scour shall meet and satisfy the applicable limit states for conditions with and without scour, and with aggradation.

Foundations shall be evaluated for the following conditions and satisfy the following limit states:

- For changes in foundation conditions resulting from a scour design flood, both during and after the flood event, bridges and walls shall be evaluated and satisfy the Strength, Service and Extreme Event limit states in Table 2.6.4.4.4-1CT assuming all the streambed material above the combined component scour has been removed and is unavailable for foundation support.
- For changes in foundation conditions resulting from a scour check flood, both during and after the flood event, bridges and walls shall be evaluated and satisfy the Extreme Event III limit state in Table 2.6.4.4.4-1CT assuming all the streambed material above the combined component scour has been removed and is unavailable for foundation support.
- For changes in foundation conditions resulting the yearly mean discharge flood event, bridges and walls shall be evaluated and satisfy the Extreme Event II limit state in Table 2.6.4.4.4-1CT assuming all the streambed material above the combined component scour has been removed and is unavailable for foundation support.

For Extreme Event I limit state, the 100%-100%-0% contribution from long-term degradation, contraction scour, and local scour, respectively addresses the following:

- A conservative assumption that a 975-year seismic event can occur near the end of an anticipated 75-year service life of a bridge when the full magnitude of long-term degradation is attained.
- The full magnitude of contraction scour would occur under the first 100-year flood event and retain the calculated value thereafter.
- 0% local scour assumes the holes refill shortly after the scour design flood event and is not considered probable in conjunction with 100% long-term degradation, 100% contraction scour, and a design seismic event.

The scour combination for Extreme Event II limit state for CV has been adapted from the *AASHTO Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges* (2009).

Table 2.6.4.4.4-1CT – Combination of Scour Components for Different Limit States (in %)

Limit State	Long-term Degradation	Contraction Scour	Local Scour	Flood
Service	100	100	100	Scour Design Flood
Strength	100	100	100	Scour Design Flood
Extreme Event I	100	100	0	Scour Design Flood
Extreme Event II, Case A1 for CV	50	50	50	Scour Design Flood
Extreme Event II, Case A2 for IC	50	50	50	Scour Design Flood
Extreme Event II, Case B for CV	50	0	0	Yearly mean discharge
Extreme Event III	100	100	100	Scour Check Flood

For the design of bridges for scour conditions, the operational importance shall be independent of the bridge's classification (critical/essential/typical) for the strength limit state. The factor related to operational importance, η_I , shall be taken as 1.05.

Since the revised factor related to operational importance affects the strength limit state, it only applies to changes in foundation conditions resulting from a scour design flood.

For the evaluation with scour conditions, at abutments, walls, and other structures subject to earth load effects, designers shall assume the earth may both remain and not remain in contact with and act upon the rear face of the stems, footings/pile caps

Scour conditions may result in the removal of only a portion of the streambed or adjacent embankment/backfill at a foundation. Load effects on foundations from all possible scour and drilled shafts/piles. Load cases shall include both balanced and unbalanced loading conditions.

The design vehicle live load shall include dynamic load allowance for load cases where the foundation components are no longer surrounded by soil because of scour.

The load effects of water shall be based on the elevations and velocities associated with the scour design flood and scour check flood.

Deep foundation design, for assessing overburden stress used for bearing resistance calculations, shall consider the effect of the lost soil due to scour as shown in Figure 2.6.4.4.4-1CT. Similarly, for shallow foundations (i.e., spread footings) located below Point C in the figure, the overburden stress used for assessing bearing resistance after scour should be calculated as shown in this figure.

Whenever total scour depth exposes deep foundation elements, the foundation evaluation shall also consider the potential for damage due to erosion, debris impacts, wood borers, corrosion from exposure to stream currents, or other environmental effects.

conditions shall be investigated. At the drilled shafts/piles soil arching should be considered.

The design vehicle live load is defined by Article 3.6.1.2.

The requirement for applying the dynamic load allowance to the design vehicle live load assumes that foundation components may no longer be surrounded by soil because of scour.

Shown in Figure 2.6.4.4.4-1CT is a simplification that can be used to calculate the overburden stress needed for foundation bearing resistance calculations for Service, Strength, and Extreme Event limit states. If a more accurate estimate of overburden stresses is needed, complex three-dimensional modeling would be required, and such modeling may be considered for use in foundation design subject to owner approval.

Additional design requirements for deep foundations with regard to scour are provided in Article 10.5.5.3.2 and 10.7.3.6, plus commentary.

