Oregon Department of Transportation

SECTION 1.10
Foundation Considerations

Revisions for the current edition are highlighted yellow. Deleted text is not marked.

Past editions of the Bridge Design Manual are available through our website.

1.10.1 Foundations, General

1.10.2 Lateral Earth Restraint

1.10.3 Underwater Construction

1.10.4 Foundation Modeling (Foundation Springs)

1.10.5 Foundation Design

1.10 FOUNDATION CONSIDERATIONS

1.10.1 Foundations, General

Coordinate and resolve differences related to the structural, hydraulic, and geotechnical aspects of foundation design prior to TS&L.

The geotechnical designer will provide data and recommendations with respect to types of footings, footing elevations, nominal and factored resistances, types of piling, pile tip reinforcing, and drilled shaft tip elevations to be used at each bridge site.

Special factors in the type of construction selected may cause a reconsideration of the original recommendation.

When geotechnical analysis shows that a bridge site will have excessive soil movement due to an earthquake, the bridge and geotechnical designers collaborate to determine the potential impact on the bridge performance and the available mitigation methods, which may lead to the need for ground improvement at the project site.

C 1.10.1

The designer should be satisfied that the recommendations are adequate with respect to factored loads and economy. When there are questions in this matter, discuss with the geotechnical designer.

Some basic guidelines include:

- When the Geotechnical Report is not available, the fact should be noted and the indicate the basis for the design of the footings.
- Except for special cases, provide a minimum of 2 feet of cover over the top of spread footings.
- Make the top of footings within the right of way of the Union Pacific Railroad a minimum of 6 feet below the bottom of the low rail to allow for future underground utilities.

See *BDM 1.17.4* for mitigation guidelines.

1.10.2 Lateral Earth Restraint

C 1.10.2

When passive earth pressures are used in design to resist seismic or other lateral loads, detail the plans to ensure assumed soil conditions exist after construction. Where possible, specify placing footings against undisturbed material.

When soil is disturbed, *SP 00510.41* requires backfilling with compacted granular material.

When there are questions concerning this, consult with the geotechnical designer.

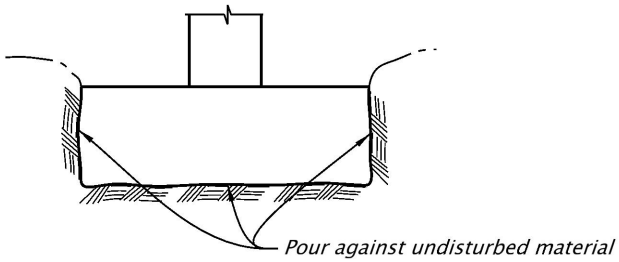
When the excavation will not stand vertically, add a reference note, **See** *Standard Specifications for Construction* to the **Structure Excavation Limits** detail shown on the plans.

The soil type may be such that it will not stand vertically after excavation. The contractor will be allowed to excavate beyond the footing limits and backfill with compacted granular structure backfill (*SP 00510.46*).

When footings do not require the lateral soil resistance for stability, do not call for pouring against undisturbed material.

These are footings such as pile supported, etc..

Figure 1.10.2-1 Lateral Earth Restraints



1.10.3 Underwater Construction

1.10.3.1 Underwater Foundation Design Considerations

For scour design requirements see *BDM 1.10.5.8*. Requirements for scour protection, potential scour depths and elevations, recommendations for riprap protection can be found in the *Hydraulic Report* and *BDM 1.10.5.8.*

Size the seal large enough to accommodate the footing plus footing forms inside the cofferdam walers. Provide a minimum of 2 feet on each side of the footing.

Require the contractor to remove all underwater formwork.

Riprap is not considered permanent protection against scour for seals.

In streams where there is a potential for scour, place riprap as soon as possible and before removal of the cofferdam.

Scour calculations do not take into account debris loading.

During extreme flood events the mobile streambed material cannot be counted on for protection.

C 1.10.3

C 1.10.3.1

Refer to the *BDM, Hydraulics Design Manual* and ultimately the *Hydraulics Report* on how scour countermeasures are incorporated into the foundation design. See *BDM 1.10.5.4.1* and *HM 10.12* for additional information.

The seal size ultimately determines the cofferdam size.

A pile of debris will cause a larger obstruction thereby increasing the scour depth.

Streambeds are often **mobile** and the top few feet or so are moving downstream all the time.

When placing a footing in a stream, the material around and over the footing has been reworked and doesn't have the in situ strength of the native streambed.

Total scour calculations include both local and degradation scour (e.g. is the stream degrading or does it have the possibility of degrading in the future).

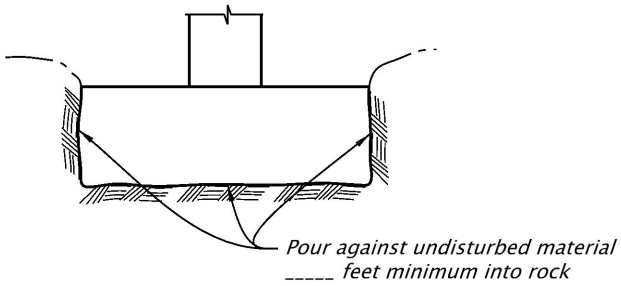
The depth component of the bearing resistance equation has the most significant contribution to the footing's ability to support the load.

1.10.3.2 Footing Embedment

C 1.10.3.2

On stream crossings and where horizontal forces are involved, include *Figure 1.10.3.2-1* on the plans if the foundation material is suitable.

Figure 1.10.3.2-1 Footing Note Callouts at Streams



Place the bottom of footings in streambeds a minimum of 6 feet below the normal streambed, except in solid rock.

In solid rock, place the top of the footing flush with the rock line.

1.10.4 Foundation Modeling (Foundation Springs)

It is common practice to first assume translational and rotational fixity of the foundation supports and perform a preliminary structural frame analysis. The resulting reactions are checked against the factored resistances.

Excessively conservative design forces are expensive to accommodate. In these cases, use foundation springs in the structural frame analysis.

Nominal geotechnical resistances are used with seismic loading conditions unless otherwise directed by the geotechnical designer. Factored resistances are used for all other load combinations. Factored resistance is the nominal resistance multiplied by the appropriate resistance factor.

C 1.10.4

This procedure underestimates global deflections but establishes an upper bound for forces. This type of foundation modeling may be sufficient in certain loading conditions, such as thermal expansion, where deflections are not a controlling factor in design provided the forces are not excessive.

Under higher lateral loading conditions, such as moderate to severe seismic loading, more accurate deflections and forces are desirable. Computer programs such as Midas Civil or SAP2000 allow the use of foundation springs. Foundation springs are typically equivalent linear springs representing the translational (horizontal), axial (vertical) and rotational load-deflection behavior of a nonlinear soil response. The use of foundation springs can significantly reduce the upper bound foundation reactions and more accurately models the entire soil-structure interaction system.

1.10.4.1 General Modeling Techniques

There are three options for foundation modeling:

- (Option 1) Fixed foundations.

C 1.10.4.1

Option 1 fixes all foundation supports in the computer model. The resulting forces are simply compared to the resistances stated either in the Geotechnical Report or as determined in this section of the BDM. When the resulting forces exceed the resistances, foundation modeling using springs is recommended.

- (Option 2) Fully coupled foundation spring model.

Option 2 allows stiffness coupling for both shear and moment and also cross-coupling (off diagonal). This option is not required for most problems. This option should be used for drilled shafts, trestle piles and for some pile foundations where the piles are connected to the substructure or superstructure such that a fixed condition exists. A massive footing with deeply embedded piles is an example. The method is applicable to all types of foundations.

- (Option 3) Uncoupled translation and rotational springs.

Option 3 is the most commonly used method to represent footing and piling flexibility. It is a simplified version of the fully coupled spring model (Option 2) and is used in cases where there is no significant moment transfer between superstructure and foundation elements. This option is appropriate for most problems except as noted in (Option 2). Use this option with vertical piling only. Battered piles result in larger lateral stiffness, which this option does not presently address.

1.10.4.2 General Procedures and Typical Values

C 1.10.4.2

Methods are presented for developing foundation springs, including factored and nominal resistances, for the following foundation types:

- Abutments and wingwalls.
- Spread footings.
- Piles and pile caps.

The following guidelines are provided for Option 3 as general information, and are intended to be supplemented with engineering judgment.

Foundation springs are typically nonlinear in form although some are represented in bilinear form. The curve typically consists of an initial (straight line) stiffness followed by a nonlinear relationship leading up to a nominal resistance. Various methods are used, depending on the type of spring, to

develop the entire nonlinear load-deflection curve (spring).

The procedures described in this section, and typical values, come from the following sources:

- *Seismic Design of Highway Bridges*, Workshop Manual by Imbsen & Associates, Inc., prepared for the FHWA, October, 1989.
- *Design and Construction of Driven Pile Foundations*, FHWA Workshop Manual, Volume 1, December, 1996.
- Design Manual 7.2, *Foundations and Earth Structures*, Dept. of the Navy, May, 1982.
- *Foundation Analysis and Design*, (4th ed.) by Joseph E. Bowles.
- *Design Manual for the Foundation Stiffnesses Under Seismic Loading*, prepared for Washington DOT by Geospectra, April, 1996.
- *Design Guidance: Geotechnical Earthquake Engineering For Highways*, Volumes I & II; FHWA Report No. *FHWA-SA-97-076-77*, May, 1997.
- Pile capacity and stiffness work done by Bridge Engineering and Geotechnical Group personnel in 1996 and 1997.

Standard Penetration Test (SPT) numbers presented in the Design Manual (**Nc** values) refer to **N** values for granular soils corrected to an effective overburden pressure of 1 tsf. Uncorrected **Nc** values should be used for cohesive soils. The geotechnical designer should be consulted for representative values to use in these methods.

1.10.4.2.1 Abutments and Wingwalls

Use translational springs in both the longitudinal and transverse directions.

TRANSLATIONAL STIFFNESS

Account for displacements resulting from expansion joints associated with seat abutments in the abutment and wingwall translational stiffness.

Determine initial backfill stiffness by the backfill of the abutment and wingwalls. Model wingwalls similar to the abutment. Direction of the wingwall contribution is into the approach fill. Adjust the initial stiffness proportional to the backwall width and height according to the following equation where the height of the backwall is normalized (*Maroney 1995*).

Use this value for unknown backfill or when the backfill does not meet the requirements shown on *DET3160*.

$$k_i = \frac{20\frac{kips}{in}}{ft}$$

When the approach fill is constructed/reconstructed using granular structure backfill and meeting the limits shown on *DET3160* the following value for initial stiffness can be used.

$$k_i = \frac{50\frac{kips}{in}}{ft}$$

The stiffness of the abutment is calculated with the following:

$$K_{abut} = k_i * W * \frac{H}{5.5ft}$$

Where:

C 1.10.4.2.1

C TRANSLATIONAL STIFFNESS

$k_i = the\ initial\ stiffness\ with\ units\ of\ \frac{\frac{kips}{in}}{ft}$

W
$= the\ width\ of\ the\ abutment\ with\ units\ of\ ft$

H
$= the\ height\ of\ the\ abutment\ with\ units\ of\ ft$

Piles: Refer to *Pile Supported Footings and Abutments* (see *BDM 1.10.4.2.4*). Use pile translational stiffnesses in tables below for loading conditions other than seismic. For seismic loading conditions, perform an analysis using soil response program such as LPILE. Consult with the geotechnical designer to verify LPILE soil properties.

TRANSLATIONAL CAPACITIES

The passive force resisting the movement at the abutment is modeled using a bi-linear curve with respect to displacement. Calculate the ultimate static passive force using the following equation. Apply maximum passive force only once the soil has been mobilized. Acceptable passive mobilization values are found in *AASHTO Table C.3.11.1-1*. When the wall is backfilled with a medium dense sand or compacted silt use a value of $0.02H$ $(where\ H\ is\ the\ height\ of\ the\ wall\ in\ ft)$ When granular structure backfill is used to fill the active and passive wedge use a value of $0.01H$ to determine if mobilization occurs. Only when this deformation is equal to or exceeds this value, then the ultimate passive force can be employed.

$$F_{ult} = WH^2 \frac{5ksf}{5.5ft}$$

Where:

F_{ult}
$= the\ maximum\ passive\ force\ with\ units\ of\ kips$

C TRANSLATIONAL CAPACITIES

$W_{bw} = the\ width\ of\ the\ backwall\ in\ feet$

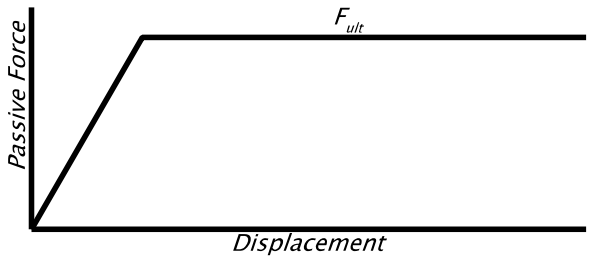
H_{bw}
$= the\ height\ of\ the\ backwall\ and\ cap\ in\ feet$

Piles: For seismic loading, use ultimate values derived from LPILE analysis by comparing the maximum yield moment of the pile to the maximum moment output from LPILE. Take end slope and side slope effects into account. Generally assume dense granular fill representing granular wall backfill. This material should be present in the entire passive wedge area. Consult with the geotechnical designer to verify LPILE soil properties. Use allowable pile capacities in tables below for loading conditions other than seismic.

TRANSLATIONAL LOAD-DEFLECTION CURVE

Use the initial stiffness up to the capacity limit. The curve form is shown in *Figure 1.10.4.2.1-1*:

Figure 1.10.4.2.1-1 Translational Load Deflection Curve



C TRANSLATIONAL DEFLECTION CURVE

1.10.4.2.2 Skewed Abutments with Wingwalls

When the abutment is skewed the passive force calculated above is further reduced using the following and applied perpendicular to the skewed bent.

