SDG 10:

Bridge Foundation Design

Chapter 10

Tennessee Department of Transportation December 17, 2025

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Version History and Revision Summary

Version history and a summary of revisions are given in table below. Minor editorial revisions may not be included in the summary.

Version	Revision Summary
04272022	Initial Release
10112023	Added Section 8; Added Version History and Revision Summary; Replaced Section 4, Revised Sections 6 and 10-302.02, Revised Figures 3 and 5
12172025	Revised 10-301.01, 10-302.01, and Section 5, Added a new Section 10-302.02 and renumbered subsequent sections

Section 1 Foundation Types

The most common foundation types used in Tennessee include:

- Spread footings
- Pile footings on point bearing piles
- Pile footings on friction piles
- Prestressed concrete pile bents
- Steel pipe pile bents
- Drilled shafts
- Seal footings
- Micropiles

Section 2 Spread Footings

Spread footings may be built on rock or soil. The minimum footing depth for spread footings shall be 3'-0".

The nominal bearing resistance given in the foundation report shall be multiplied by the appropriate resistance factor in AASHTO Table 10.5.5.2.2-1 to determine the factored (i.e., allowable) bearing resistance at the Strength Limit State. The resistance factor at the Extreme Event Limit State is 1.0 per AASHTO 10.5.5.3. Do not use service limit state bearing pressures for the bearing pressure design check. The factored bearing resistance must be greater than the Strength and Extreme Event Limit State bearing pressures. Most spread footings are on rock and therefore have a resistance factor of 0.45 at the Strength Limit State.

For example, if a spread footing on rock is assigned a nominal bearing resistance of 40 ksf in the foundation report, the factored bearing resistance would be 0.45 x 40 ksf = 18 ksf. The maximum Strength Limit State bearing pressure shall not exceed 18 ksf for this example.

All bridge footings are eccentrically loaded. For all spread footings, select the Leap Bridge Concrete option shown in Figure 1 to use AASHTO 10.6.1.3. This article helps mitigate the effects of load eccentricity and uplift in footings.