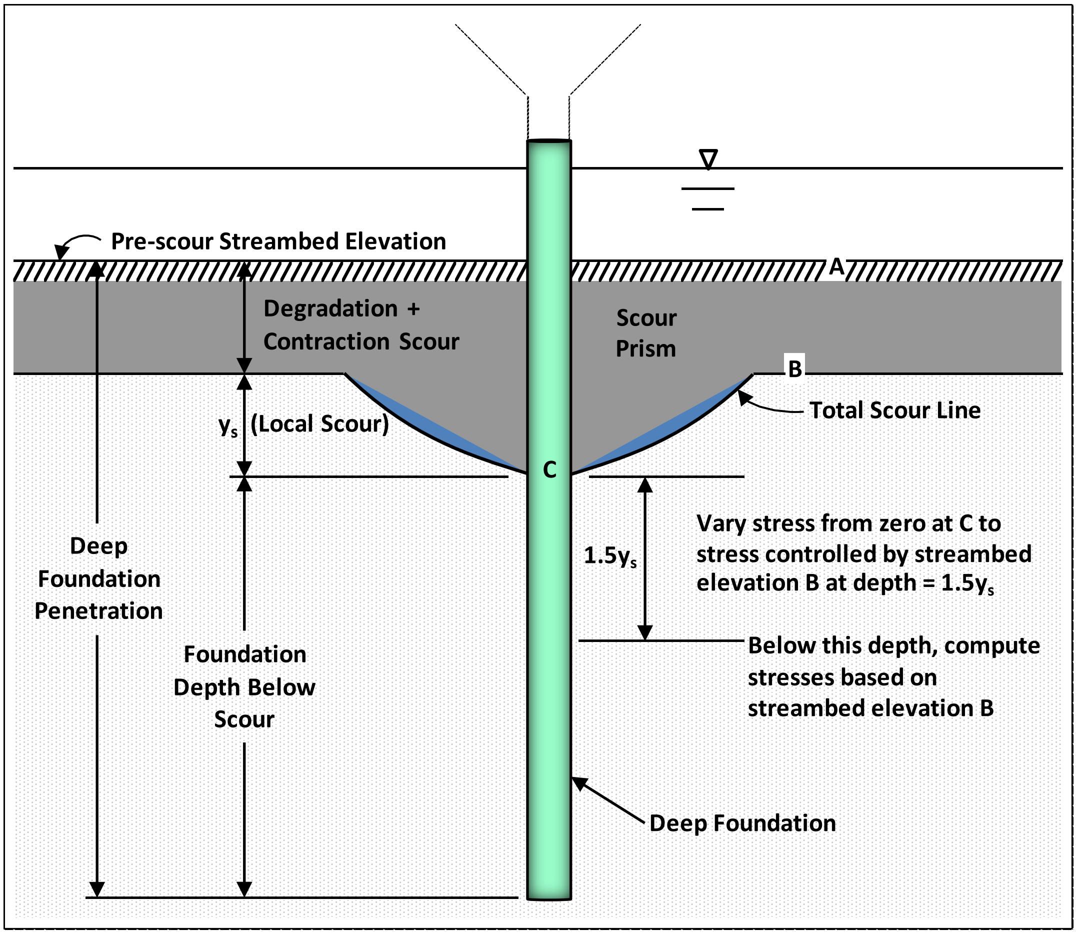


Figure 2.6.4.4.4-1CT Illustration of Scour Prism and its Effects on Deep Foundations (adapted FHWA-NHI-18-024 ,GEC10 - Drilled Shaft Manual)

2.6.4.4.5CT—Scour Countermeasures

Scour countermeasures shall meet the requirements of the CTDOT Drainage Manual.

For new bridges and new walls retaining highways, changes in foundation conditions resulting from a scour design flood and scour check flood shall be determined without the benefits provided by scour countermeasures.

For existing bridges and walls retaining highways undergoing rehabilitation where the existing substructure and walls will be retained, if the existing foundations do not meet the

C2.6.4.4.4CT

The limits, placement and details of scour countermeasures shall meet the requirements of the CTDOT Drainage Manual. The manual refers to the use of the document *Bridge Scour and Stream Instability Countermeasures: Experience, Selection, and Design Guidance*, 3rd Edition, dated September 2009, FHWA-NHI-09-112, Hydraulic Engineering Circular No. 23 (HEC-23) for additional guidance on countermeasure selection and design. For additional information is also included in the *Technical Brief (TechBrief)* FHWA-HIF-19-007.

For new bridges and new walls retaining highways, the use of revetments, armoring, permanent steel sheet piling, or permanent cofferdams, to reduce scour impact and foundation depths may only be permitted if the placement of the foundations to meet Article 2.6.4.4.3CT is proven not constructable due to the project constraints.

design criteria for new bridges and new walls retaining highways, scour countermeasures may be used to mitigate scour.

When fendering or other pier protection systems are used, their effect on pier scour and collection of debris shall be taken into consideration in the design. Since scour prediction equations are not available for this scenario, the structural, hydraulic, and geotechnical aspects of the design, based on advanced modeling, local experience and engineering judgment, shall be coordinated and differences resolved prior to implementation of fendering and other pier protection methodologies.

Advanced three-dimensional modeling may be needed to assess the effect of fendering or other pier protection systems on scour.