$F_{ult-skew} = F_{ult} * R_{skew}$

$R_{skew} = e^{-\frac{\theta}{45}^{\circ}}$

Where:

F_{ult}
$= the\ maximum\ static\ passive\ resistance.$

$F_{ult-slew}$
$= the\ ultimate\ static\ passive\ resistance\ on\ a\ skewed\ bridge\ permitted\ by\ ODOT.$

θ
$= skew\ angle\ of\ the\ bridge\ with\ units\ of\ degrees$

$R_{skew} = the\ reduction\ factor$

To allow for modeling of the bridge in the longitudinal direction the ultimate static passive resistance distribution is modified by increasing it at the obtuse corner of the bridge deck by 1.25 and decreasing at the acute corner by 0.75 as shown in *Figures 1.10.4.2.2-1* and *1.10.4.2.2-2*.

C 1.10.4.2.2

Recent large scale testing and numerical modeling of skewed abutments with wingwalls parallel with the roadway show a significant reduction in passive pressure as well as increased displacements, and increased bending moment that must be accounted for in design (*Rollins and Snow, 2019*).

Figure 1.10.4.2.2-1 Skewed Abutment Loading, Plan View

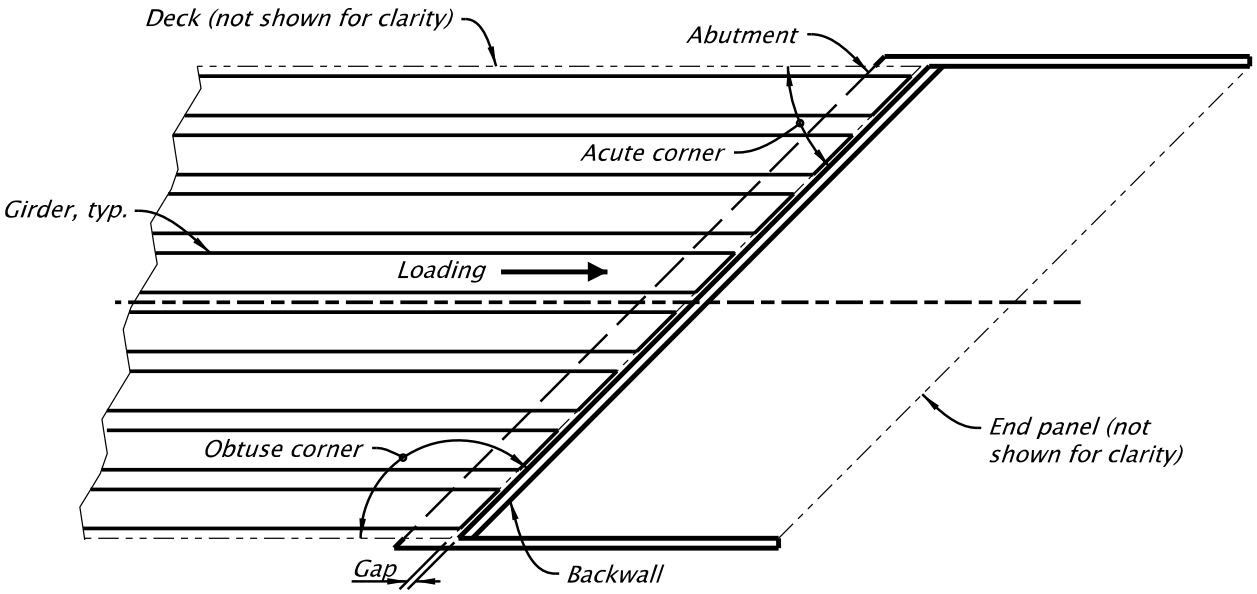
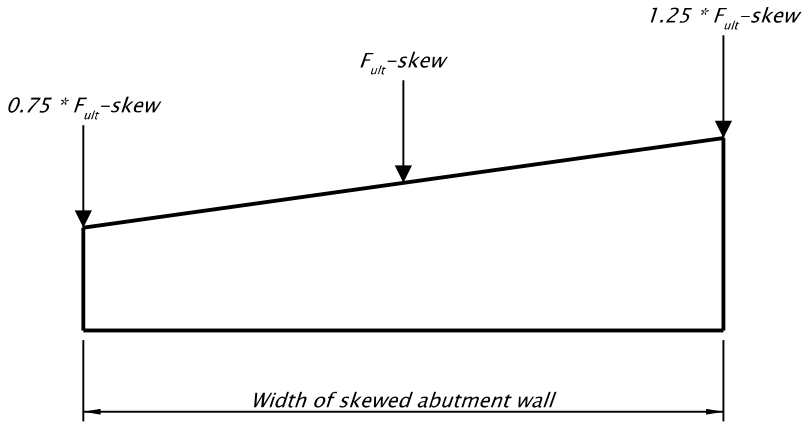


Figure 1.10.4.2.2-2 Skewed Abutment Loading, Elevation View



1.10.4.2.3 Spread Footings

Unless constructed on solid bedrock, use translational and rotational springs in both the longitudinal and transverse directions.

C 1.10.4.2.3

In general, footings keyed into a rock mass that has an elastic (Young's) modulus typically greater than 14,000 ksf (Unconfined Compressive Strength = 1000 psi) can be considered **fixed** against both rotation and translation. Consult with the geotechnical designer to determine the compressibility of very soft or highly fractured bedrock materials.

TRANSLATIONAL AND ROTATIONAL STIFFNESSES

Use the equivalent circular footing formulas with information from *Table 1.10.4.2.3-1*, to develop translational and rotational spring constants. Consult with the geotechnical designer for the appropriate soil values to use in *Table 1.10.4.2.3-1*.

C TRANSLATIONAL AND ROTATIONAL STIFFNESSES

Table 1.10.4.2.3-1 Soil Properties

Soil Classification	**SPT "Nc"***	**E (ksf)**	**Poisson's Ratio (*v*)**	**G (ksf)**
Granular				
V. Loose	4	300	.35	110
Loose	10	1000	.35	370
Medium	30	2000	.35	750
Dense	50	3000	.35	1100
Cohesive				
Soft	4	400	.50	150
Stiff	8	1000	.50	350
Very Stiff	16	1500	.50	500
Hard	32	2000	.50	650

Tables notes:
* **Nc** is the average of **Nc** values over a depth of 2B below the footing, ($B = footing\ width$).

STIFFNESS CALCULATIONS FOR SPREAD FOOTINGS

Obtain spring constants for rectangular footings by modifying the solution for a circular footing bonded to the surface of an elastic half-space. The formula is as follows where:

$k = initial\ stiffness\ (spring\ constant)$

α
$= foundation\ shape\ correction\ factor;\ (from\ graph)$

C STIFFNESS CALCULATIONS FOR SPREAD FOOTINGS

$\beta = embedment\ factor, (from\ graph)$

$K0$
$= stiffness\ coefficient\ for\ the\ equivalent\ circular\ footing\ (see\ formulas\ in$ Table 1.10.4.2.3 $- 2)$

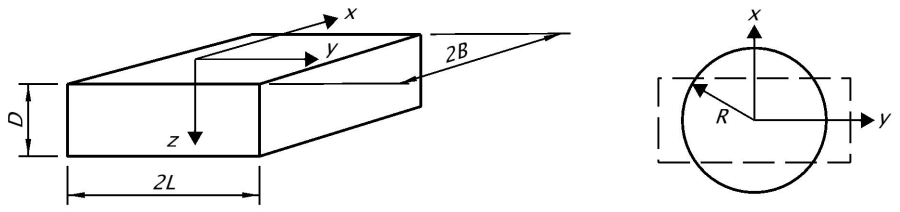
The stiffness term, K_0, is calculated using the equations in *Table 1.10.4.2.3-2*.

Table 1.10.4.2.3-2 Stiffness Equations for Spread Footings

Displacement Degree-of-Freedom	**K_0**
Vertical translation	$4GR/(1-v)$
Horizontal translation	$8GR/(2-v)$
Torsional rotation	$16GR^3/3$
Rocking rotation	$8GR^3/(3(1-v))$

Table notes:
G = Shear Modulus (low strain range)
v = Poisson's ratio for elastic half-space material
R = Equivalent footing radius as determined from the following equations

Figure 1.10.4.2.3-1 Equivalent Radii, R, for Rectangular Footing Spring Constants



RECTANGULAR FOOTING **EQUIVALENT CIRCULAR FOOTING**

Figure 1.10.4.2.3-2 Shape Factors for Rectangular Footings

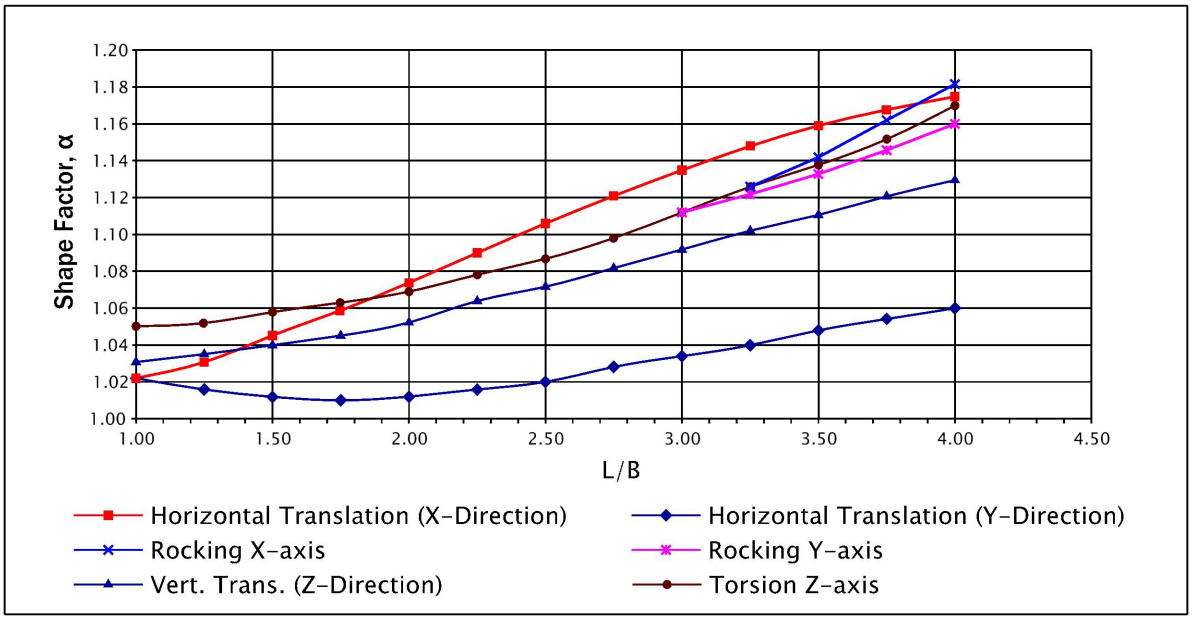
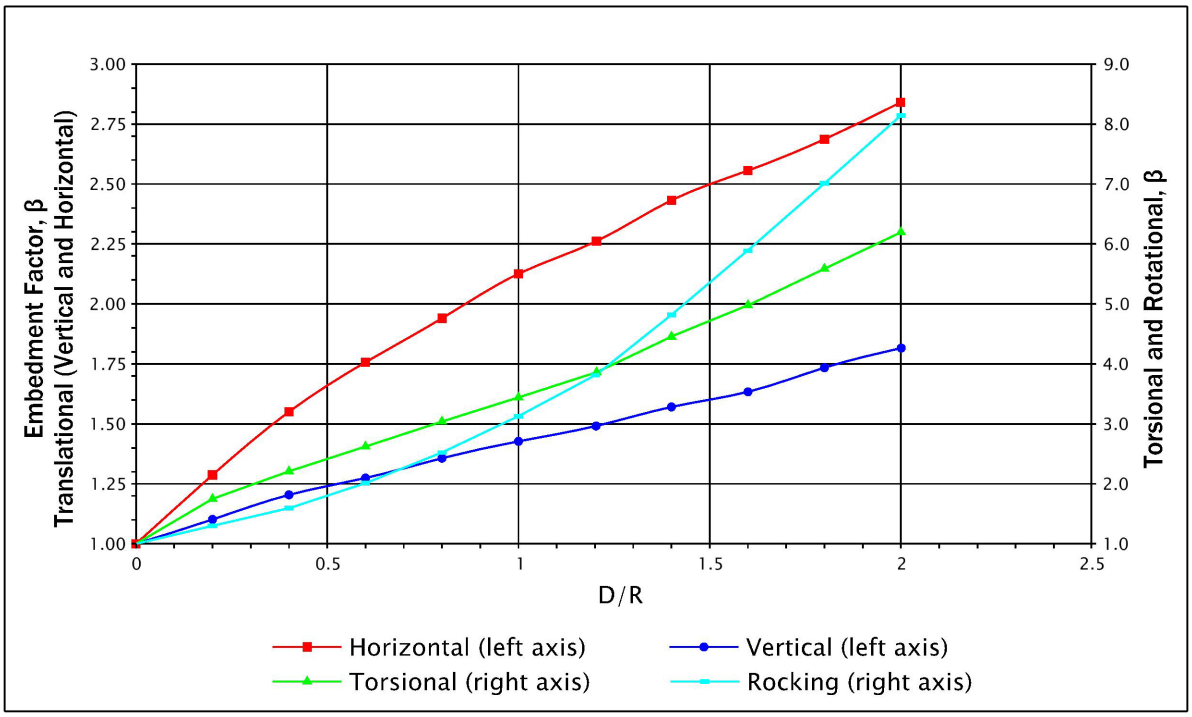


Figure 1.10.4.2.3-3 Embedment Factors for Footings, β



EQUIVALENT RADIUS:

TRANSLATIONAL:

$$R = \sqrt{\frac{4BL}{\pi}}$$

ROTATIONAL:

$$R = \left[\frac{(2B)(2L)^3}{3\pi}\right]^{1/4} \quad \text{; for x-axis rocking}$$

$$R = \left[\frac{(2B)^3(2L)}{3\pi}\right]^{1/4} \quad \text{; for y-axis rocking}$$

$$R = \left[\frac{4BL(4B^2 + 4L^2)}{6\pi}\right]^{1/4} \quad \text{; for z-axis torsion}$$

TRANSLATIONAL CAPACITIES

The use of the following values depends on the footing construction method (i.e. formed with backfill material or poured against undisturbed material). Only the passive resistance developed from the front face of the footing, combined with the shear resistance along the footing base, is considered. Column and footing side resistance is neglected.
Use the values from *Table 1.10.4.2.3-3* in the general formula:

$$Force\ Capacity = (Kp\ x\ effective\ unit\ stress\ x\ footing\ face\ area) + (Su\ x\ footing\ face\ area) + (\square\ x\ support\ reaction) + (Su\ x\ footing\ base\ area)$$

Use appropriate components depending upon soil type. Consult with the geotechnical designer for the appropriate soil values to use.