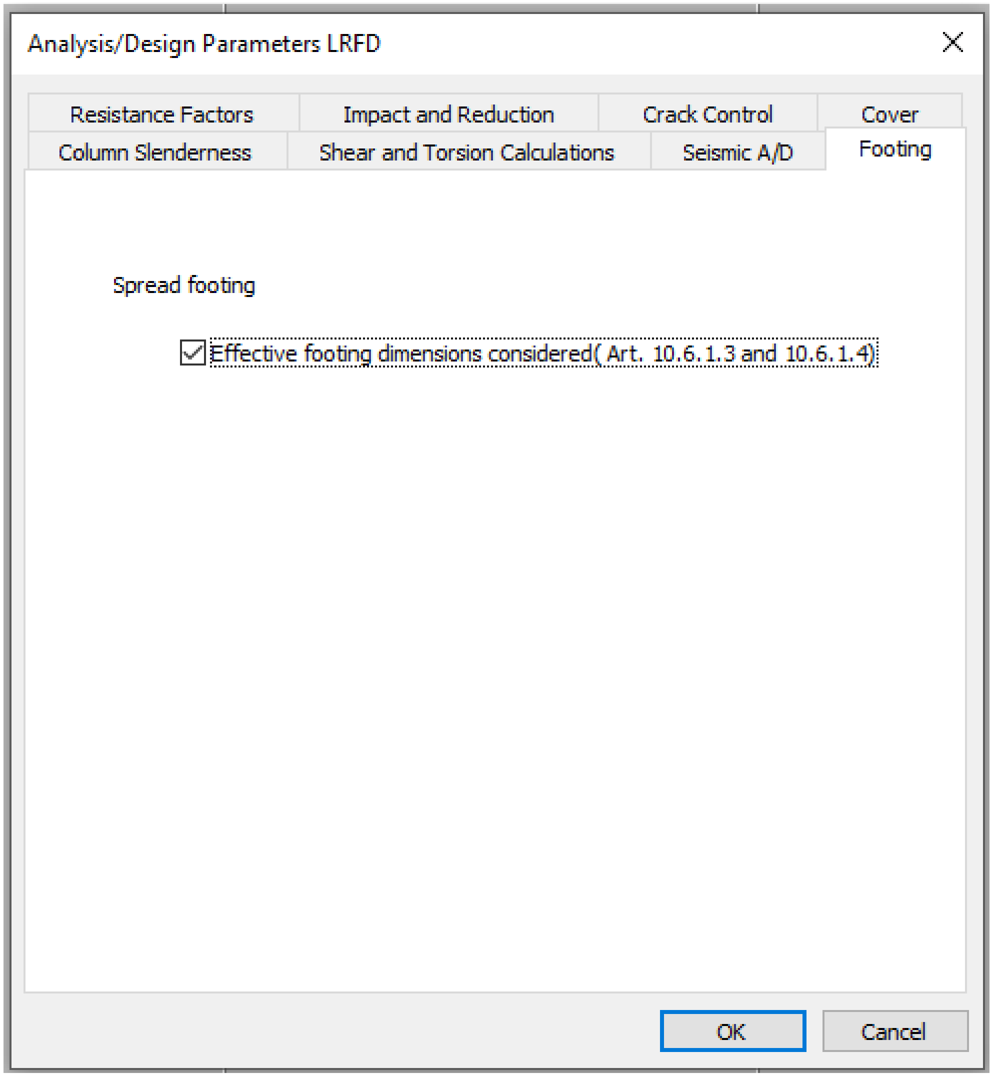


Figure 1. Leap Bridge Concrete Screenshot for Spread Footings

Section 3 Pile Footings

The minimum footing depth for pile footings shall be 4'-0".

10-301.00 Point-Bearing Piles

Point-bearing piles are common in Regions 1, 2, and 3. These piles are typically H-piles, but pipe piles are occasionally used in point-bearing applications. The minimum pile length (measured from the bottom of the footing, abutment beam, or wing beam) to the pile tip shall be 7'-0" for driven point-bearing piles. The minimum pile embedment into the footing, abutment beam, or wing beam shall be 1'-0". See Figure 2.

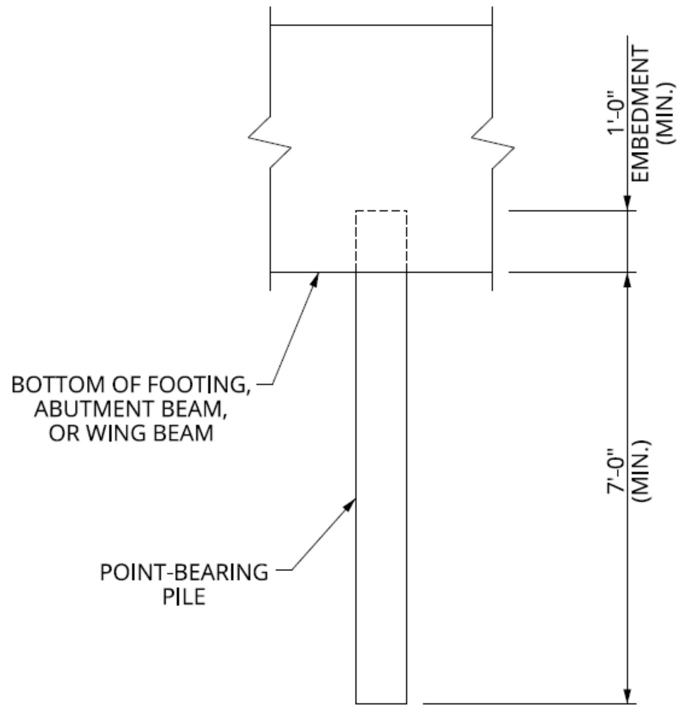


Figure 2. Minimum Point-Bearing Pile Length and Embedment

10-301.01 Required Bearing Value

The allowable design pile capacity, F_{pile}, for point-bearing piles shall be taken as:

$$F_{pile} = 0.35F_yA_g \quad (1)$$

Where: F_y = yield stress of pile = 50 ksi

A_g = area of pile

When designing point-bearing piles, the maximum Strength and Extreme Event pile reactions shall not exceed F_{pile}.

The required bearing value is the minimum bearing value to which the piles must be driven. This value shall be listed on the plans. When calculating the required bearing value, the phi factor for Strength pile reactions shall be taken as 0.6 as shown on the following page. The phi factor taken from AASHTO Table 10.5.5.2.3-1 is 0.1 when using the Engineering News Record (EN) bearing value equations. As given in the Standard Specifications, these equations include a safety factor of 6. This safety factor is not included in the same equations given in the AASHTO LRFD Bridge Design Specifications. Therefore, the 0.1 phi factor can be multiplied by 6 to give an effective phi factor of 0.6.

The required bearing value shall be listed on the plans and shall be taken as the greater of:

$$\frac{Maximum\ Strength\ Pile\ Reaction}{0.6}\ and\ \frac{Maximum\ Extreme\ Event\ Pile\ Reaction}{1.0}$$

As previously stated, the maximum Strength and Extreme Event pile reactions shall not exceed F_{pile} for point-bearing piles. Using the equations above, the required bearing value given on the plans shall be taken as:

Pile	Required Bearing Value
HP10x42	180 tons
HP12x53	225 tons

Table 1. Required Bearing Value

The plans shall specify that the piles shall be driven to the minimum required bearing value as determined above or refusal. Most point-bearing piles reach practical refusal as defined in Section 606.08 of the Standard Specifications. Practical refusal is considered refusal. In the event that practical refusal is not reached, the piles shall be driven to at least the required bearing value as determined above and shown on the plans.