2.6.4.4.6CT—Existing Substructure Components

For bridge replacement projects with foundations adjacent to or within waterways, the extent of the removal of existing substructure components and elements, either wholly or partially, must be addressed during the design phase. The existing substructure components and elements may include abutment stems, wall stems, pier stems, footings, pile caps, and piles.

Considerations for leaving existing substructure components and elements, either wholly or partially, in place include efforts to minimize construction costs, facilitate water handling, simplify construction, reduce construction duration, limit hydraulic affects, or reduce environmental impacts. These efforts are constrained by the need to ensure that leaving existing substructure components and elements in place can be accurately reflected in both the hydraulic and scour analysis, will be environmentally permittable, and any future changes in the position of the components will not exacerbate conditions due to any event that result in an unacceptable hydraulic condition, scour conditions worse than the original design, or have a negative environmental impact.

The removal limits of existing substructure components and elements shall be coordinated, developed, and justified with a multi-discipline group of structural, hydraulic, geotechnical and environmental engineers along with representatives from the CTDOT Office of Environmental Planning. The removal limits shall be clearly specified in the contract documents and shall agree with the final scour evaluation report, the final hydraulics report, and the environmental permits.

New work shall be independent of the existing component parts that remain in place and shall not rely on existing component parts to provide any structural benefit.

2.6.4.5—Roadway Approaches to Bridge

This section shall be supplemented with the following:

Retaining walls shall be designed for scour as specified in Articles 11.6.3.4, 11.7.2.3, 11.10.1, and 11.10.2.2.

C2.6.4.5

This section shall be deleted and replaced with the with the following:

Highway embankments on floodplains serve to redirect overbank flow, causing it to flow generally parallel to the embankment and return to the main channel at the bridge~~.~~ Roadway embankment and retaining wall designs should include countermeasures where necessary to limit damage caused by overbank flow parallel to the embankment. Such countermeasures may include:

- relief bridges, culverts, or other structural openings,
- retarding the velocity of the overbank flow by promoting growth of trees and shrubs on the floodplain and highway embankment within the highway right-of-way or constructing small dikes along the highway embankment,
- protecting fill slopes subject to erosive velocities by use of riprap or other erosion protection materials on highway fills and spill-through abutments, and
- where overbank flow is large, utilize guide banks to protect abutments of main channel and relief bridges from turbulence and resulting scour.

Additional information and design guidelines on scour countermeasures are provided in HEC-23.

Although overtopping may result in failure of the embankment, this consequence is preferred to failure of the bridge. The low point of the overtopping section should not be located immediately adjacent to the bridge, because its failure at this location could cause damage to the bridge abutment. If the low point of the overtopping section must be located close to the abutment, due to geometric constraints, the scouring effect of the overtopping flow should be considered in the design of the abutment. Design studies for overtopping should also include evaluation of any flood hazards created by changes to existing flood flow patterns or by flow concentrations in the vicinity of developed properties.

Bridge approach embankment slopes exposed to scour should be protected with properly designed scour countermeasures designed in accordance with HEC-23 where possible, considering any regulatory requirements.

The risk of bridge approach fill failure due to scour may be an acceptable risk as the approach fill typically can be replaced quickly to restore access to the bridge crossing. The impact of such approach fill loss to bridge approach structures such as wing walls, bridge approach slabs, and small (i.e., short in height and length) retaining walls that support the approach embankment side slopes will need to be considered. This is especially important if significant stream channel migration risk is not low, as much more of the embankment could be affected, or, as illustrated in HEC-18, Figure 8.7(c), the bridge abutment could become like an intermediate bridge pier with regard to increased scour depth due to local and contraction scour.

The length of bridge approach embankment or wall relative to the bridge abutment location that can be affected by scour, and how deep the scour is likely to occur, will depend on several factors, including the length of the approach embankment within the floodplain and the potential for stream migration. For the portion of the approach retaining wall up on the flood plain (i.e., outside the main channel), scour due to long-term degradation is no longer applicable, and only contraction scour and local scour should be considered to locate the wall footing or wall base.

Section 3 shall be supplemented as follows:

Revisions to Section 3 are needed due to the changes to Articles 2.6.4.4.1, 2.6.4.4.2, 2.6.4.4.3CT and 2.6.4.4.4CT.

Revise the 7th bullet and insert a new 8th bullet in Article 3.4.1 as follows:

- Extreme Event II—Load combination relating to ice load, blast load, and collision by vessels and vehicles.
- Extreme Event III- Load combination for design considering the effects of check flood scour condition.

Revise the 4th and 5th bullets in the bullet list in Article C3.4 as follows:

- Although these limit states include water loads, *WA*, the effects due to *WA* are considerably less significant than the effects of changes to foundation condition due to scour. Article 2.6.4.4.4CT addresses the effects of scour combined with extreme event limit states.
- The joint probability of *BL, EQ, CT, CV,* and *IC* is extremely low, and, therefore, the events are specified to be applied separately. Under these extreme conditions, the structure may undergo considerable inelastic deformation by which locked-in force effects due to *TU*, *TG*, *CR*, *SH*, and *SE* are expected to be relieved.