Note: $Effective\ Unit\ Stress = (Buoyant\ Unit\ Weight\ x\ Depth\ to\ middle\ of\ footing)$

C TRANSLATIONAL CAPACITIES

Consult with the geotechnical designer for recommended soil properties, groundwater levels and proper effective unit stress to use in the analysis. Also consider scour effects.

Table 1.10.4.2.3-3 Translational Capacities

Soil Classification	SPT "Nc"	Static Capacity: K_p	Static Capacity: Su (ksf)	Static Capacity: μ	Total Unit Wt. (k/ft^3)
Granular					
V. Loose	4	2.7	0	.34	0.090
Loose	10	3.0	0	.40	0.100
Medium	30	3.7	0	.47	0.115
Dense	50	4.6	0	.56	0.120
Cohesive					
Soft	4	-	0.5	-	N.A.
Stiff	8	-	1.0	-	N.A.
Very Stiff	16	-	2.0	-	N.A.
Hard	32	-	4.0	-	N.A.

Deflection required to fully activate capacities (⌾max):

Granular:

$$Loose = .06H$$
$$Dense = .02H$$

Cohesive:

$$Soft = .04H$$
$$Stiff = .02H$$

H = $Soil\ surface\ to\ middle\ of\ footing\ depth$

Specific applications may require the use of less than the full capacity due to deflection restrictions.

ROTATIONAL CAPACITIES

The bearing resistance of footings with overturning moments and eccentricity are determined using **effective** footing dimensions.

C ROTATIONAL CAPACITIES

The rotational capacity is typically determined by comparing the total footing pressure, including the overturning moment, to the factored bearing resistance provided in the Geotechnical Report, unless otherwise directed by the geotechnical designer.

TRANSLATIONAL LOAD-DEFLECTION CURVE

The following equation may be used in conjunction with the translational stiffnesses and capacities for developing a translational load-deflection curve for spread footings and pile caps.

$$P = \frac{\Delta}{\frac{1}{k_{max}} + \left[R_f \times \frac{\Delta}{P_{ult}} \right]}$$

where: P = *Load at deflection* Δ

$Pult$ = *Ultimate passive force (neglect base shear for pile caps)*

$kmax$ = *Initial stiffness*

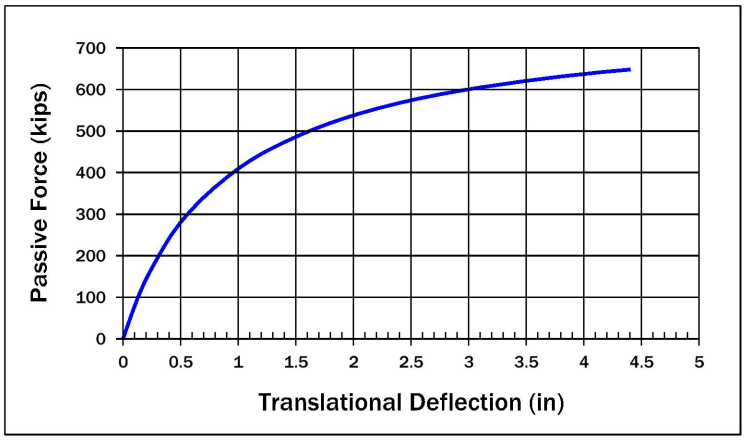
Rf = *Ratio between the actual and the theoretical ultimate force.* Rf can be determined by substituting Δmax from the previous section for Δ and $Pult$ for P in the above equation and solving for Rf.

Δ = *Translational deflection, inches*

C TRANSLATIONAL LOAD-DEFLECTION CURVE

An example of the use of this equation is given in *Figure 1.10.4.2.3-4*. This graph represents the form of the equation only.

Figure 1.10.4.2.3-4 Pile Cap Stiffness

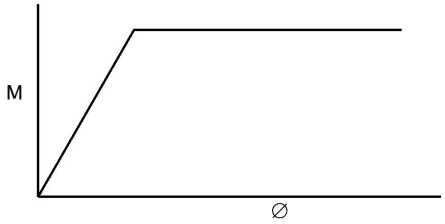

Passive Force (kips)
700
600
500
400
300
200
100
0
0
0.5
1
1.5
2
2.5
3
3.5
4
4.5
5
Translational Deflection (in)

ROTATIONAL LOAD-DEFLECTION CURVE

Use the initial stiffness up to the capacity limit. The curve form is show in *Figure 1.10.4.2.3-5.*

C ROTATIONAL LOAD-DEFLECTION CURVE

Figure 1.10.4.2.3-5 Rotational Load-Deflection Curve Form



1.10.4.2.4 Pile Supported Footings and Abutments

Use translational and rotational springs for pile supported footings in both the longitudinal and transverse directions. This approach is recommended in cases where seismic loading is the controlling factor in the structural frame analysis. Springs may also be used to model pile supported footings in non-seismic conditions at the designer's discretion. Nominal resistances may be used for both non-seismic and seismic design conditions unless otherwise recommended by the geotechnical designer.

In cases where seismic loading is not the maximum group loading for the structure, the stiffnesses and nominal lateral resistances given in the following tables are acceptable for most design cases, provided the site conditions generally satisfy the assumptions made in developing these values. In general, for soils with *Nc* values less than four, the pile translational stiffness should be evaluated using a soil response program such as LPILE programs and the geotechnical designer should be consulted for further guidance.

The use of battered piles is generally discouraged due to the greatly increased stiffness contribution from the battered piles. This in turn can result in excessive battered forces and induce undesired or unrealistic uplift forces in adjacent piles. In lieu of battered piles, it is recommended to use vertical piles throughout the footing.

Refer to the seismic design example problem for further clarification.

TRANSLATIONAL STIFFNESSES

Normally the translational stiffness should include the lateral pile stiffnesses (total pile group stiffness) plus the passive soil stiffness on one side of the footing. Typically, a single lateral pile-head stiffness is estimated from either the pile-top, load-deflection curve generated by LPILE soil response program output or from pile stiffness values given in the following tables. This single pile-head stiffness is then multiplied by the number of piles in the group and the

resulting group stiffness value is then multiplied by a group reduction factor depending on pile spacing. Instead of using a group reduction factor, account for pile group effects using p-y curve multipliers as described under **Pile Group Reduction Factors and p-y Multipliers**.

Pile cap, or footing, stiffnesses should be developed using the methods described under **Spread Footings**, except the soil stiffness contribution along the base of the pile cap should be neglected. This is accomplished by calculating the stiffness of the pile cap (footing) at the ground surface ($D = 0$) and subtracting this value from the stiffness calculated for the embedded pile cap footing. The resulting stiffness curve is then combined with the pile group stiffness curve as described in **Translational Load-Deflection Curve**.

SEISMIC CONTROLLED LOADING CONDITION – EXTREME EVENT LIMIT STATE

The pile-head translational stiffness curve is generated using a soil response program such as LPILE using soil input parameters supplied by the geotechnical designer. Pile head boundary conditions (fixed, free or fixed-translational) must be assigned by the designer. Refer to the LPILE computer program manuals. This method is shown in *Figure 1.10.4.2.4-1*.

NON-SEISMIC LOADING CONDITIONS

Use the following pile stiffnesses for non-seismic loading conditions provided the site conditions generally satisfy the assumptions given.

Table 1.10.4.2.4-1 Pile Translational Stiffness (kip/inch) – HP Piles

Axis - W=Weak S=Strong	**SPT "Nc"***	**HP 10x42**		**HP 12x53**		**HP 12x74**		**HP 14x89**		**HP 14x117**	
Granular		W	S	W	S	W	S	W	S	W	S
V. Loose	4	5	8	6	10	7	11	9	13	10	14
Loose	10	12	14	12	18	14	20	16	24	18	24
Medium	30	16	20	18	27	20	30	25	38	28	41
Dense	50	25	34	29	44	31	46	40	61	44	64
Cohesive											
Soft	4	2	2	2	3	2	3	3	4	3	4
Stiff	8	4	6	5	7	6	8	7	9	7	9
Very Stiff	16	8	10	9	12	10	13	12	15	12	16
Hard	32	14	19	17	22	18	24	21	27	23	30

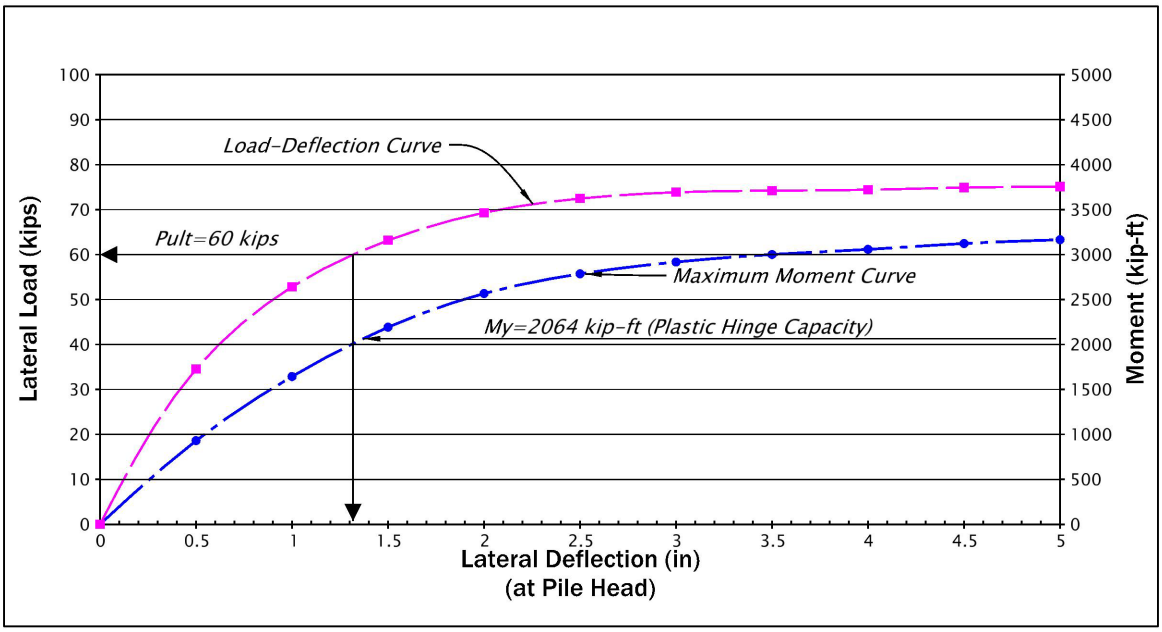
Table 1.10.4.2.4-2 Pile Translational Stiffness (kip/inch) – Pipe Piles

Pipe Piles	**SPT "Nc"***	**12x 0.25**	**12x 0.38**	**16x 0.38**	**16x 0.50**	**24x 0.38**	**24x 0.50**
Granular							
V. Loose	4	7	8	11	12	20	22
Loose	10	14	15	20	21	33	37
Medium	30	20	23	29	34	48	57
Dense	50	32	37	46	54	81	87
Cohesive							
Soft	4	2	3	3	3	4	4
Stiff	8	6	7	8	9	11	12
Very Stiff	16	10	11	13	14	18	18
Hard	32	18	20	24	26	34	36

Table 1.10.4.2.4-3 Pile Translational Stiffness (kip/inch) – Prestressed Piles

Prestressed Piles	**SPT "Nc"***	**12 inch**	**14 inch**	**16 inch**
Granular				
V. Loose	4	8	8	11
Loose	10	12	14	19
Medium	30	22	24	28
Dense	50	34	38	45
Cohesive				
Soft	4	3	3	3
Stiff	8	7	7	8
Very Stiff	16	12	12	14
Hard	32	22	23	26

Figure 1.10.4.2.4-1 Example of Top of Pile Load-Deflection Curve (Translational Spring)



TRANSLATIONAL CAPACITIES

The base shear resistance of pile supported footings, or caps, is typically not included in calculating the nominal passive resistance. The same equation used for determining the nominal translational capacity of footings should be used for pile caps, neglecting all base shear resistance. The nominal passive resistance of pile caps can be used for both seismic and non-seismic design conditions.

Use the nominal resistances in the following table for non-seismic loading conditions provided the site conditions generally satisfy the assumptions given.

Table 1.10.4.2.4-4 Nominal Pile Translational Resistances (kip/inch) – HP Piles

H-piles W=Weak S=Strong	**SPT "Nc"***	**HP 10x42**		**HP 12x53**		**HP 12x74**		**HP 14x89**		**HP 14x117**	
Granular	--	W	S	W	S	W	S	W	S	W	S
V. Loose	4	12	21	14	25	25	43	29	50	41	69
Loose	10	13	23	16	27	28	48	33	55	46	82
Medium	30	16	26	17	31	31	53	37	62	51	86
Dense	50	17	29	20	34	34	59	41	69	57	93
Cohesive											
Soft	4	16	25	17	28	29	47	34	53	45	69
Stiff	8	20	34	22	37	38	63	43	70	59	94
Very Stiff	16	24	43	25	47	49	83	55	90	76	122
Hard	32	30	54	29	58	58	104	63	113	92	155

*The **Nc** values to use are the averaged **Nc** values over a depth of 8 to 10 pile diameters (8D to 10D).

Table 1.10.4.2.4-5 Nominal Pile Translational Resistances (kip/inch) – Pipe Piles

Pipe Piles	**SPT "Nc"***	**12x 0.25**	**12x 0.38**	**16x 0.38**	**16x 0.50**	**24x 0.38**	**24x 0.50**
Granular							
V. Loose	4	22	29	43	52	85	103
Loose	10	25	32	48	57	95	113
Medium	30	29	37	54	65	107	130
Dense	50	31	41	60	71	118	143
Cohesive							
Soft	4	26	34	46	55	82	98
Stiff	8	34	44	60	72	104	126
Very Stiff	16	42	56	74	91	130	158
Hard	32	50	69	91	110	151	187

*The **Nc** values to use are the averaged **Nc** values over a depth of 8 to 10 pile diameters (8D to 10D).