Pipe piles are primarily used as friction piles. In cases where they are used as point-bearing piles, the minimum required bearing value shall be determined using the equations above.

10-301.02 Pile Uplift Capacity

Pile uplift forces at the Strength and Extreme Event Limit States shall not exceed 20 kips per pile unless a higher value is determined in accordance with AASHTO.

10-301.03 Pile Lateral Capacity

The lateral capacity of vertical point-bearing piles at the Strength and Extreme Event Limit States shall be calculated using Equation (2).

$$P = \frac{3EI\Delta}{L^3} \qquad (2)$$

Where: P = pile lateral capacity at the Strength and Extreme Event Limit States

E = pile modulus of elasticity (ksi)

I = pile moment of inertia about its bending axis (in^4)

Δ = assumed maximum one-way pile head deflection = 1 in.

L = distance from point of fixity to bottom of footing, not to be taken less than 120 in. for this calculation. (in.)

When higher lateral capacity is required, such as for closed abutments, piles shall be battered to resist the lateral forces. The maximum batter (horizontal:vertical) shall not exceed 4:12.

10-301.04 Pre-drilled Point-Bearing Piles

Pre-drilled piles may be considered when a 7'-0" pile length as shown Figure 2 cannot be achieved by pile driving and lowering the footing to be a spread footing on rock is not desired. The minimum pile length (measured from the bottom of the footing, abutment beam, or wing beam) to the pile tip shall be 10'-0" for pre-drilled piles. See Figure 6 in Appendix B for an example of the details to be shown on the bridge plans for pre-drilled point-bearing piles.

10-302.00 Friction Piles

Friction piles are very common in Region 4 and are either prestressed concrete piles or steel pipe piles. Steel HP piles shall not be designed as friction piles.

Steel pipe piles shall be ASTM A 252, Grade 3 modified with a minimum yield stress of 50 ksi unless alternate piles are approved by the Design Manager.

10-302.01 Driving Criteria – Static Load Test

The terms "driving criteria" (used in AASHTO) and "bearing value" (used in the TDOT Standard Specifications) are synonymous and refer to the actual value that the piles are to be driven to after the design load is calibrated based on the results of the load test.

The term "design load" is based on the maximum pile reaction determined by the design.

When piles are to be load tested using a static load test, the design load shall be established as follows:

Method: A successful static load test of at least one pile per site condition without dynamic testing is performed.

Resistance factor, φ_{dyn}, for determining the pile design load at the Strength Limit State: 0.75 (AASHTO Table 10.5.5.2.3-1)

Resistance factor, φ_{dyn}, for determining the pile design load at the Extreme Event Limit State: 1.0 (AASHTO 10.5.5.3.3.)

The required design load shall be listed on the plans and shall be taken as the greater of:

$$\frac{Maximum\ Strength\ Pile\ Reaction}{0.75}\ and\ \frac{Maximum\ Extreme\ Event\ Pile\ Reaction}{1.0}$$

The required design load shall not exceed the limits given in Table 2.

In the event that driving the test pile to at least the minimum tip elevation or driving the production pile full length might damage the pile because of excessively hard driving, the contractor shall use other methods approved by the Engineer for installing the piles such as jetting (where allowed) or pre-drilling holes. However, all piles must be driven by hammer for the last few feet of penetration. No measurement for payment will be made for pre-drilling holes or for jetting piling to obtain the required pile penetration.

Friction Pile Type	Maximum Design Load
Prestressed Concrete	150 tons
Steel Pipe Piles (static load test)	200 tons
Steel Pipe Piles (dynamic testing)	Based on design and with approval of the Design Manager

Table 2. Maximum Design Load

10-302.02 Driving Criteria – Dynamic Testing

The terms "driving criteria" (used in AASHTO) and "bearing value" (used in the TDOT Standard Specifications) are synonymous and refer to the actual value that the piles are to be driven to after the design load is calibrated based on the results of the dynamic testing.

The term "design load" is based on the maximum pile reaction determined by the design.

When dynamic testing will be used, the design load shall be established as follows:

Method: Dynamic testing in accordance with TDOT Special Provision 930PDA. Quality control shall be accomplished by dynamic testing of at least two piles per substructure, but not less than 2% of the production piles. Dynamic testing requires signal matching, and best estimates of nominal resistance are made from a restrike.

Resistance factor, φ_{dyn}, for determining the pile design load at the Strength Limit State: 0.65 (AASHTO Table 10.5.5.2.3-1)

Resistance factor, φ_{dyn}, for determining the pile design load at the Extreme Event Limit State: 1.0 (AASHTO 10.5.5.3.3.)