Add as the new 2nd paragraph of Article C3.4.1 as follows:

Design for the scour check flood has been included in the Extreme Event III limit state to highlight the loads that will act on the bridge during such events. Furthermore, conditions of the foundations under scour check flood are evaluated to consider any reduction in geotechnical resistance and stiffness due to scour.

The Extreme Event limit states in Table 3.4.1-1 shall be revised as follows:

Add the following to Article C3.4.1:

Changes in foundation conditions resulting from a scour check flood shall be considered at the Extreme Event III limit state. The load factor LL, IM, CE, BR, PL, and LS is specified as 1.00 to ensure that the bridge can remain operational until the extent of any damage can be evaluated and repaired. PennDOT has a similar requirement. Consider a case where a flood event damages the highway approach to a multi-span bridge and scours material around abutment and pier foundations to a depth greater than the depth calculated for a scour design flood but less than a depth calculated for a scour check flood. By quickly rebuilding the highway approach the bridge can be opened for at least limited use by a design vehicle live load until the extent of any damage at the abutments and piers can be evaluated and repaired.

Table 3.4.1-1 – Load Combination and Load Factors – Revised Extreme Event Limit States for Scour

Load Combination Limit State	*DC DD DW EH EV ES EL PS CR SH*	*LL IM CE BR PL LS*	*WA*	*WS*	*WL*	*FR*	*TU*	*TG*	*SE*	Use One of These at a Time				
										EQ	*BL*	*IC*	*CT*	*CV*

Extreme Event I [1]	1.00	γ_{EQ}	1.00	—	—	1.00	—	—	—	1.00	—	—	—	—
Extreme Event II [1]	1.00	0.50/1.00	1.00	—	—	1.00	—	—	—	—	1.00	1.00	1.00	1.00
Extreme Event II, Case A1 [1]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	—	—	1.00
Extreme Event II, Case A2 [1]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	1.00	—	—
Extreme Event II, Case B [2]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	—	—	1.00
Extreme Event III [3]	1.00	1.00	1.00	—	—	1.00	—	—	—	—	—	—	—	—
Notes: 1 - Flood event = Scour Design Flood 2 - Flood event = Yearly mean discharge 3 - Flood event = Scour Check Flood														

Delete Articles 3.7.5 and C3.7.5

Add the following as a new paragraph and bulleted list following the last paragraph of Article 3.14.1:

Evaluation of the following vessel collision events shall be combined with foundation conditions due to scour.

- Case A - A drifting empty barge breaking loose from its moorings and striking the bridge.
- Case B - A ship or barge tow striking the bridge while transiting the navigation channel under typical waterway conditions.

Delete the 13th paragraph and bulleted list that follows in Article C3.14.1

Section 10 shall be supplemented as follows:

Revisions to Section 10 are needed due to the changes to Articles 2.6.4.4.1, 2.6.4.4.2, 2.6.4.4.3CT and 2.6.4.4.4CT.

Revise the 1st paragraph in Article 10.4.6.6 as follows:

Consideration should be given to the physical characteristics of the rock and the condition of the rock mass when determining a rock's susceptibility to erosion in the vicinity of bridge foundations. Physical characteristics that should be considered in the assessment of erodibility include cementing agents, mineralogy, unconfined compressive strength, rock quality designation (RQD), joint spacing and orientation, joint roughness and alteration, and weathering.

Revise the 1st paragraph in Article C10.4.6.6 as follows:

There is no consensus on how to determine erodibility of rock masses near bridge foundations. Refer to HEC-18 (FHWA, 2012) and Arneson et al. (2012) when determining the potential for a rock mass to scour.

Add as the new 3rd paragraph in Article 10.5.1 as follows:

Changes in foundation conditions resulting from scour, as specified in Article 2.6.4.4.4CT, shall be considered.

Revise the 3rd bullet in the list of Article 10.5.2.1 as follows:

- scour design flood

Revise the 3rd paragraph in C10.5.2.1 as follows:

The scour design flood is defined in Article 2.6.4.4 and its consideration under the service limit state specified in Article 2.6.4.4.4CT.

Revise the 2nd bullet in the list of Article 10.5.3.1 as follows:

- loss of lateral and vertical support due to scour, and

Revise the 4th paragraph of Article C10.5.3.1 as follow:

The scour design flood is defined in Article 2.6.4.4. and its consideration under the strength limit state is specified in Article 2.6.4.4.4CT.

Revise Article C10.5.4.1 as follows:

Extreme events include scour check flood, vessel and vehicle collision, seismic loading, and other site-specific situations that the Engineer determines should be included. Appendix A10 gives additional guidance regarding seismic analysis and design.