Table 1.10.4.2.4-6 Nominal Pile Translational Resistances (kip/inch) – Prestressed Piles

Prestressed Piles	**SPT "Nc"***	**12 inch**	**14 inch**	**16 inch**
Granular				
V. Loose	4	12	16	23
Loose	10	15	18	25
Medium	30	18	20	29
Dense	50	20	23	31
Cohesive				
Soft	4	18	19	25
Stiff	8	22	23	31
Very Stiff	16	27	27	36
Hard	32	29	29	43

*The **Nc** values to use are the averaged **Nc** values over a depth of 8 to 10 pile diameters (8D to 10D).

The above translational stiffnesses and allowable capacities are based on the Broms' method and the following assumptions:

- Free head condition, no applied moment.
- Pile top at the ground surface.
- Level ground surface.
- One, uniform soil layer with uniform soil properties.
- No groundwater.
- Static loading, no cyclic soil degradation.
- Constant pile properties and dimensions.
- Stiffnesses are for first ½ inch deflection (initial secant modulus).
- Values are for **long** pile conditions and minimum pile embedment depths are required. When pile lengths are less than 75 percent of the assumed penetration lengths in *Table 1.10.4.2.4-7*, a separate Broms' or LPILE analysis is required.

Table 1.10.4.2.4-7 Pile Penetration Lengths

Soil Classification	"Nc"	Assumed Length (feet)
Granular	4	55
	10	50
	30	40
	50	35
Cohesive	4	55
	8	50
	16	40
	32	35

The geotechnical designer should be consulted for piles installed in conditions outside of the above stated assumptions or a LPILE analysis should be performed.

For seismic design conditions, the maximum moment capacity of the pile (My) must be calculated separately and compared to the LPILE output to determine the nominal lateral resistance and associated deflection. An example is shown in *Figure 1.10.4.2.4-1*.

TRANSLATIONAL LOAD-DEFLECTION CURVE

Translational Load Non-seismic - Deflection estimates for piles designed under non-seismic conditions should be determined using the initial pile stiffness values given in the above tables extended up to the nominal pile resistance (bilinear curve). This curve, representing the pile

group, is then added to the load-deflection curve developed for the pile cap. A LPILE analysis may also be used as described below if so desired.

Translational Load Seismic - Deflection estimates for seismic design conditions are determined from the composite load deflection curves developed by combining the pile group stiffness from the LPILE analysis with the stiffness contribution from the pile cap. An example of this procedure is provided in the section on **Load-Deflection Curves, Stiffness Iteration Analysis and Capacity Checks**.

PILE GROUP EFFECTS AND P MULTIPLIERS

The P multiplier approach, utilizing the LPILE program, is recommended to evaluate the response of pile groups subjected to lateral loads. The P multipliers are applied to standard p-y curves to account for pile group effects. *LRFD 10* should be referenced for the P multiplier values to be used in the analysis. The P multipliers are dependent upon the center to center spacing of piles in the group in the direction of loading expressed in multiples of pile diameter. The geotechnical designer should be consulted for the procedures to use in this design approach.

ROTATIONAL STIFFNESSES

Normally the rotational stiffness should only include the moment versus rotation stiffness from the pile group. The pile cap is usually considered rigid in this analysis and no additional stiffness due to soil bearing at the base of the pile cap/footing is included. Therefore, the rotational stiffness of pile caps is simply a function of pile axial compression and the pile group layout. See the example problem in the Bridge Example Designs notebook for more details. Static formulas for pile compression are typically used. The computer program APILE may also be used for a more detailed analysis of the predicted load-deflection behavior of a single, axially loaded pile. This program takes into account unusual soil conditions and the nonlinear aspects of pile-soil interaction. The geotechnical designer should be consulted for axial pile stiffnesses using the APILE program.

The following formulas for axial pile stiffness may be used in developing rotational stiffnesses for pile supported footings. For friction piles, the APILE program may also be utilized to better model axial stiffness when axial loads are greater than about half of the nominal resistance of the pile.

End bearing pile:
$Kv = \frac{AE}{L}$
Friction Piles:
$Kv = \frac{2AE}{L}$

where: $Kv = Axial\ Pile\ Stiffness\ (kN/mm)$

$A = Area\ of\ pile\ normal\ to\ load$

$L = Length\ of\ pile$

$E = Young's\ Modulus\ of\ Pile\ Material$

Compute the rotation stiffness (M vs. ∅) for a pile group as follows:

Assume a pile head deflection for the pile farthest from the pile group centroid.

Using the appropriate relation from above, determine the pile force accompanying this assumed pile head deflection. Prorate the other pile forces by their location relative to the group centroid. Piles on one side of the centroid will have positive forces and piles on the other side will have negative forces (uplift).

Determine the pile group moment by summing the product of the pile force and the pile-to-group centroid distance for all piles. This is the moment (M) required to rotate the footing through an angle of ∅. Determine the angle ∅ as the arctan of the assumed extreme pile head deflection divided by the pile-to-centroid distance.

The relation of M to ∅ is the initial rotation stiffness.

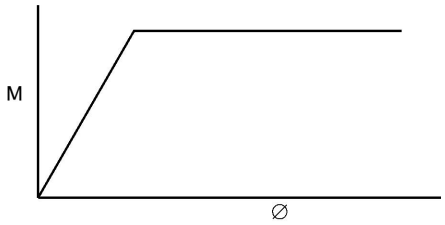
ROTATIONAL RESISTANCES

For pile supported footings, compare computed pile loads to nominal axial pile resistances for seismic cases and to factored axial pile resistances for non-seismic cases, unless otherwise recommended by the geotechnical designer.

ROTATIONAL LOAD-DEFLECTION CURVE

Use the initial stiffness up to the resistance limit. The curve form is:

Figure 1.10.4.2.4-2 Rotational Load-Deflection Curve



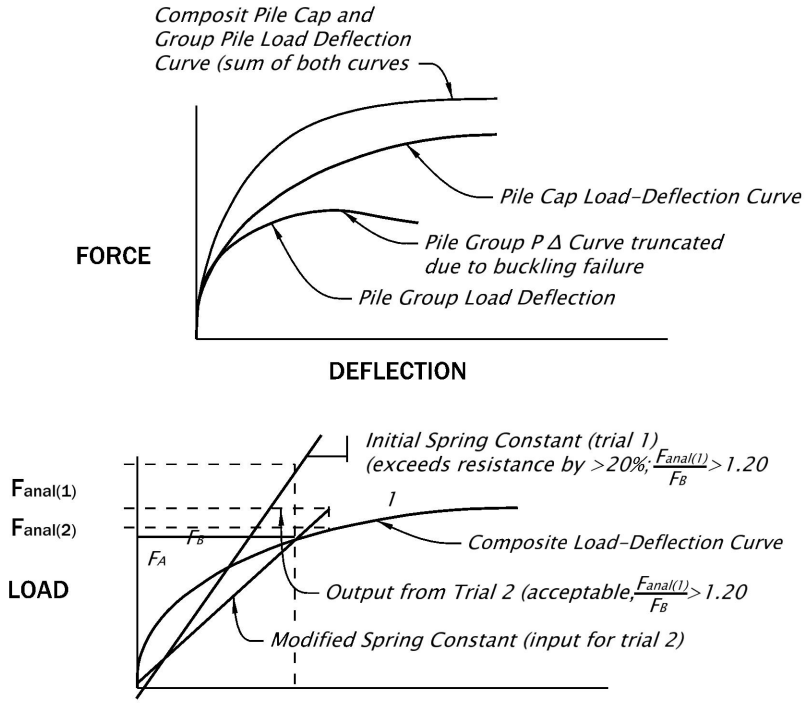
LOAD-DEFLECTION CURVES, STIFFNESS ITERATION ANALYSIS AND CAPACITY CHECKS

Using the previous information one develops a composite load-deflection relationship for each applicable support spring. Next, an initial spring constant is assumed, the structure and loading analyzed and the resulting load-deflection position compared to the initial assumption. Cycling through this process may be needed to achieve reasonable closure. See the graphical explanation below.

It is also necessary to check the required resistance against the factored or nominal resistance. Resistance factors of 1.0 are typically used in the case of seismic design, however this should be verified by the geotechnical designer. Factored resistances are used for all other cases. For the rotational capacity, this is normally done by checking the resultant forces against the maximum (nominal), effective soil bearing resistance (footings) or nominal pile resistance.

For lateral pile resistances, the nominal resistance is either the maximum determined from the LPILE analysis (based on My of the pile for seismic design), or from the tables. The nominal resistance may also be a function of maximum allowable structural deflections. When the limiting resistance is exceeded when using the initial spring coefficient then modified springs are required as shown in the graphical explanation below, *Figure 1.10.4.2.4-3*.

Figure 1.10.4.2.4-3 Modified Springs: (A) Development of Composite Load, Deflection Curve (B) Spring Iteration Process and Resistance Check



1.10.4.3 Drilled Shaft Modeling (Fully Coupled)

Programs Midas Civil or SAP2000 and LPILE can be used in an iterative approach to model a drilled shaft supported structure. The approach is to determine the approximate force magnitudes for the controlling loading and then use these forces to develop a better representation of the superstructure/shaft/soil problem. This allows a good approximation of soil stiffness non-linearity as well as the non-linearity of the shaft-soil interaction.

The following steps would be typical for drilled shaft modeling for design and checking:

- Develop a full Midas Civil or SAP2000 model (superstructure with substructure) using shaft fixity at two shaft diameters below the groundline. Using the model, run the controlling load case – typically seismic loading will be the controlling case and the worst effect, either longitudinal or transverse, will be used for the next steps.
- Develop LPILE models (shaft with soil) for each bent using the full shaft from its tip to its connection to the superstructure.
- Using the top of shaft shear and moment results from the first Midas Civil or SAP2000, load the LPILE models to develop a stiffness matrix for each shaft. This represents a condensing of the substructure/soil effect to the point of connection with the superstructure. The LPILE program can develop a stiffness matrix.

- Develop a new Midas Civil or SAP2000 model using only the superstructure and supports represented by the LPILE developed substructure stiffness matrices. Run the same controlling load case.
- Use the top of shaft shear and moment results from this latest Midas Civil or SAP2000 to again load the LPILE models to develop new substructure stiffness matrices.
- Use the latest Midas Civil or SAP2000 model with the most recent substructure stiffness matrices and again run the same controlling load case.
- Compare the results of this Midas Civil or SAP2000 with the previous Midas Civil or SAP2000 run for correlation. When the results do not correlate well, cycle through steps five and six to get better convergence. Results which change no more than 15 percent per cycle are normally sufficiently close and further cycling is not required.

1.10.5 Foundation Design

Perform foundation design in accordance with the most current version of the *AASHTO LRFD Bridge Design Specifications*. Follow the policies and guidelines described in the ODOT Geotechnical Design Manual.

Conducted subsurface investigations for all structures in accordance with the *ODOT Geotechnical Design Manual Section 4.2.*

C 1.10.5

FHWA foundation design manuals are also acceptable methods for use in foundation design.

1.10.5.1 Foundation Design Process

Coordinate and resolve differences related to the structural, hydraulic, and geotechnical aspects of foundation design prior to approval of TS&L.

See *GDM 2.7* for details on each geotechnical deliverable and the process associated with milestones or phases throughout the ODOT project delivery process.

1.10.5.1.1 Geotechnical Memo

The purpose of this memo is to support preparation of the TS&L Report by providing sufficient data for develop TS&L plans and cost estimates. The memo is to be finalized no later than 75 percent of the way through the TS&L design process. Prepare the Bridge TS&L Report and the Geotechnical Memo at the same time, with each document preparer working in close coordination and collaboration with the other document preparers.

C 1.10.5.1

The foundation and bridge designers must establish and maintain good communication and exchange of information throughout the entire bridge design process. Bring questions regarding foundation design issues to the attention of the geotechnical designer as early as possible in the design process.

See *GDM 2 and 3* for specific foundation design project workflow elements.

For most typical bridge design projects two Geotechnical Reports are provided, the Geotechnical Memo and the Geotechnical Report.

C 1.10.5.1.1

See *GDM 2.7.9* and *GDM 20.4.2* for what information is provided in the Geotechnical Memo. See *BDM 2.9.5* for additional information on the Bridge TS&L Report.

1.10.5.1.2 Geotechnical Report (GTR)

This report is provided with the Preliminary Design phase deliverables.

In order to conduct a proper foundation investigation and complete this report the bridge designer needs to provide the geotechnical designer the following information:

- Bent locations and layout.
- Proposed roadway grade (fill heights) (in collaboration with the roadway designer).
- Anticipated foundation loads.
- Foundation size/diameter and depth required to meet structural needs.
- Allowable structure settlements (total and differential).
- Proposed retaining wall locations.
- Estimated scour depths (from Hydraulics Report).
- Construction or environmental constraints that could affect the type of foundation selected.

Distribute the final Plans and Special Provisions for the structure to the geotechnical designer for review to make sure they are consistent with the design recommendations provided in the Geotechnical Report. Resolve any discrepancies. Addendums to the Geotechnical Report are issued when necessary, per *GDM 2.7.14*.

C 1.10.5.1.2

The GTR provides the final foundation design recommendations for the structure and a Geotechnical Data Sheet for each structure. Refer to *GDM 20.4.3* for contents of the Geotechnical Report and *GDM 20.7.3* for all Geotechnical Files.

Construction recommendations are included along with project specific specifications, which are to be included in the contract Special Provisions. Seismic foundation design recommendations are provided including site characterization and soil coefficients, estimated ground acceleration and any liquefaction mitigation measures considered necessary (See *BDM 1.17*).

1.10.5.2 Bridge Foundation Records

As-constructed records on existing bridge foundations may be found in the Salem Bridge Engineering Office from the following sources:

- Pile Record books.
- **As-constructed** Bridge Plans (available through ODOT intranet).
- Microfilm Construction Records.
- Bridge Maintenance Files.