The required design load shall be listed on the plans and shall be taken as the greater of:

$$\frac{Maximum\ Strength\ Pile\ Reaction}{0.65}\ and\ \frac{Maximum\ Extreme\ Event\ Pile\ Reaction}{1.0}$$

The required design load shall not exceed the limits given in Table 2.

In the event that driving the piles to at least the minimum tip elevation or driving the production pile full length might damage the pile because of excessively hard driving, the contractor shall use other methods approved by the Engineer for installing the piles such as jetting (where allowed) or pre-drilling holes. However, all piles must be driven by hammer for the last few feet of penetration. No measurement for payment will be made for pre-drilling holes or for jetting piling to obtain the required pile penetration.

10-302.03 Pile Uplift Capacity

Pile uplift forces at the Strength and Extreme Event Limit States are limited to the lesser of:

- the geotechnical capacity of the pile due to side friction in uplift
- the structural capacity of the pile in tension based on the seismic attachment (See STD-6-1.)

The structural steel capacity of the seismic attachment shall be determined using a φ- factor of 0.9 at the Strength Limit State and a φ-factor of 1.0 at the Extreme Event Limit State. For prestressed concrete piles, two seismic attachments are detailed on STD-6-1, but base the structural capacity of the pile attachment in uplift on four C6- bars drilled and grouted into the pile. The capacity of this attachment is calculated as:

- φT_n = 0.9 × 4 bars × 0.44 in^2/bar × 60 ksi = 95 kips (Strength Limit State)
- φT_n = 1.0 × 4 bars × 0.44 in^2/bar × 60 ksi = 106 kips (Extreme Event Limit State)

When evaluating pile patterns at the Extreme Event Limit State, it is permissible to apply the plastic moment of the column (instead of the over-strength plastic moment) to the footing when hinging is the basis of design. See Article C6.4.2 of the AASHTO Guide Specifications for LRFD Seismic Bridge Design (Guide Specifications) for this allowance.

10-302.04 Pile Lateral Capacity

The lateral capacity of vertical friction piles at the Strength and Extreme Event Limit States shall be calculated using Equation (2) from Section 10-301.03. Battered friction piles are not permitted for bridge foundations. However, battered friction piles are permitted for retaining wall foundations. The maximum batter (horizontal:vertical) shall not exceed 4:12.

Section 4 Abutment Pile Deflection Limits

For all integral abutments supported by either prestressed concrete or steel piles, the total anticipated pile movement due to thermal forces shall not exceed 2 inches. Otherwise, an expansion joint shall be provided at the abutment. Prestressed concrete piles may be used at abutments if the total anticipated pile movement due to seismic forces does not exceed 5 inches. Otherwise, steel piles shall be used, and the pile embedment in the abutment beam shall be increased to 2'-0". Any consideration for deviation from this policy shall require Director approval.

Section 5 Drilled Shafts

Cross-hole Sonic Logging (CSL) testing with the addition of 3D tomography shall be performed for all drilled shafts. If CSL testing is indicated on a project with CEI oversight, the CEI firm shall supply a subconsultant that is qualified to perform the testing. If the project does not have CEI oversight, then the Department will supply a consultant to perform the testing. Since the contractor never hires the CSL consultant, CSL testing is never a bid item on the bridge plans. Specify 1 CSL tube per foot of shaft diameter, but not less than 4 CSL tubes. Specify Core Drilling and Sampling (Item Number 204-05.01) at each shaft location as a means of verifying estimated shaft tip elevations.

The Geotechnical Section shall be consulted to determine if permanent casing is required.

The shaft capacity for axial load shall be based on either end bearing capacity or side fiction capacity, but not a combination of both. The minimum rock socket length shall be as directed by the geotechnical engineer, but in no case shall be less than 1.5 times the rock socket diameter.

All drilled shafts shall have a lateral load analysis performed. Contact the Geotechnical Section for more information about this analysis.

The top of permanent casing shall not extend more than 5'-0" above the drilling platform (ground surface, barge, rock pad, etc.) without approval of the Design Manager.

Drilled shafts shall not be used at abutments without approval of the Design Manager.

When karst conditions at drilled shaft locations are identified prior to the design, drilled shafts shall not be designed with the axial load capacity based on side friction without approval of the Design Manager.

Section 6 Cofferdams and Seal Footings

When pier footings are to be placed a significant depth below the water surface, the use of cofferdams is necessary. After constructing the cofferdam, it shall be sealed to make it as nearly water-tight as possible. Pumps are then used to de-water the cofferdam. If the sealing efforts are successful, the cofferdam will remain relatively dry, allowing construction of the structural footing.

For foundations on porous material or fractured rock, de-watering may be impossible as water removed by the pumps is replaced with water forced up through the foundation material. A structural footing cannot be constructed in turbulent or unstable water conditions. In this situation a seal footing shall be required. The seal footing serves to resist the static head created by the water elevation differential between the inside and the outside of the cofferdam.