Revise the 2nd paragraph of Article 10.5.5.1 as follows:

A resistance factor of 1.0 shall be used to assess the ability of the foundation to meet the specified deflection criteria after soil removal due to the scour design flood.

Revise the 3rd paragraph of Article 10.5.5.2.1 as follows:

The foundation resistance after the scour design flood shall provide adequate foundation resistance using the resistance factors given in this Article. The resistance factors shall be those used in the Strength Limit State, without scour.

Revise the 7th paragraph of Article C10.5.5.2.1 as follow:

Design for the scour design flood must satisfy the requirement that the factored foundation resistance is greater than the factored load determined with the scoured soil removed.

Revise the title of Article 10.5.5.3 as follows:

10.5.5.3—Extreme Event Limit States

Revise the 1st paragraph of Article 10.5.5.3.2 as follows:

The provisions of Article 2.6.4.4.4CT shall apply to the changed foundation conditions resulting from scour. Resistance factors at the extreme event shall be taken as 1.0 except that for uplift resistance of piles and shafts, the resistance factor shall be taken as 0.80 or less.

Revise Article C10.5.5.3.2 as follows:

The specified resistance factors should be used provided that the method used to compute the nominal resistance does not exhibit bias that is unconservative. See Article C10.7.3.6 for guidance regarding bias values for pile resistance prediction methods.

Design for scour is discussed in Hannigan et al. (2016).

Revise the 1st paragraph of Article 10.6.1.2 as follows:

Revise the 2nd paragraph of Article C10.6.1.2 as follows:

Where the potential for scour, erosion or undermining exists, shallow foundations (i.e., spread footings) shall be located such that the top of the footing is located at or below the maximum anticipated depth of scour, erosion, or undermining as specified in Article 2.6.4.4.3CT.

For spread footings founded on excavated or blasted rock, attention should be paid to the effect of excavation and/or blasting. Blasting of highly resistant competent rock formations may result in overbreak and fracturing of the rock to some depth below the bearing elevation. Blasting may reduce the resistance to scour within the zone of overbreak or fracturing. See Article 10.4.6.6 regarding factors affecting erodibility of rock.

Revise the 1st and 2nd paragraphs of Article 10.7.3.6 as follows:

The effect of scour shall be considered in determining the minimum pile embedment and the required nominal driving resistance, R_{ndr}. The pile foundation shall be designed so that the pile penetration after soil has been removed due to the scour design floods satisfies the required nominal axial and lateral resistance.

The resistance factors shall be those used in the design without scour. The side resistance of the material lost due to scour should be determined using a static analysis and it should not be factored, but consideration should be given to the bias of the static analysis method used to predict resistance.

Revise Article C10.7.3.6 as follows:

The piles will need to be driven to the required nominal bearing resistance plus the side resistance that will be lost due to scour. The nominal resistance of the remaining soil is determined through field verification. The pile is driven to the required nominal bearing resistance plus the magnitude of the side resistance lost as a result of scour, considering the prediction method bias. Bias is defined as the measured/predicted value of resistance, in this case pile side resistance. Typically, the average method bias, based on available databases should be used. This bias can be based on a national database, or based on a local database, if enough measurements are available to reliably establish an average value. Example bias values for various pile static resistance prediction methods based on a national database are provided in Paikowsky et al. (2004) and Allen (2005).

To use bias values to adjust pile resistance predictions, since a bias greater than 1.0 means the method predicts less resistance than is actually present and a bias less than 1.0 means that the method predicts more resistance than is actually present, the bias adjusted resistance is determined by multiplying the resistance lost due to scour by the bias value. Since in this case the goal is to estimate lost resistance due to scour, a conservative estimate is obtained when the method predicts more resistance than is actually present.

Another approach that may be used takes advantage of dynamic measurements. In this case, the static analysis method is used to determine an estimated length. During the driving of test piles, the side resistance component of the bearing resistance of pile in the scourable material may be determined by a signal matching analysis of the restrike dynamic measurements obtained when the pile tip is below the scour elevation. The material below the scour elevation must provide the required nominal resistance after scour occurs.

In some cases, the flooding stream will carry debris such as wood or ice that will induce horizontal loads on the piles.

Additional information regarding pile design for scour is provided in Hannigan et al. (2016).

Revise the 3rd paragraph of Article 10.7.4 as follows:

When designing for scour, the pile foundation design shall be conducted as described in Article 10.7.3.6, except that the scour resulting from the check flood and resistance factors consistent with Article 10.5.5.3.2 shall be used.

Revise the last bullet in the list of Article 10.7.7 as follows:

- extreme event limit state nominal bearing resistance considering scour as specified in Article 10.7.4.

Revise Article 10.9.3.3 as follows:

10.9.3.3—Scour

The provisions of Article 10.7.3.6 shall apply.

Revise C10.9.3.3 as follows:

See Article C10.7.3.6.