C 1.10.5.2

See *GDM 3.2.2.2.2 and 3.2.2.2.3* for information on office research for bridge foundation work.

1.10.5.3 Spread Footing Foundation Design

Consider spread footings early on in the design process as a possible economical foundation option when the foundation conditions are suitable. The design of spread footings is an interactive process between the geotechnical and structural designers.

When spread footings are recommended, the design recommendations per *GDM 20.6.1* are provided in the Geotechnical Report.

C 1.10.5.3

See *GDM 17.4* for spread footing design requirements.

See *BDM 1.10.5.8* for scour design requirements.

Refer to *GDM 20.6.1* for additional information on these design recommendations.

1.10.5.4 Pile Foundations

When spread footings are unsuitable or uneconomical for foundation support, consider driven piles. Consult with the geotechnical designer to determine the most appropriate pile type, size and bearing resistance to support the desired pile loads.

The geotechnical designer provides the nominal resistances required to achieve the required resistance for each limit state per *GDM 20.6.2.1*.

C 1.10.5.4

The most common pile sizes used on ODOT projects are listed in *GDM 17.5.4*.

The structural resistance is ($0.9f_y$ x Sectional area of pile).

When steel piles are installed under environmental conditions meeting corrosion criteria as described in *BDM 1.10.5.4.1(10)*, specify a method of corrosion protection for the steel piles or determine required cross-section of the steel piles throughout the minimum design life. Corrosion rates for sacrificial thickness are specified in *BDM 1.10.5.4.1(10)*.

Do not use precast prestressed or timber piles.

1.10.5.4.1 Piling Considerations

(1) PILE RESISTANCE

Nominal pile resistances will be provided according to *AASHTO LRFD* design procedures. The resistance factor will be provided according to *GDM 20.6.2.1*. Confer with the geotechnical to make sure the pile types and sizes selected take full advantage of the available geotechnical and structural resistances when possible.

(2) DOWNDRAG LOADS

Pile downdrag loads, due to soil settlement other than that caused by dynamic (seismic) loading, are added to the factored vertical dead loads on the foundation in the Strength Limit state. Do not include transient loads with the downdrag loads in either the strength or service limit state calculations. Consider downdrag loads resulting from liquefaction or dynamic (earthquake) induced soil settlement in the Extreme Event limit state pile design. Do not combine downdrag loads resulting from soil liquefaction with those caused from static loading in the Extreme Limit state analysis.

C 1.10.5.4.1

C (1)

C (2)

See *GDM 20.6.2.2* for downdrag information provided in the Geotechnical Report and *GDM 17.1.2.6* for design standards of downdrag loads.

(3) UPLIFT CAPACITY

In general, the uplift resistance is the same as the pile friction (side) resistance. Refer to *GDM 20.6.2.4* and the Geotechnical Report for resistance factors and factored uplift resistances. Consider friction resistance in downdrag zones available for uplift resistance.

C (3)

Consult the geotechnical designer regarding the ability of the piles to resist uplift forces under various loading conditions (static or dynamic).

4) MINIMUM PILE TIP ELEVATION

Show the required pile tips elevations on the plans and label as **Pile Tip Elevation for Minimum Penetration** for each bent.

C (4)

Large lateral loads due to seismic, or other, conditions may result in the need for additional piling, or larger piles, in order to satisfy lateral deflection criteria or other requirements. This may in turn result in individual axial pile loads being much less than the maximum factored resistances available (either geotechnical or structural). Conversely, when pile tip elevations are needed to meet scour, uplift, or other requirements, the piles may need to be driven through very dense materials to nominal resistances much higher than needed for supporting just the axial loads. Close communication is needed between the geotechnical and bridge designers to determine the most economical foundation design under these conditions. See *GDM 17.5.1* and *20.6.2.6* for additional information. See *BDM 1.10.5.8* for scour design requirements.

(5) PILE GROUP SETTLEMENT

Compare pile group settlement to the maximum allowable settlement. Adjust pile depths or layout when necessary to reduce the estimated settlement to acceptable levels.

C (5)

See *GDM 17.5*.

(6) PILE GROUP EFFECTS

For pile group lateral load analysis use the P-multiplier methods described in *LRFD* and the FHWA Manual on the *Design and Construction of Driven Pile Foundations*.

C (6)

See *GDM 17.5.*

(7) PILE SPACING

Use a minimum spacing of 3 feet for piles placed underwater. Use a minimum spacing of 2.5B for above water piles.

C (7)

(8) PILE TIP TREATMENT

Where pile tip reinforcement is required, specify commercial cast steel points or shoes.

Where closed-ended pipe piles are required, specify a welded end plate or a welded end plate with stiffeners having the same diameter as the pipe pile. Provide dimensions for the end plate and stiffeners on plan sheets for each project.

Deviation from the minimum dimensions in *Table 1.10.5.4.1-3 and 1.10.5.4.1-4* requires project specific shell and buckling analyses and driving stress from the geotechnical designer.

Use the minimum dimensions *Table 1.10.5.4.1-3* and *Table 1.10.5.4.1-4* with *Figures 1.10.5.4.1-1 and Figures 1.10.5.4.1-2.*

C (8)

An analysis was performed for a range of pipe pile sizes which verified sufficiency and the minimum dimensions for the end plate and stiffeners. This analysis was conducted using $0.9fy$ as the maximum load, which is the maximum stress the pile will undergo and what is allowed during pile driving.

See *GDM 17.5.4* and *20.6.2.7* for additional information.

The figures are available online in the standard details at the following location Welded Plate Details. These details are available for project specific use.

Figure 1.10.5.4.1-1 Welded End Plate

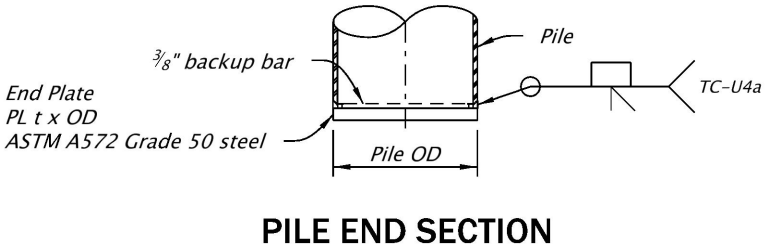


Figure 1.10.5.4.1-2 Welded End Plate with Stiffeners

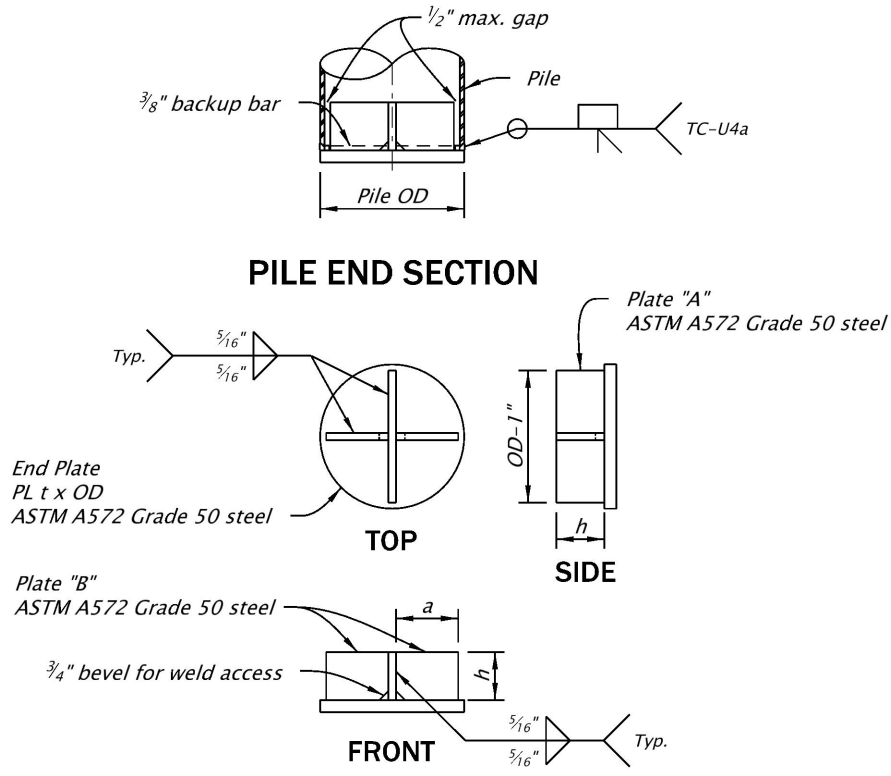


Table 1.10.5.4.1-3 Minimum Dimensions for Welded End Plates – No Stiffeners

Pipe Pile Size	Gr 2 Pipe	Gr 3 Pipe
	t	t
PP12 ¾ x 3/8	2.25	2.5
PP12 ¾ x ½	2.50	2.75
PP16 x 3/8	2.50	2.75
PP16 x ½	2.75	3.00
PP18 x 3/8	2.50	2.75
PP18 x ½	3.00	3.25
PP24 x ½	3.25	3.75

Table notes:
t = thickness; h = height; a = length

Table 1.10.5.4.1-4 Minimum Dimensions for Welded End Plates – With Stiffeners

	Gr 2 Pipe			Gr 3 Pipe		
Pipe Pile Size	End Plate	Stiffeners (Plates A & B)	Plate B Length	End Plate	Stiffeners (Plates A & B)	Plate B Length
-	t	h x t	a	t	h x t	a
PP12 ¾ x 3/8	1.00	4 x 0.375	5.500	1.25	4 x 0.500	5.250
PP12 ¾ x ½	1.00	4 x 0.500	5.375	1.75	4 x 0.500	5.125
PP16 x 3/8	1.00	6 x 0.375	7.125	1.25	6 x 0.375	6.938
PP16 x ½	1.25	6 x 0.375	7.000	1.50	6 x 0.500	6.750
PP18 x 3/8	1.00	6 x 0.500	8.125	1.25	6 x 0.500	7.875
PP18 x ½	-	-	-	-	-	-
PP24 x ½	-	-	-	-	-	-

Table notes:
t = thickness; h = height; a = length

(9) PILE FOUNDATION DESIGN RECOMMENDATIONS

The geotechnical designer will provide final foundation recommendations in the Geotechnical Report, or earlier in the design process as needed. See *GDM 20.6.2.1* for the recommendations that will be provided.

C (9)

(10) PROTECTION FOR STEEL PILING

Assess all steel piling used in permanent structure applications for corrosion potential and design for the long term effects of corrosion. Steel pilings do not need a corrosion allowance when they are used in temporary works.

Recommended pile corrosion assessment measures are provided below for corrosive and non-corrosive environments.

C (10)

Reference *LRFD 10.7.5* for design requirements and guidance regarding steel pile corrosion assessment and protection. The design requirements and guidance provided in *LRFD* are further defined and supplemented in this section and the *ODOT GDM*.

Buried steel piling corrosion requires oxygen and moisture. Corrosion risk is typically greatest from the bottom of pile cap/footing down to 3 feet below the lowest recorded/measured ground water elevation.

CORROSIVE AREAS

Protect all steel piling in corrosive areas with one or more corrosion protective system.

C CORROSIVE AREAS

When there is concern of microbial corrosion, conduct a Biological Activity Reaction Test (BART). A good warrant for this test is a site in proximity showing evidence of microbial corrosion. Contact the Bridge Corrosion Engineer with questions.

The following areas are considered corrosive areas for the purpose of this section:

- Marine areas, defined in *BDM 1.2.4.1.*

Project sites located beyond marine areas may still have a significant potential for pile corrosion depending on site specific conditions. For example, if brackish water is present, or there are indications of potential corrosive conditions. Steel piling in waterways that extend above round such as in estuaries, lakes or streams may also be subject to significant corrosion.

- Pile in the atmospheric environment (as defined below) on Interstates and in snow and ice areas, as defined in *BDM 1.2.4.1*.

Consider exposure due to runoff, joint failure, splashing or overspray. This does not apply to sites where piles do not have any exposure (e.g. piles completely buried behind an MSE wall.)

Investigate the following corrosive sites with a thorough soil and groundwater testing program outlined in the *GDM*:

- Landfills or fill materials composed of cinders, ash or slag.
- Sources of mine or industrial drainage (acidic groundwater).
- Sites with stray electrical currents, such as electric (DC) transit systems, or high voltage power lines.

The circumstances or site conditions listed are highly corrosive and indicate the potential for severe corrosion. Corrosion rate depends primarily on electrochemical properties of the surrounding soil. Existing fill (especially human-made fill); disturbed native soils; and native soils containing cinders, slag, or ash are relatively likely to be considered aggressive compared other native soils.

Both surface and subsurface materials should be characterized to ensure representation of the strata at the site. Do not take water samples in flowing streams and rivers when the water level is elevated due to storm conditions as they may dilute the chemical concentrations.

Guidance on the extent of site investigations, including the soil sampling and testing required for corrosion assessment, are presented in the *ODOT GDM*.

Consider an area corrosive when it has one or more of the following conditions:

- Soil resistivity is less than 2000 ohm-cm.
- Soil pH is less than 5.5.
- Sulfate concentrations are greater than 1000 ppm.
- Chloride concentrations are greater than 500 ppm.

Test results should be representative of the environment, not a specific test location. Consult with the geotechnical engineer when needed.

NON-CORROSIVE AREAS

Non-corrosive areas are all locations not designated as corrosive areas per *BDM 1.10.5.4.1*.

Design buried steel piling in non-corrosive areas with a minimum sacrificial steel thickness defined in *Table 1.10.5.4.1-5*.

C NON-CORROSIVE AREAS

In non-corrosive areas, piles that are permanently buried and are always below the water table have a low potential for steel corrosion. Often no corrosion protection system is warranted.

Additional guidance and background information on the corrosion of steel piling in non-corrosive areas can be found in NCHRP Report 408 and *AASHTO R-27-01 (2015)*. The minimum pH criteria of 4.0 recommended in these reference documents does not supersede the 5.5 value used in *LRFD*.

CORROSION PROTECTIVE SYSTEMS

Provide corrosion protection systems using one or more of the following: sacrificial thickness, protective coatings, cathodic protection.