Prior to pouring the seal footing, the cofferdam is filled with water to an elevation equal to that outside the cofferdam. The hydrostatic pressure is thus stabilized, and the seal concrete shall be poured using a tremie to deposit concrete evenly and uniformly in the bottom of the cofferdam. The structural footing could not be constructed in this manner since the reinforcement would prevent the necessary freedom of movement.

The cofferdam must be de-watered after construction of the seal footing, so the seal footing shall be thick enough to balance the hydrostatic pressure from the water outside the cofferdam. Using unit weights of 62.4 pcf for water and 145 pcf for unreinforced concrete gives a required seal footing thickness of 0.43 times the outside water depth. If the cofferdam is constructed in a manner to provide adequate anchorage to the seal footing, then the cofferdam weight may be included in the calculations to reduce the required seal footing thickness.

When seal footings are required for submerged footings on piles, a portion of the frictional resistance of the piles in uplift may be used in resisting the hydrostatic forces. An upward load test may be required to verify this resistance.

When sizing a seal footing, a 3'-0” work zone shall be provided on all sides of the structural footing on top of the seal. The 3'-0” work zone gives the Contractor room to place pumps for de-watering the cofferdam. The least plan dimension of the seal shall not be less than half the seal depth.

In cases where it is determined that a seal footing is required, the contract plans shall include bid items for Cofferdams, Class S Concrete (Foundation Seal), and Core Drilling (Concrete Seal). It is common for Rock Excavation (Bridges) to also be required in order to provide the required seal depth.

A typical seal footing is shown in Figure 3. An example of calculations to determine the minimum seal footing depth are given in Appendix C.

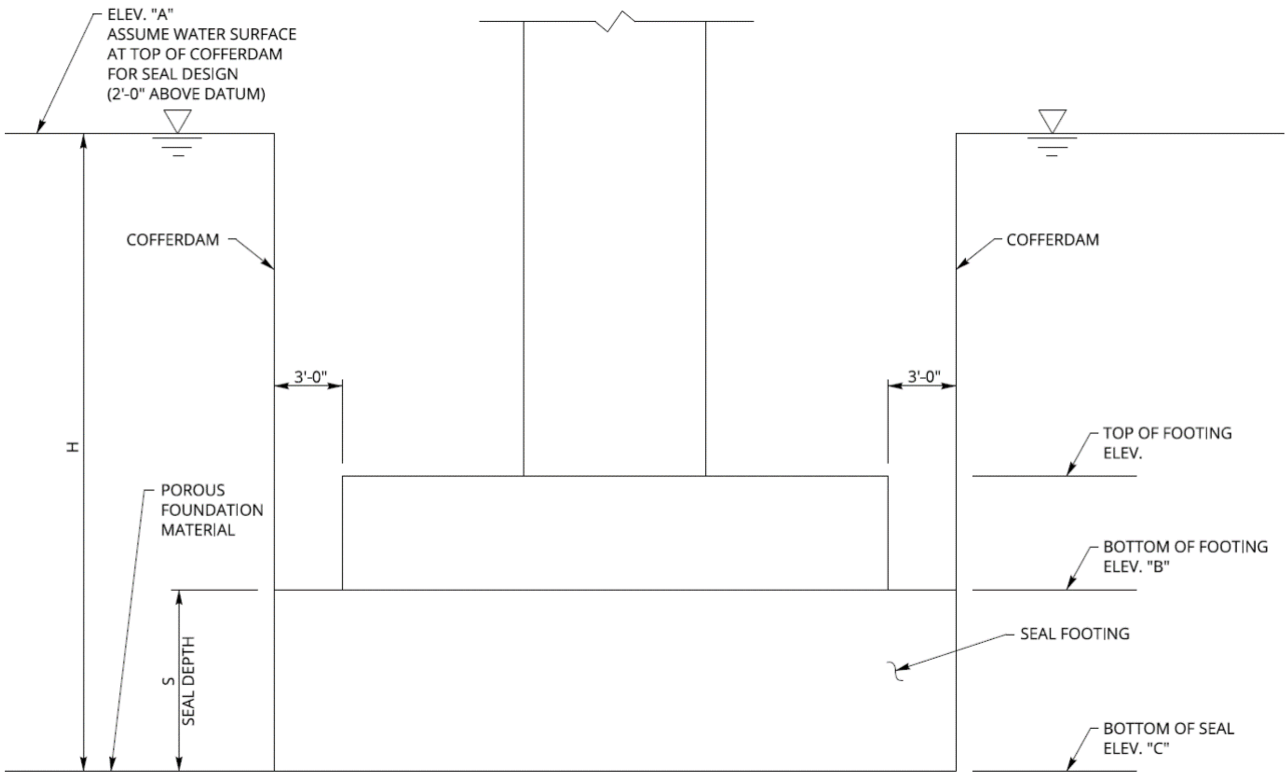


Figure 3. Cofferdam and Seal Footing Details

Section 7 Micropiles

Micropiles are commonly used for accelerated bridge construction (ABC) projects where foundations must be installed under an existing bridge. In these scenarios, there is usually not enough vertical clearance to use a pile driver. Micropiles can be drilled with smaller equipment that requires much less vertical clearance.