Section 11 shall be supplemented as follows:

Revisions to Section 11 are needed due to the changes to Articles 2.6.4.4.1, 2.6.4.4.2, 2.6.4.4.3CT and 2.6.4.4.4CT.

Revise the 1st paragraph in Article 11.6.3.4 as follows:

For walls constructed along rivers and streams, scour of foundation materials shall be evaluated during design, as specified in Article 2.6. Where potential problem conditions are anticipated, adequate protective measures shall be incorporated in the design, including, but not limited to, locating the top of the wall footing below the scour depth determined in accordance with Articles 2.6.4.4.3CT and 2.6.4.5 or adding scour countermeasures to protect the wall footing.

Revise the 1st paragraph in Article 11.7.2.3 as follows:

The scour potential shall be determined, and the design shall be developed to prevent failure from this condition as specified in Articles 2.6.4.4.3CT and 2.6.4.5.

Revise the 4th paragraph in Article C11.10.1 as follows:

The potential for catastrophic failure due to scour is high for MSE walls if the reinforced fill is lost during a scour occurrence. Consideration should be given to lowering the base of the wall or to alternative methods of scour protection, such as sheet pile walls and/or riprap of sufficient size, placed to a sufficient depth to preclude scour.

Revise the 3rd paragraph in Article 11.10.2.2 as follows:

For walls constructed along rivers and streams, Article 11.6.3.4 applies, except that the embedment depths shall be established at a minimum of 2.0 ft below potential scour depth.

Revise the 2nd paragraph in Article 11.11.4.1 as follows:

Passive pressures shall be neglected in stability computations, unless the base of the wall extends below the scour depth as specified in Article 11.6.3.4, freeze–thaw, or other disturbance. For these cases only, the embedment below the greater of these depths may be considered effective in providing passive resistance.

Revise Article 11.11.4.5 as follows:

Bin walls may be used in scour-sensitive areas only where their suitability has been established. The provisions of Article 11.6.3.4 shall apply.

5.14.2 Spread Footings on Soil *(Rev. 01/09)*

The top of all footings in soil should be a minimum of 12 inches below the finished grade. The bottom of all footings in soil shall not be less than 4 feet below, measured normal to the finished grade.

Generally, the use of footing keys to develop passive pressure against sliding is not allowed. The use of passive earth pressure along the sides of foundations to prevent sliding is also not allowed. Resistance from sliding shall be attained through friction between the foundation and the supporting material.

The contract shall show the following:

- The maximum design foundation pressure for the controlling Strength and Service Limit States.

  Maximum Design Foundation Pressure = 3.2 TSF (Strength I)
  2.8 TSF (Service I)

- If applicable, also show the maximum design foundation pressure for the Extreme Event Limit State.

  Maximum Design Foundation Pressure = 3.6 TSF (Extreme Event II)

5.14.3 Foundations on Rock

Generally, for cast-in-place construction, gravity-type or semi-gravity L-type sections shall be used for abutments, wingwalls and retaining walls founded on rock. There is no minimum embedment for footings placed on competent rock. Generally, structural underwater concrete is not permitted.

5.14.4 Driven Piles *(Rev. 01/09)*

Piles may be either end bearing or friction or a combination of the two. Piles end bearing on bedrock or dense hardpan typically are steel H-piles. Piles driven through a high compacted fill or into native soil containing numerous boulders and cobbles shall be steel H-piles. Generally, H-piles shall conform to the requirements of ASTM A709 Grade 50. Friction piles shall be used for most other cases. Generally, friction piles are precast concrete, cast-in-place concrete or prestressed concrete. Timber piles are not permitted.

Piles may be installed vertical or battered. The path of battered piles should be checked to insure the piles remain within the right of way and do not interfere with piles from adjacent and existing substructure units, nor conflict with temporary sheeting or cofferdams.

Pile foundations supporting abutments shall have a minimum of two rows of piles, unless the piles are incorporated into a fully integral abutment. Piles shall be anchored to and embedded in the footings a minimum of 12 inches.

Maximum pile spacing and maximum nominal resistance per pile should be utilized to minimize the number of piles. The lateral resistance of a pile pattern is the combination of the lateral component of the force acting on the battered piles and the lateral resistance of each pile, vertical and battered, in the pattern.

Test piles are typically required to establish pile order lengths and pile capacity for friction piles. If pile driving records and pile load test data are available for a site (e.g. a bridge widening where the same pile type is proposed), specifying the pile order length in the contract may be considered. Test piles with dynamic monitoring will still be required to establish the ultimate pile capacity. Static pile load tests may also be considered.

Pile order lengths for end bearing piles on bedrock should be provided in the contract when there is confidence in the subsurface profile. If a test pile(s) is being considered to establish order lengths for end bearing piles on bedrock, the benefit of the refined pile order length shall be weighed against the time required to obtain the production piles. Test piles with dynamic monitoring and/or static load tests may still be required to establish the ultimate pile capacity. The need for test piles is evaluated by the geotechnical engineer and should be included in the Geotechnical Report.