Investigate the use of protective coating systems in the following order of preference: sacrificial thickness, protective coating, cathodic protection.

C CORROSION PROTECTIVE SYSTEMS

Other protective measures, such as concrete encasement or special steel alloys, may also be considered on a case by case basis. Contact the Bridge Corrosion Engineer for additional guidance.

Each system has advantages and disadvantages, as described below. Project budget typically drives the selection of a corrosion protection system. Some other factors can include, but are not limited to: site corrosion potential, driving forces, structure loading, site environmental constraints, aesthetics and site accessibility.

Combining corrosion protection systems may extend the service life of the system. Pair this with a cost effective approach.

(1) Sacrificial Steel Thickness

Use a sacrificial steel thickness to account for steel section loss over the design life of the structure in non-corrosion and in corrosive areas only in soil embedded zones.

At a minimum, specify a thickness loss of 1/16 inch to account for possible corrosion loss occurring in the steel piles. Corrosion rates for use in determining thickness loss are specified in *Table 1.10.5.4.1-5*.

Determine thickness loss over a minimum design life based on project design criteria. Use the reduced thickness in the pile design.

C (1) Sacrificial Steel Thickness

Sacrificial thickness allows for more rehab options in the future. It is also the most constructable option.

Do not use when the increase in pile size becomes uneconomical or unreasonable to for pile driving. Appropriate sites to use sacrificial thickness include, but are not limited to:

- Sites with variable or marginal test results (e.g. only failing one criteria).
- Sites with limited or deep exposure to native soil.

The corrosion rates in *Table 1.10.5.4.1-5* are based on information in the WSDOT *BDM, 2014, CALTRANS March 2018, Corrosion Guidelines, version 3.0*, the *FHWA NHI-16-009 Design and Construction of Driven Pile Foundations - Volume I*, and the *Washington State Ferries Terminal Design Manual, 2016.*

The potential for scour need not be considered when choosing a design corrosion rate as it relates to zones of exposure. It is assumed any significant scour would be repaired and the applicable zone of a structural element would not be changed.

Table 1.10.5.4.1-5 Steel Pile Corrosion Rates (inch/year)

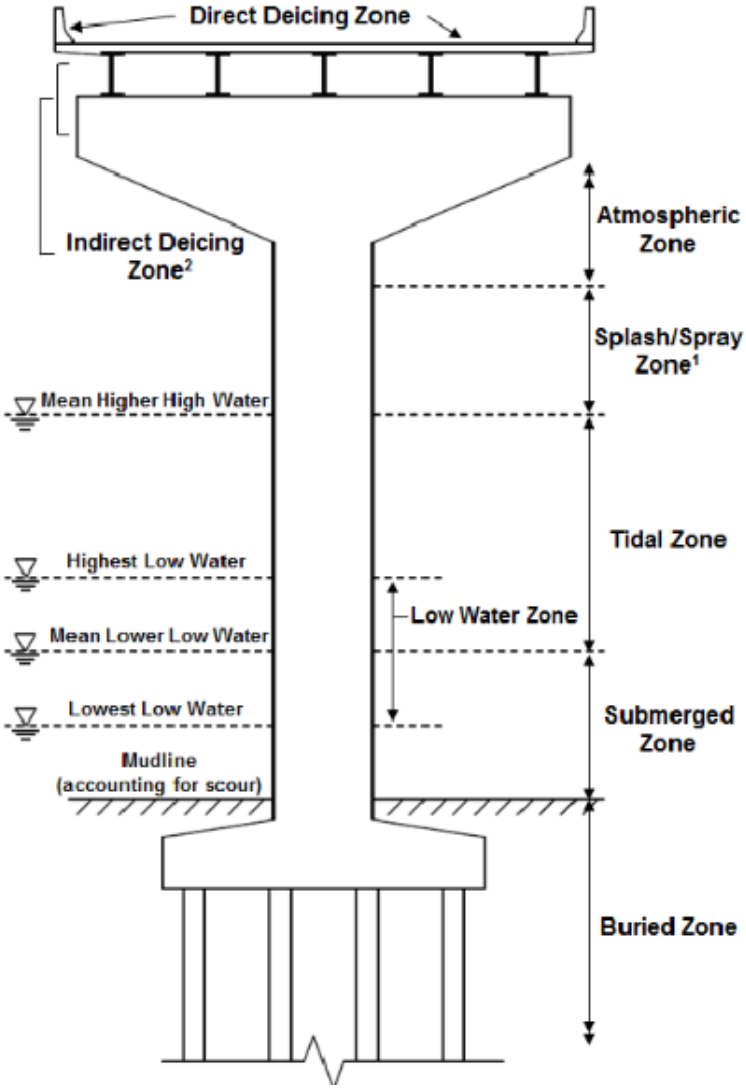
Pile Environment	Corrosive Areas	Non-Corrosive Areas
Soil embedded zone (undisturbed soil)	0.001	0.0005*
Soil embedded zone (fill or disturbed natural soils)	0.0015	0.00075*
Immersed zone	0.003	0.0015
Tidal zone	0.004	—
Splash zone	0.006	—
Atmospheric	0.002	0.001

Tables notes:
* 1/16 inch sacrificial steel thickness is suitable for pipe piles, shells, and casings with a service life of 75 years or less.

Definitions of the terms in *Table 1.10.5.4.1-5* are as follows and depicted in *Figure 1.10.5.4.1-1*:

- Immersed zone - continuously submerged in water. Immersed non-corrosive: environments with fresh water or are tested and found not to meet the marine area or corrosive area values.
- Tidal zone - between the Mean Low Water (MLW) and the Mean High Water (MHW).
- Splash zone - above the MHW plus five additional feet or as otherwise determined for a specific site.
- Atmospheric - above the splash zone or above ground line as applicable.

Figure 1.10.5.4.1-1 Typical exposure conditions of bridge elements



Double the corrosion loss for steel H-piling.

Steel H-piles have two surfaces on either side of the web and flanges that are exposed to corrosive conditions.

Apply the corrosion allowance only to the exterior surface of the pile for pipe piles, shells, and casings.

The interior of the pile will not be exposed to sufficient oxygen to support significant corrosion.

(2) Protective Coatings

Two types of coating systems are available: a 3-coat system and a 4-coat system.

C (2) Protective Coatings

A coatings effective life is generally assumed to be 15 years and there is no system maintenance. Coating is done off-site so installation does not require ground access to individual piles. Specify an appropriate coating system to withstand damage from handling and driving.

Calculate section loss outside of coating service life and provide sacrificial thickness based on the section of pile with the highest corrosion rate not readily accessible to maintenance.

Areas not accessible to maintenance include embedded and immersed zones. Other may include steep and uneven terrain or sites with excessive access coordination and costs such as urban congestion, public land use, or railroads/utilities. When sacrificial thickness is not practical or too expensive, consider pile geometry that is conducive to future cathodic protection. (i.e. Choose pile configurations which minimize soil/water contact area, consider bracing details which reduce the number of anode beds, identify locations and/or install electrical attachment details, etc.).

Account for variability in estimated tip elevations when designing coating lengths. This may increase the length of required coating.

Specify the 3-coat protective coating system for piles in direct contact with water. Coat piles full length at bents with pile tips within 25 feet of groundline.

The 3-coat protective coating system is comprised of a zinc-rich primer followed by two coats of moisture-cured urethane/urethane-tar. It comes in very limited color options, contact the Senior Structure Coating Engineer for additional information.

Specify 4-coat protective coating system based on site preferences. Coat all other piles from final cutoff elevation to 25 feet below groundline.

The 4-coat protective coating system includes the same 3-coat protective coating system followed by a top coat. The top coat is primarily an aesthetic coating which provides more color options, but limited additional protective qualities.

Use both *SP 00520* and *SP 00594* for protective coatings.

SP 00520 identifies where and what kind of coating system is to be used. *SP 00594* provides the preparation, application, materials, testing, measurement and payment for the specified coating system.

(3) Cathodic Protection

Use cathodic protection in conjunction with protective coatings on new steel piling installations to minimize anode consumption.

Use cathodic protection on existing steel pilings installations to address existing corrosion concerns.

C (3) Cathodic Protection

Cathodic protection should be considered in the most highly corrosive areas in new construction. Installation requires direct access to piles. Cathodic protection is a good fit for sites with stray currents or when driving conditions are not conducive to protective coatings.

Cathodic protection does require regularly scheduled inspections of the exposed piling and maintenance, but if it is properly maintained it has the longest service life of the three systems.

Guidance for design and installation of anode systems for pilings in marine waters is provided by *NACE Standard SP0176*. Guidance for design and installation of anode systems for pilings in soil is provided by *NACE Standard SP0169*.

Make the electrical connection to the piling by brazing a brass stud on the downstream side of the piling. Provide the following project note on the plans:

> Remove coating from piling surface (approximately 4 by 4 inch area) and install ½ by 13 by 2 inch brass stud using an approved brazing process. Provide 2 brass nuts and 2 brass washers with stud. Recoat piling surface with protective coating after installation of brazed stud.

1.10.5.4.2 Piling Details

(1) STEEL PILE FOOTING EMBEDMENT TO DEVELOP FIXITY

Two types of pile head connection, i.e., pinned and fixed, are allowed for assumed boundary conditions to analyze piles or pile groups embedded in a reinforced concrete cap. This article describes how to achieve the boundary condition through pile embedment depth and steel reinforcing details in the pile cap.

For bridges in SDC A, when there is insignificant lateral load and tension capacity is not needed, pile embedment depth into the pile cap of 6 inches may be used. Assume a pinned boundary condition in structural analysis. Do not specify pile anchoring details, when pinned head piles are used.

For bridges in SDCs B, C, and D, when lateral load resistance is required, piles must be embedded adequately into pile caps with sufficient confinement reinforcing to develop the full bending moment capacity of the pile section.

Minimum required embedment depths for steel piles, cap dimensions, and steel reinforcing are shown in *Table 1.10.5.4.2-1*.

C1.10.5.4.2

Previous tests showed that a small embedment depth in the pile cap, such as 12 inches, can develop partial fixity. This partial fixity makes it difficult to accurately determine forces in the pile or displacement of the pile bent system. To reduce the uncertainty of the connection type, a very short embedment depth of 6 inches was chosen for the pinned connection assumption.

Designers may utilize the pinned boundary condition for some design situations to achieve desired displacement or reduce resultant force developed in pile section near the cap. For bridges located in low seismic zones, pinned head details can be specified. The shallow pile embedment simplifies steel reinforcing details in the pile cap and allows for easier pile cap construction. Research showed that a small size and number of reinforcing bars welded on the perimeter of pile extending into the pile cap could develop partial fixity.

Table 1.10.5.4.2-1 Minimum Pile Embedment and Design Details for Pile Cap

Shape	M_p (K-ft)*	λ_{mo}	M_{po} (K-ft)	L_{emb} (in)**	Minimum Cap Width (in)	Minimum Cap Height (in)	U-bar *** # legs
PP 12¾ x 3/8	285.6	1.20	342.7	20	36	40	4#7
PP 12¾ x ½	355.4	1.20	426.5	22	36	40	4#7
PP 16 x 3/8	478.0	1.20	573.6	22	36	40	6#7
PP 16 x ½	590.9	1.20	709.1	24	36	42	6#7
PP 18 x 3/8	625.7	1.20	750.8	24	40	42	6#7
PP 18 x ½	770.2	1.20	924.2	26	40	48	6#7
PP 20 x ½	978.2	1.20	1173.8	28	44	48	6#8
PP 24 x ½	1483.2	1.20	1779.8	32	48	52	8#8
HP 10 x 42	221.4	1.20	265.7	20	36	36	4#7
HP 10 x 57	304.8	1.20	365.8	22	36	40	4#7
HP 12 x 53	339.2	1.20	407.0	22	36	40	4#7
HP 12 x 74	481.3	1.20	577.6	26	36	42	4#7
HP 12 x 89	578.9	1.20	694.7	28	40	48	4#7
HP 12 x 117	782.8	1.20	939.4	32	40	52	6#7
HP 14 x 89	669.2	1.20	803.0	28	40	48	6#7
HP 14 x 117	889.2	1.20	1067.0	32	40	52	4#8
HP 16 x 141	1210.0	1.20	1452.0	36	44	54	6#8
HP 16 x 183	1599.6	1.20	1919.5	40	44	60	6#8
HP 18 x 204	1984.6	1.20	2318.5	42	48	62	8#8

Table notes:

* The plastic moment capacity of piles, M_p, was calculated using expected material properties, i.e. 1.3 x f'_c for concrete (f'_c = 3.3 ksi), 1.1 x F_y for steel piles (F_y = 45 ksi for pipe piles and F_y = 50 ksi for H piles). The moment capacity of pipe piles was based on a pipe pile section filled with concrete and was calculated according to the plastic stress distribution method *LRFD C6.12.2.3.3*. The moment capacity of H piles was obtained about the section's strong axis.

**The minimum embedment length was calculated using specified minimum material properties.

***Example of U-bar details is shown in *Figure 1.10.5.4.2-1*.

The pile embedment depth shown in *Table 1.10.5.4.2-1* is a minimum required embedment depth regardless of seismic design category or stress level in the steel pile.

The plastic moment M_p and the overstrength plastic moment M_{po} may be used for steel pile and pile cap designs for the Extreme Event limit state. For strength limit state design, use specified minimum material properties to calculate pile plastic moment capacity.

Laterally loaded pile design with a fixed pile head is typically governed by the Extreme Event load combinations. Therefore, load and resistance factors for the Extreme Event limit state were used to determine the embedment depth shown in the table. The embedment depths were calculated considering equilibrium of the resultant of the upper compression block developed at the face of the piles against concrete taken to be equal to that of the lower compression block caused by the overstrength plastic bending moment as described in *FHWA/MT-13-001/8203 (MDT) report (2013)*.