Micropiles are usually 9.625" in diameter and include a single high-strength reinforcing bar. They have steel casing for at least part of their length. In order to meet the requirements of the Buy American Act, all casing shall have mill certification to verify that it is of domestic origin. When micropiles are specified on a project, Special Provision SP625MP shall be included in the contract documents. SP625MP is project specific and must be reviewed and edited for each project.

Section 8 Top of Footing Elevations

For spread footings on rock:

- For non-hydraulic crossings, top of footings shall be set at least two feet below the top of the finished ground with the footing seated in competent rock.
- For hydraulic crossings, top of footings shall be set at least two feet below the top of the streambed with the footing seated in competent rock. If rock is encountered, the top of footing may be raised with the approval of the Hydraulics Manager.

For footings supported by piles:

- For non-hydraulic crossings, top of footings shall be set at least two feet below the top of the finished ground with the piles driven to competent rock or achieving adequate friction values.
- For hydraulic crossings, top of footings shall be set at least two feet below the top of the streambed with the piles driven to competent rock or achieving adequate friction values and having a minimum embedment of 15 feet below the 500-year scour elevation.

For seal footings, contact the Design Manager for guidance.

Appendix A: Pipe Pile Design Information

For pipe piles, the fully plastic moment can be calculated with Equations (3) through (9).

- D = outside diameter of pipe
- t = pipe wall thickness
- P_u = axial compressive load on the pile
- F_y = yield stress of the pipe pile material

$$R = \frac{D-t}{2} \tag{3}$$

$$\alpha = \frac{2\pi R F_y t - P_u}{4 R F_y t} \tag{4}$$

$$T = 2R\alpha F_y t \tag{5}$$

$$C = 2R(\pi - \alpha)F_y t \tag{6}$$

$$x_T = \frac{R\sin(\alpha)}{\alpha} \tag{7}$$

$$x_C = \left(\frac{\alpha}{\pi - \alpha}\right) x_T \tag{8}$$

$$M_p = T x_T + C x_C \tag{9}$$

The equation for V_u in Figure 4 can be used to determine the pile embedment, L_e.

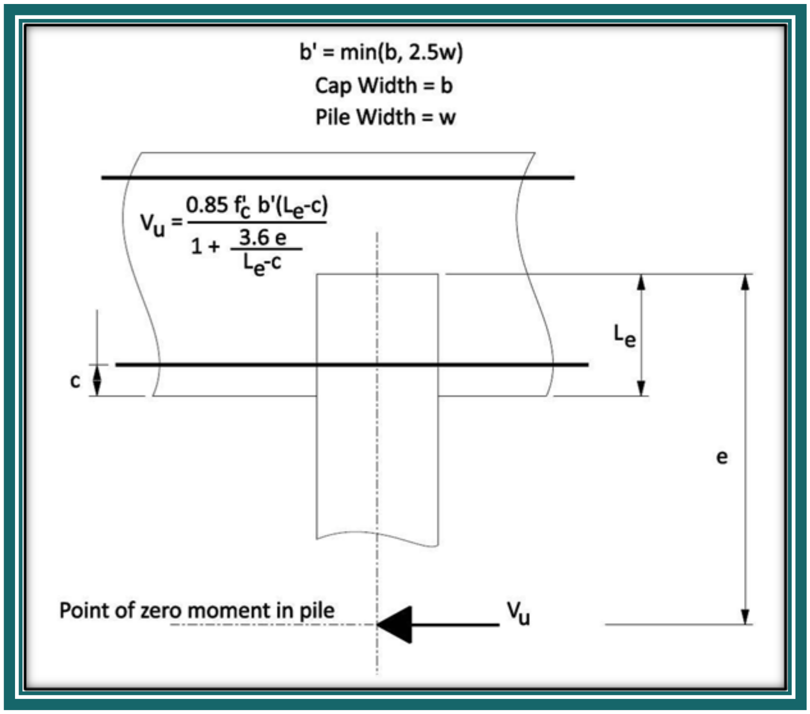


Figure 4. Pile Embedment Requirements

When designing pipe pile bents with high seismic demands, it is common for the piles to be pushed past the elastic limits and become ductile members. In this case it is helpful to fill the piles with concrete to satisfy the limiting width-thickness ratios in Table 7.4.2-1 of the Guide Specifications. Figure 5 shows details that may be used when using pipe piles filled with concrete.