Readily available pile types should be used whenever possible; especially if the order length cannot be established until after test piles are driven. The location of test piles and load tests should be in areas that are readily accessible, and allow enough room for the Contractor to perform the work. A preconstruction test pile program may be considered on large projects where a benefit can be realized by establishing pile type(s) and pile capacity during the design phase.

If no test piles are specified for a given substructure, the estimated pile length shall be used as the pile order length. For these cases, the estimated pile length should be increased slightly to insure there is sufficient length.

The top of all footings should be a minimum of 12 inches below the finished grade. The bottom of all pile caps shall not be less than 4 feet below, measured normal to the finished grade.

For H-piles, pile point reinforcement and splices shall be prefabricated. The contract shall specify that the Contractor is required to submit pile point reinforcement and splice details to the Engineer for review and approval.

This following note shall be included on the contract drawings:

Prior to driving the piles, the Contractor shall submit to the Engineer for review and approval his method and sequence of pile driving.

The pile plan(s) included in the contract drawings shall show or note the following:

- A legend denoting vertical, battered and test piles.
- The number, location and length of test piles, if applicable.
- The location of load test pile(s), if applicable.
- The location of dynamic monitoring (pda) pile(s), if applicable.
- The number, location and estimated length for vertical and battered production piles. If no test pile is specified or if the estimated pile length is intended to be used as the order length, the following note shall be included:

The estimated pile length(s) is to be used as the pile order length.

- Material designation of piles, including pile point reinforcement and splices.
- The maximum design pile load for the controlling Strength and Service Limit States for each foundation section.

Maximum Design Pile Load =	57 Tons (Strength I)
	55 Tons (Service I)

If applicable, also show the maximum design pile load for the Extreme Event Limit State.

Maximum Design Pile Load =	67 Tons (Extreme Event II)

- The ultimate pile capacity for each foundation section, as defined as:
  Ultimate Pile Capacity = (Factored Design Load)/φ + Scour + Downdrag

  Scour= The estimated skin friction resistance of the soil above the predicted scour depths.

Downdrag= The estimated side friction resistance of a compressible soil above the neutral point (determined when computing the downdrag load (DD) due to settlement).

Φ = The resistance factor based on the design load limit state, resistance determination method, and required field testing.

Example:

ULTIMATE PILE CAPACITY	
Abutment 1	XX tons
Pier No. X	XX tons
Abutment 2	XX tons

The Scour and Downdrag resistance along with Φ will be determined by the geotechnical engineer and included in the geotechnical report. The Contractor will use the Ultimate Pile Capacity to properly size pile driving equipment and load testing apparatus. The Engineer will use the Ultimate Pile Capacity to establish the required driving resistance and validate load test results.

5.14.5 Drilled Shafts

Vacant

5.15 EARTH RETAINING SYSTEMS AND COFFERDAMS *(Rev. 01/09)*

5.15.1 Highway Applications *(Rev. 01/09)*

The location and limits, both horizontal and vertical, of all temporary and permanent earth retaining system contract items shall be shown at each location.

5.15.1.1 Permanent Steel Sheet Piling *(Rev. 01/09)*

Permanent Steel Sheet Piling is defined as a required and permanent structural element integral to the design of the structure. Permanent Steel Sheet Piling is designed and engineered by the Designer. To avoid unnecessary proprietary specificity, permanent steel sheet piling should be specified and designated by AASHTO or ASTM material classification and minimum required section modulus.

5.15.1.2 Temporary Earth Retaining Systems *(Rev. 01/09)*

Temporary earth retaining system shall be any type of adequately braced temporary retaining wall which the Contractor elects to build to satisfy, and which does satisfy, the condition that existing facilities be properly retained during excavation or fill for the placement of substructure or other facilities. A Temporary Earth Retaining System shall be designated in the contract to be left in place only if its removal may be detrimental to the structure. The item "Earth Retaining System Left in Place" shall be used only for a Temporary Earth Retaining System designated by the Designer to be left in place. A Temporary Earth Retaining System requested by the Contractor to be left in place for his own convenience is not compensable for additional payment.

5.15.2 Railroad Applications *(Rev. 01/09)*

The location and limits, both horizontal and vertical, of all temporary and permanent earth support systems necessary for the construction of railroad structures must be shown in the contract. All contract items for temporary and/or permanent earth support systems for railroad structures and facilities must be submitted to the affected railroad for review during the standard project design submission phases. Contract plan details for temporary earth support in railroad applications must be specific in name and include a railroad parenthetical in the contract item name. Typical items for such use are "Temporary Sheet Piling (Railroad)" and "Soldier Pile and Lagging (Railroad)". The Designer should select the most appropriate temporary retaining system type in consideration of existing soil conditions and construction access limitations.