When design parameters and details, such as pile material properties, pile section, additional reinforcing steel inside the pipe pile, etc., deviate from the parameters used for the minimum required embedment depth and details shown in *Table 1.10.5.4.2-1*. The embedment depth, L_{emb}, of steel piles can be calculated as follows:

$$L_{emb} = \sqrt{\frac{2 \times M_{po}}{D_{pile} \times \beta_1 \times (1 - \frac{\beta_1}{2}) \times \varphi_b \times \alpha \times f'_c}}$$

where,

M_{po} = overstrength plastic moment of pile taken as λ_{mo} x M_p (kip-in)

λ_{mo} = overstrength magnifier = 1.20

D_{pile} = diameter or minimum width of pile (in.)

β_1 = stress block factor varied with concrete strength

ϕ_b = resistance factor for bearing on concrete = 0.70

α = concrete confinement factor = 1.50

f'_c = concrete strength (ksi)

Several methods for calculating flexural strength of pipe pile filled with concrete are described in *AASHTO Guide Specifications 7.6.2* (Exact Geometry and Approximate Geometry methods) and *LRFD 6.12.2.3.3* (the plastic stress distribution and strain compatibility methods). It was found that the plastic stress distribution method yielded slightly larger flexural strength of pipe pile filled with concrete compared to the other methods. The Approximate Geometry method generally resulted in the least flexural strength and may conservatively be used for pile design.

When bending moment demand at the pile-to-pile cap location is less than 0.60 M_p, unfilled pipe piles may be used. Specify the embedment depth shown in *Table 1.10.5.4.2-1*.

Specify concrete-filled pipe piles for piles resisting bending moment demand at the pile-to-pile cap location equal to or greater than 0.60 M_p or when the embedded piles are located in SDC C or D. To ensure that the full bending moment capacity of the pile section is developed, flexural capacity of the cap must be equal to or greater than the overstrength plastic bending moment of the pile plus dead load negative bending moment and bending moment due to the plastic shear.

Provide U-bars at the bottom of the embedded pile according to *Table 1.10.5.4.2-1*. Place U-bars as close to the bottom fiber of the pile cap as possible and within the bottom half of the embedment depth. Provide #5 spirals with a 4-inch maximum pitch near the top of the embedded pile within the location shown in *Figure 1.10.5.4.2-1*.

In addition to steel reinforcing design for shear, satisfy the joint design requirements according to *AASHTO Guide Specifications 8.13*.

Figure 1.10.5.4.2-1 shows minimum steel reinforcing in pile caps. Stirrups with a 4-inch maximum spacing are required to ensure confinement for concrete around the embedded pile head. In cases where a wider cap width is specified, ensure that stirrups provided in the confinement zone meet the following equation:

$$A_{sh} = 0.12 \times s \times b_{cap} \times \frac{f'_c}{f_y}$$

where,

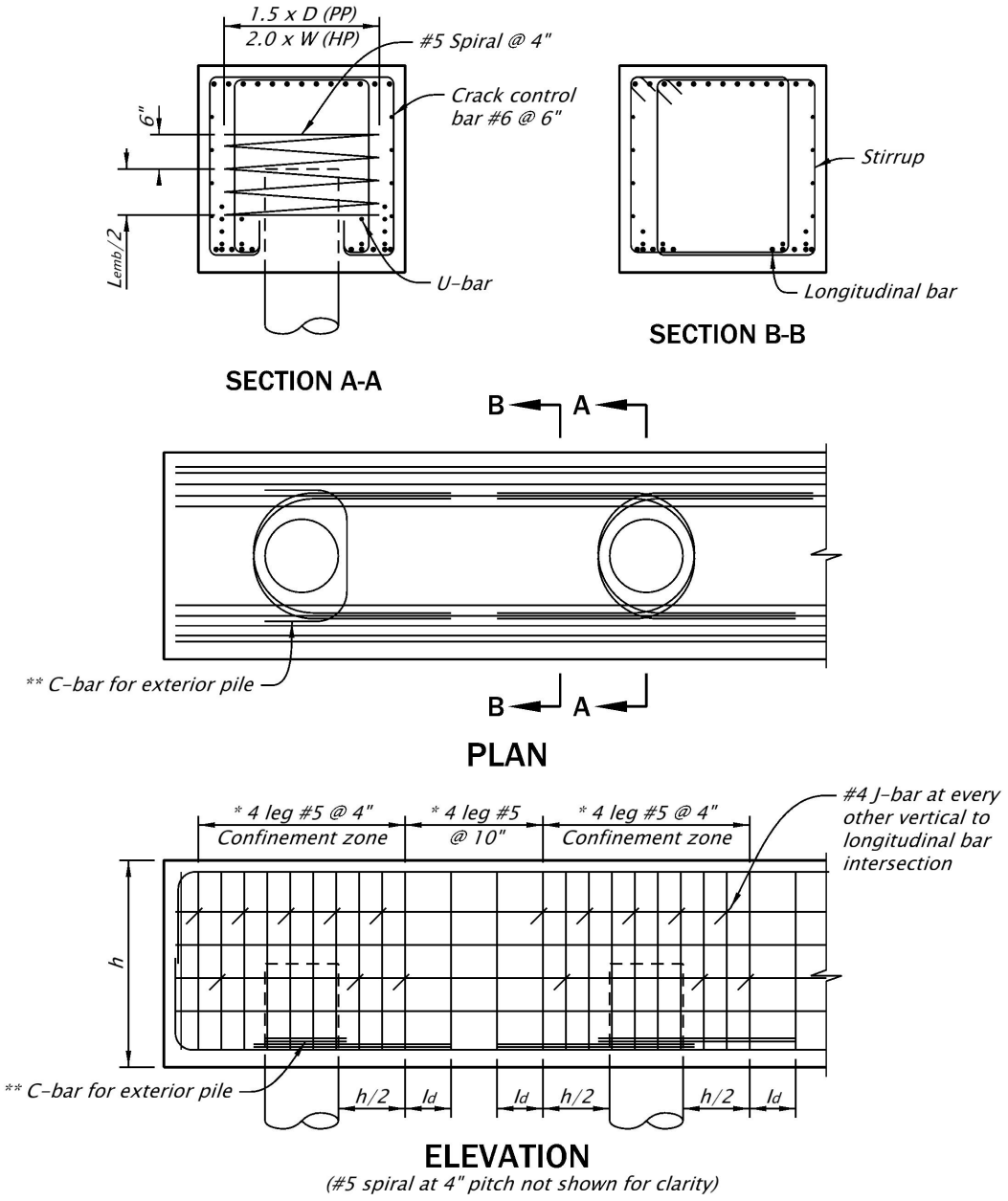
A_{sh} = total area of stirrup legs (in^2)

s = stirrup spacing (in.) (no greater than 4 in.)

b_{cap} = width of the pile cap (in.)

f_y = yield strength of rebar (ksi)

Figure 1.10.5.4.2-1 Minimum Steel Reinforcing Details for Pile Caps with Pile Moment Demand ≥ 0.60 M_p or Located in SDCs C or D



** Use #6 bars for PP 24x1/2 and HP 18x204.*

*** Provide C-bars at each exterior pile having the same bar size and number of U-bars. Provide hook length = 90 degree bend + pile diameter + the cantilever length of pile cap or 90 degree bend + pile diameter + h/2 + ld, whichever is less.*

Fill the embedded portion of pipe pile with concrete plus 5 feet below the pile cap soffit. Do not specify coating on the surface of embedded pile. When protective coating is used, stop the pile coating 2 inches maximum from the pile cap soffit. When tension capacity of the embedded pile is required, provide pile anchoring details according to *BDM 1.10.5.4.2(4).*

When space is limited for U-bars, such as in footing pile caps, provide #5 spiral at 4-inch pitch around the embedded pile portion from the top of the bottom longitudinal reinforcing mat to 6 inches above the embedded pile.

According to the *MDT report*, the damage threshold of concrete in the cap specimens during the tests was 0.75 M_P. For conservative design, 0.60 M_P was chosen as the loading limit before the rigorous steel reinforcing details are required. This is to ensure that plastic hinging will occur in steel pile section with limited concrete cracking in the pile cap. Use of the U-bars around the embedded pile followed the recommendations described in the *MDT report*. The top reinforcing spiral was required to avoid premature internal concrete cracking in the cap and increase the confinement. Transverse steel reinforcing in the embedded pile zone were also detailed according to the *MDT report*.

Several methods for casting concrete inside pipe piles include using formwork installed at the specified elevation or filling the pipe pile with gravel or controlled low strength materials (CLSM) up to the specified elevation before pouring the structural concrete.

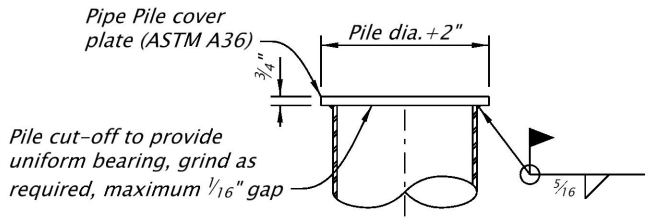
(2) PIPE PILE COVER PLATES

For unfilled pipe piles provide a welded cover plate as detailed in *Figure 1.10.5.4.2-2*.

C (2)

A cover plate is not needed when piles are concrete filled.

Figure 1.10.5.4.2-2 Pipe Pile Cover Plate



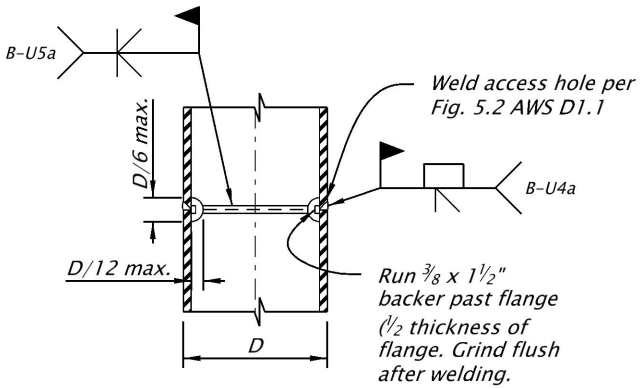
PIPE PILE (CLOSED END)

(3) STEEL PILE SPLICES

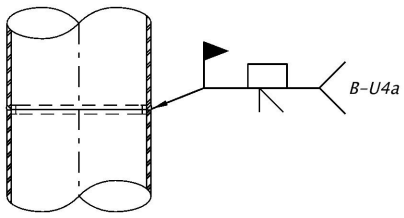
When splicing of steel piles is anticipated, show one or both of the details in *Figure 1.10.5.4.2-3* on the plans.

C (3)

Figure 1.10.5.4.2-3 Steel Pile Splices



H-PILE SPLICE*



PIPE PILE SPLICE

* – Manufactured A709 grade 36 H-pile splices may be used when located a minimum of 40 feet below the bottom of the footing and installed according to the manufacturer's recommendations.

(4) ANCHOR PILES

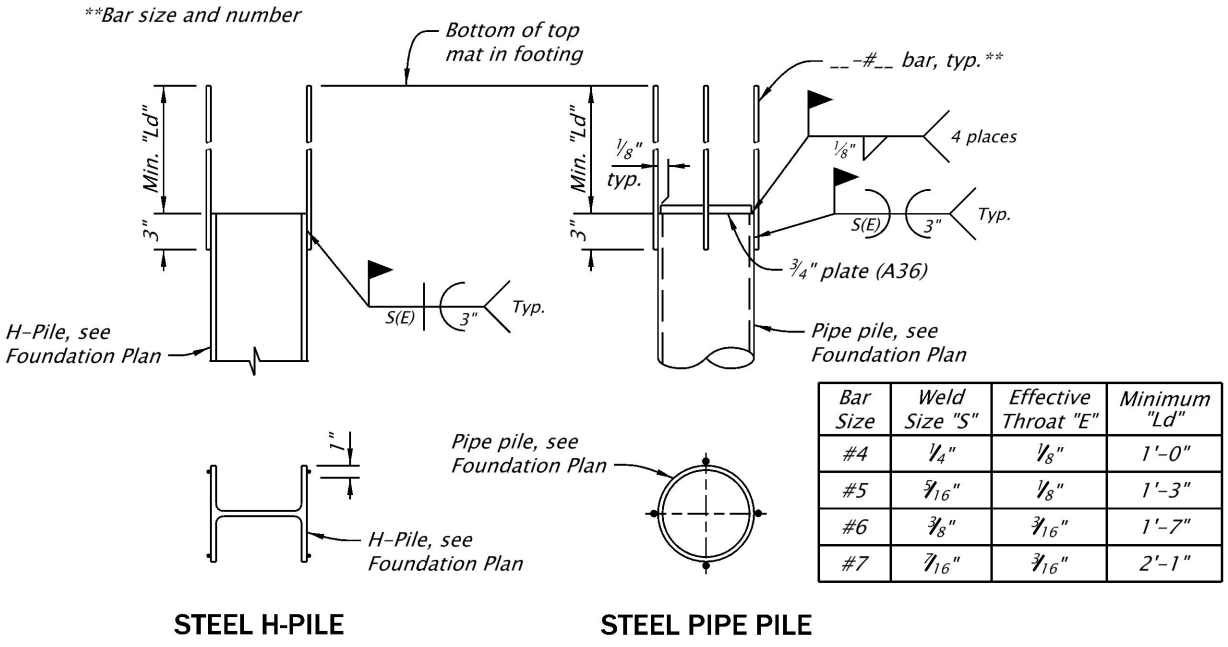
C (4)

Two methods of anchoring piles are shown in *Figure 1.10.5.4.2-4*.

When pile anchoring details are used, specify embedment depth according to *Table 1.10.5.4.2-1* and assume fixed boundary conditions in the structural analysis.

Other methods such as extending the top plate and using welded studs or other shear connectors may be appropriate. Submit proposed details to State Bridge Engineer for approval before use.