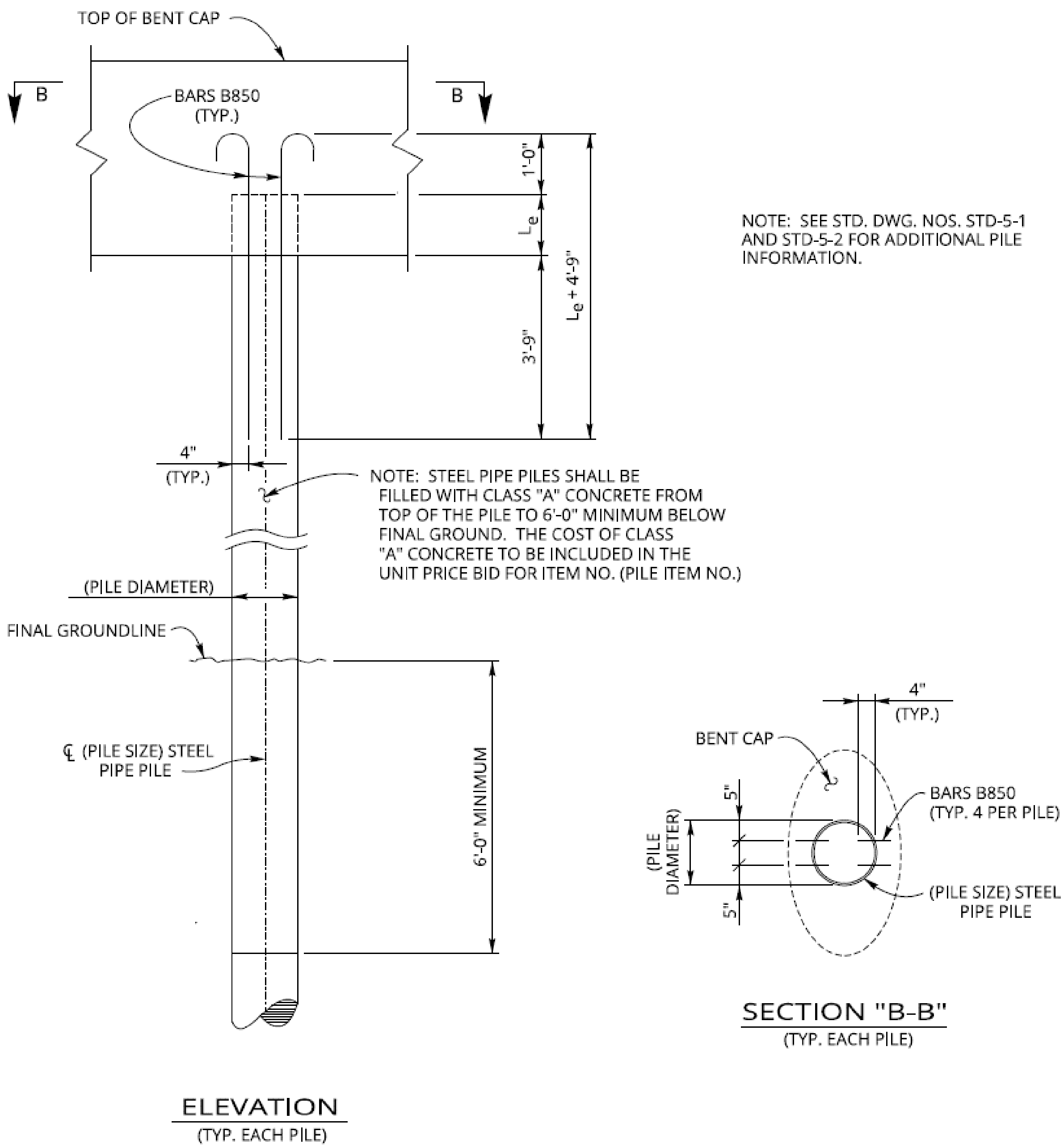
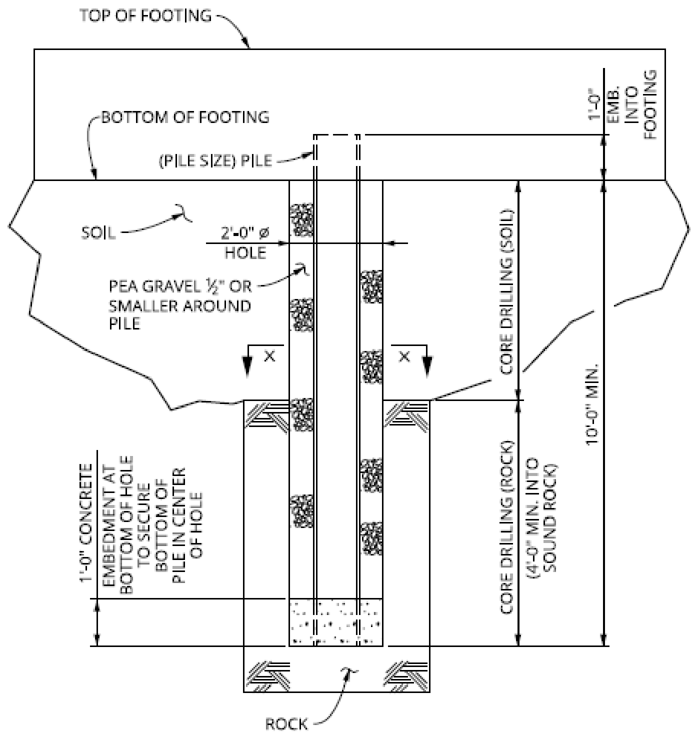