5.15.2.1 Permanent Steel Sheet Piling *(Rev. 01/09)*

Permanent Steel Sheet Piling adjacent to railroad tracks shall be designed for each specific location and shown in the contract. It should be specified and designated as noted in **BDM** [5.15.2] above.

5.15.2.2 Temporary Earth Support Systems *(Rev. 01/09)*

Through communication with the affected railroad company during a project's scope development, a determination will be made whether or not a complete design for a temporary earth support system is required. When the temporary earth support system is fully designed by the Designer, the contract special provision for the subject item should allow for the Contractor to submit an alternate design.

The Designer shall evaluate the global stability of the railroad embankment for the temporary (and permanent) condition. In cases where the stability of the embankment may be compromised, the Designer shall either provide a complete design or include the necessary constraints for a Contractor to properly design the earth support system.

Working drawings and design calculations prepared by the Contractor shall be submitted to the **CTDOT** and affected railroad company for review.

Items, such as "Sheet Piling left in Place (Railroad)" and "Soldier Pile and Lagging Left in Place (Railroad)" shall be designated in the contract only if their removal may be detrimental to the structure, as determined by the Designer in concert with the affected railroad company. Any system requested to be left in place by the Contractor for his convenience is not compensable.

5.15.3 Water-Handling-Cofferdams and Temporary Water Redirection *(Rev. 01/09)*

Water-Handling-Cofferdams and various methods to temporarily redirect water from the site are used so that construction can take place in the dry. Various methods and items may be used to achieve this end depending on the nature of the site, the nature of the construction, and the amount of water encountered. Because of the need to secure environmental permits, considerable detail of the dewatering plan intended may be required in the contract.

5.15.3.1 Structure Excavation (Complete) *(Rev. 07/04)*

The items "Structure Excavation-Earth (Complete)" or "Structure Excavation-Rock (Complete) are generally used where water intrusion into the excavation results from groundwater seepage or very minor stream or drainage flow. There is no additional payment for dewatering or temporarily diverting water since the work required to construct in the dry is considered to be of such a minor nature that it can be considered to be incidental to the excavation items. Any cofferdams, temporary redirection, pumping, or any other dewatering methodology is included in the cost of the work. Generally, since the impact on water resources is so small, very little detailing is required in the contract.

5.15.3.2 Handling Water *(Rev. 07/04)*

If a structure cannot be constructed in the dry, a cofferdam is needed. The item "Handling Water" is generally used where a temporary redirection of a watercourse is required and is generally used for construction of culverts or retaining walls adjacent to the watercourse. This lump sum item includes any temporary water handling structures such as barriers, temporary pipes, or drainage channels, necessary to complete the work. Also included is any excavation required to accomplish the temporary redirection of surface water.

Any required excavation for the permanent construction will be paid for under the items "Structure Excavation-Earth (Complete)." "Structure Excavation-Rock (Complete)" or appropriate excavation items. A conceptual scheme showing all temporary water handling structures such as barriers, temporary pipes, and drainage channels, and a conceptual scheme for staging of construction for water handling must be shown in the contract and will usually be included in permit applications. The hydraulic design of the aforementioned temporary facilities is based on the watercourse hydrology and information contained in the **Drainage Manual** [6.15]. A temporary design water surface elevation associated with the proposed temporary hydraulic facility should be shown in the contract and the permit plates.

The contractor will be required to submit working drawings to detail the proposal shown in the contract. If the contractor's working drawings differ from the proposal shown on the contract to the extent that a revision to the permit is required, the contractor will be required to prepare and apply for any revisions required to the permit.

5.15.3.3 Cofferdam and Dewatering *(Rev. 07/04)*

The item "Cofferdam and Dewatering" is generally used where substructure elements are located partly or wholly in the streambed and where the bottom of footing is below water level, or on foundation work where considerable flow or concentration of water is present that cannot be conveniently temporarily redirected from the site. The extent of work involved in placing and dewatering the cofferdam is such that it is more than a minor part of the excavation items and is not considered incidental to those items. This item should be used in conjunction with the items "Structure Excavation-Earth (Excluding Cofferdam and Dewatering)" and "Structure Excavation-Rock (Excluding Cofferdam and Dewatering)".

A cofferdam forms an enclosure that may be completely dewatered to allow work in the dry. It may consist of steel sheet piling or any other material the contractor elects to use to satisfy this requirement. Some sides of the cofferdam may be formed by the existing stream bank or by existing structures such as abutments or retaining walls. However, any existing structures intended to be used must extend below the anticipated bottom of excavation and must be resistant to intrusion of significant amounts of water from below the footing.

Cofferdams that encroach into water channels must be hydraulically analyzed based on the watercourse hydrology and information contained in the **Drainage Manual** [6.15]. A temporary design water surface elevation associated with the proposed temporary hydraulic facility should be shown in the contract and the permit plates.

The horizontal and vertical limits of the cofferdam must be shown in the contract. Cofferdams are designed and detailed by the Contractor and submitted to the Engineer for review.