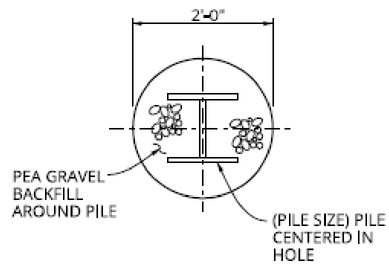
Figure 5. Concrete-Filled Pipe Pile Details

Appendix B: Pre-drilled Point-Bearing Pile Details

An example of typical plans details for pre-drilled piles is given in Figure 6. These details may vary based on recommendations from the Geotechnical Section.



PRE-DRILLED PILE DETAILS



SECTION "X-X"

ALL PILES AT (LOCATION) SHALL BE INSTALLED IN ACCORDANCE WITH THE DETAILS ON THIS SHEET. STEEL PILE TIPS (ITEM NO. ________) ARE NOT REQUIRED.

1. A 2'-0" DIAMETER HOLE (10'-0" MINIMUM IN LENGTH) SHALL BE DRILLED IN THE PLANS LOCATION FOR THE PILE. THE AMOUNT OF CORE DRILLING (SOIL) AND CORE DRILLING (ROCK) WILL VARY DUE TO LOCATION OF ROCK LINE.
2. THE HOLE SHALL BE THOROUGHLY CLEANED OF ALL LOOSE MATERIAL, MINIMUM OF 12 INCHES OF HIGH EARLY STRENGTH CONCRETE PLACED IN THE BOTTOM OF THE HOLE, THE PILE INSERTED AND CENTERED IN THE HOLE, AND THE PILE SUPPORTED UNTIL THE CONCRETE REACHES ITS INITIAL SET STRENGTH.
3. AS SOON AS THE CONCRETE HAS OBTAINED ITS INITIAL SET AND IT IS ABLE TO STABILIZE THE PILE TIP LOCATION IN THE BOTTOM OF THE HOLE, THE HOLE SHALL BE BACKFILLED WITH PEA GRAVEL. THE TOP OF THE PILE SHALL BE BRACED IN A MANNER SO THAT ITS VERTICAL POSITION IN THE CENTER OF THE HOLE IS MAINTAINED.
4. CONTRACTOR SHALL TAKE ALL NECESSARY PRECAUTIONS SO AS NOT TO DAMAGE THE EXISTING BRIDGE FOUNDATION DURING PILE PRE-DRILLING. CONTRACTOR IS RESPONSIBLE FOR THE COST TO REPAIR ANY DAMAGED AREAS.

BASIS FOR PAYMENT:
THE COST OF ALL MATERIAL AND LABOR FOR DRILLING THE 2'-0" HOLES, PILE END ENCASEMENT, BACKFILLING WITH PEA GRAVEL, BRACING OF THE PILES, AND ANY OTHER INCIDENTALS REQUIRED FOR FULL INSTALLATION OF THE PILES SHALL BE INCLUDED IN ITEM NO. (PILE ITEM NO. AND DESCRIPTION).

Figure 6. Pre-drilled Point-Bearing Pile Details

Appendix C: Example Seal Footing Depth Calculations

See Section 6 and Figure 3.

Footing size: 4’-0” x 12’-0” x 12’-0”

Seal size: S x 15’-0” x 18’-0”

Elev. “A” = 316.00

Elev. “B” = 300.00

S = 0.43H

H = Elev. “A” – Elev. “B” + 0.43H

0.57H = 16

H = 28.1

Elev. “C” = Elev. “A” – H

= 316.00 – 28.1

= 287.9

Required Seal Footing Depth (S) = 0.43 H = 12.1 ft.

Works Cited

AASHTO, *LRFD Bridge Design Specifications.* 9th. Washington, D. C.: American Association of State Highway and Transportation Officials, 2020.

AASHTO, *Guide Specifications for LRFD Seismic Bridge Design.* 2nd. Washington, D. C.: American Association of State Highway and Transportation Officials, 2011.