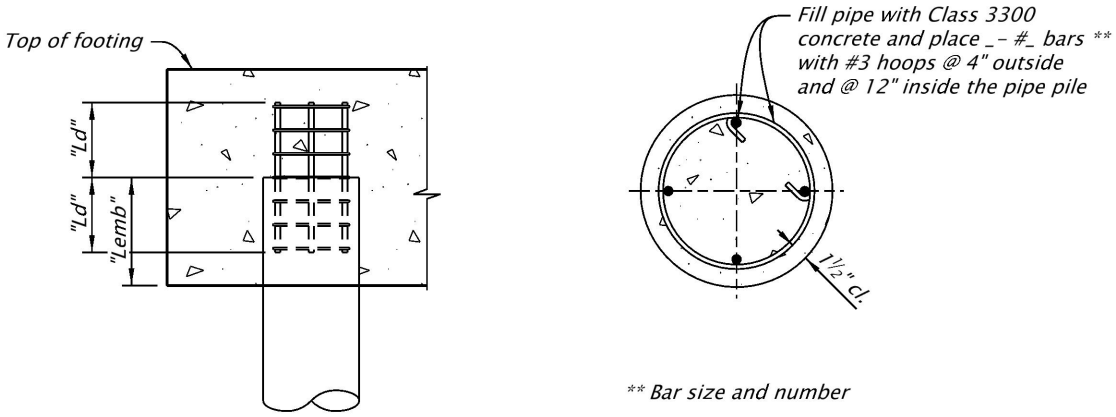
Figure 1.10.5.4.2-4 Anchor Pile Details (A) Steel H-Pile (B) Steel Pipe Pile (C) Filled Pipe Pile



Bar Size	Weld Size "S"	Effective Throat "E"	Minimum "Ld"
#4	1/4"	1/8"	1'-0"
#5	5/16"	1/8"	1'-3"
#6	3/8"	3/16"	1'-7"
#7	7/16"	3/16"	2'-1"

* Provide ASTM A706, except ASTM A615 Grade 60 or ASTM A496 may be used if copies of the chemical composition analysis are submitted and approved as weldable by the Engineer.

ANCHOR PILE DETAILS



FILLED PIPE PILE ANCHOR DETAILS

1.10.5.5 Drilled Shafts

Use drilled shafts for bridge foundations only when:

- The geotechnical designer has recommended drilled shafts for the preferred foundation type AND
- The design is economical (relative to other deep foundation designs).

Consult with the geotechnical designer regarding site constraints, environmental issues, constructability and lateral loads before selecting drilled shafts for foundation design. Locate drilled shafts early in the design process so an exploration drill hole can be located as close as possible to all drilled shaft locations for design and construction purposes.

C1.10.5.5

A Drilled Shaft Task Force Group exists to aid geotechnical and bridge designers in resolving constructability issues, revising specifications, and successfully implementing new technology. The Drilled Shaft Task Force Group is led by the Senior Bridge Geotechnical Engineer and comprised of ODOT personnel and representatives from the drilled shaft industry. Consider engaging this group early in the design process.

1.10.5.5.1 Design Considerations

(1) DRILLED SHAFT DIAMETERS, COVER REQUIREMENTS, AND HORIZONTAL TOLERANCES

The geotechnical and bridge designers should confer early in the design process to decide the most appropriate shaft diameter(s) to use for the bridge given the axial and lateral loads, column diameter, subsurface conditions, and other relevant factors.

See *GDM 17.6* for additional design considerations.

Do not design columns the same diameter as the shaft. Consider allowable horizontal tolerances, 3 inches for 6 feet shaft or smaller; and 6 inches for shaft larger than 6 feet, in shaft sizing and design. Provide additional shaft capacity to resist possible load demands in shaft that is not constructed at as-specified location.

Size drilled shaft diameters, concrete cover in drilled shafts, and column diameter using the *Table 1.10.5.5.1-1*.

Table 1.10.5.5.1-1 Drilled Shaft Dimensions

Shaft Diameter (feet)	Max. Column Diameter (feet)	Concrete Cover (inches)	Horizontal Tolerance (inches)
3	2	3	3
3.5	2.5	4	
4	3		
4.5	3.5		
5	4		
5.5	4.5		
6	4	6	6
7	5		
8	6		
9	7		
10	8		
11	9		
12	10		

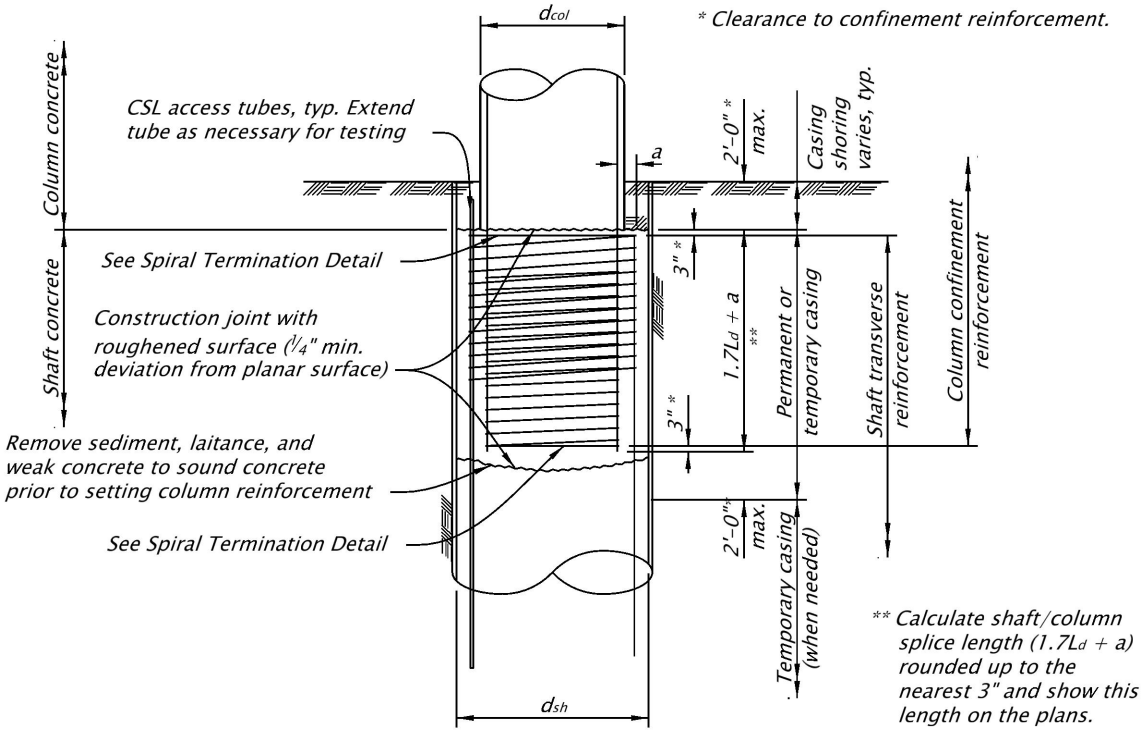
(2) NON-CONTACT SHAFT/COLUMN SPLICE

Detail shaft/column splice regions in accordance with *Figures 1.10.5.5.1-1* or *1.10.5.5.1-2*. The splice region is $(1.7Ld + a)$ rounded up to the nearest 3 inches. Note that Ld is the modified development length per *LRFD 5.10.8.2.1*. The non-contact splice detail allows the column to be adjusted horizontally when the shaft is slightly out of position (but still within the horizontal tolerance for the shaft). The shaft vs. column size limits are selected to ensure this adjustment can be made without increasing the tolerance more than the standard 1 inch for the column.

Non-contact splices require a lot of equipment, often in space-limited areas. Use of permanent casing should be considered for instances when the geotechnical designer recommends or identifies caving conditions, restricted space, and worker safety. Permanent casing changes the resistance of the drilled shaft. Use of permanent casing must be communicated with the geotechnical designer so new depth vs resistance graphs can be developed and provided to the bridge designer. Permanent casing may result in longer drilled shafts.

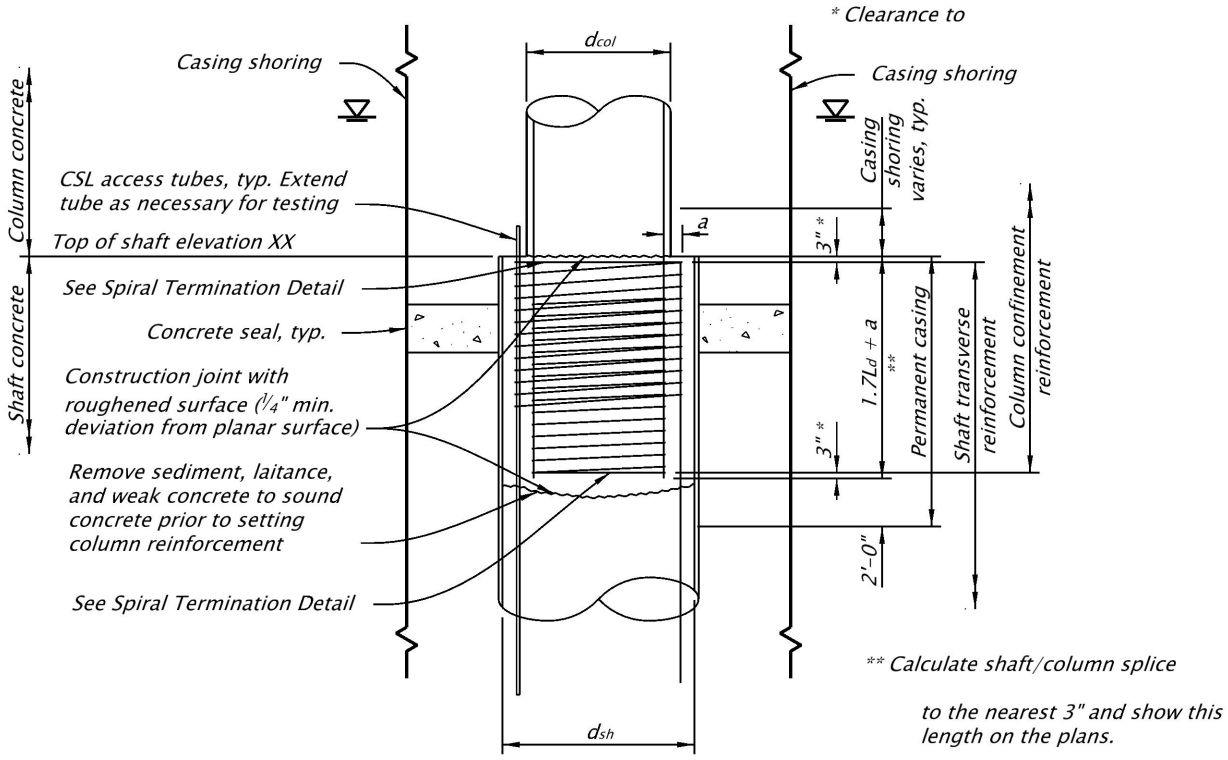
Often, during construction temporary casing cannot be extracted. Bridge designers should review the consequences of temporary casing becoming permanent casing. Communicate with the geotechnical designer these consequences so that an appropriate Drilled Shaft Installation Plan is approved during construction. Casing configurations shown on *Figures 1.10.5.5.1-1* and *1.10.5.5.1-2* are shown as examples only.

Figure 1.10.5.5.1-1 In-Ground Non-Contact Drilled Shaft Splice



$a = \frac{1}{2}$ (shaft spiral dia. – column spiral dia.)
d_{col} = column diameter
d_{sh} = shaft diameter
L_d = Modified tension development length per LRFD 5.11.2.1.1

Figure 1.10.5.5.1-2 In-Water Non-Contact Drilled Shaft Splice



a = ½ (shaft spiral dia. – column spiral dia.)
d_{col} = column diameter
d_{sh} = shaft diameter
L_d = Modified tension development length per LRFD 5.11.2.1.1

(3) DOWNDRAG LOADS

Downdrag loads, due to soil settlement other than that caused by dynamic (seismic) loading, are added to the factored vertical loads on the foundation in the Strength Limit state. Load Factors for downdrag loads will be provided by the geotechnical designer. Downdrag loads resulting from liquefaction or dynamic (seismic) induced soil settlement should be considered as a permanent load and included the Extreme Event Limit State shaft design.

(4) SHAFT UPLIFT RESISTANCE

Shaft uplift resistance is usually the same as the side friction resistance. Friction resistance in downdrag zones should be considered available for uplift resistance.

(5) SHAFT ROCK SOCKETS

Minimum shaft embedment depths into hard rock, or rock sockets, may be required due to one or more of the following design requirements or conditions:

- Lateral Load, due to earthquake loading.
- Scour.
- Liquefaction.
- Uplift loads.
- Settlement or downdrag.
- Required soil/rock bearing strata.

For rock sockets constructed inside shafts that will require either temporary or permanent casing, consider designing the diameter of the rock socket smaller than the diameter of the cased shaft above the rock socket. This is necessary to accommodate rock auger tools which are smaller in diameter than the nominal outside diameter of the cased shaft. Reduce the shaft diameters of rock sockets by at least 6 inches in these cases.

The required rock socket embedment depths should be shown on the plans. Under this condition, shaft tip elevations should be shown as **Estimated Tip Elevations** since they are likely to change depending on the actual elevation of the top of rock or hard bearing strata encountered during construction. The geotechnical designer will provide an additional shaft length, see *GDM 20.6.3.5*. In these cases, add the additional reinforcement required for this additional shaft length into the estimated quantities provided in *SP 00512*. Also adjust the concrete quantities to include this additional length. Extra reinforcement length can quickly and easily be cut off to provide the proper cage length once the final tip elevation is determined.

(6) SHAFT SETTLEMENT

Refer to *AASHTO* methods to calculate the settlement of individual shafts or shaft groups. Compare this settlement to the maximum allowable settlement and modify the shaft design if necessary to reduce the estimated settlement to acceptable levels. End bearing shafts on soil will typically settle more than friction shafts in order to mobilize end bearing resistance.

(7) SHAFT GROUP EFFECTS

For group lateral load analysis use the p-y multiplier methods described in *AASHTO* and the FHWA Manual *Drilled Shafts: Construction Procedures and Design Methods*.

(8) SHAFT SPACING

Use a minimum spacing of 3 feet for drilled shafts.

(9) SHAFT FOUNDATION DESIGN RECOMMENDATIONS

The geotechnical designer will provide final foundation recommendations in the Geotechnical Report, or earlier in the design process as needed. The following recommendations will be provided. See *GDM 20.6.3* for additional information.

- Shaft Resistance.
- Shaft Settlement.
- Shaft Uplift Resistance.
- P-Y Curves.
- Special Provisions.

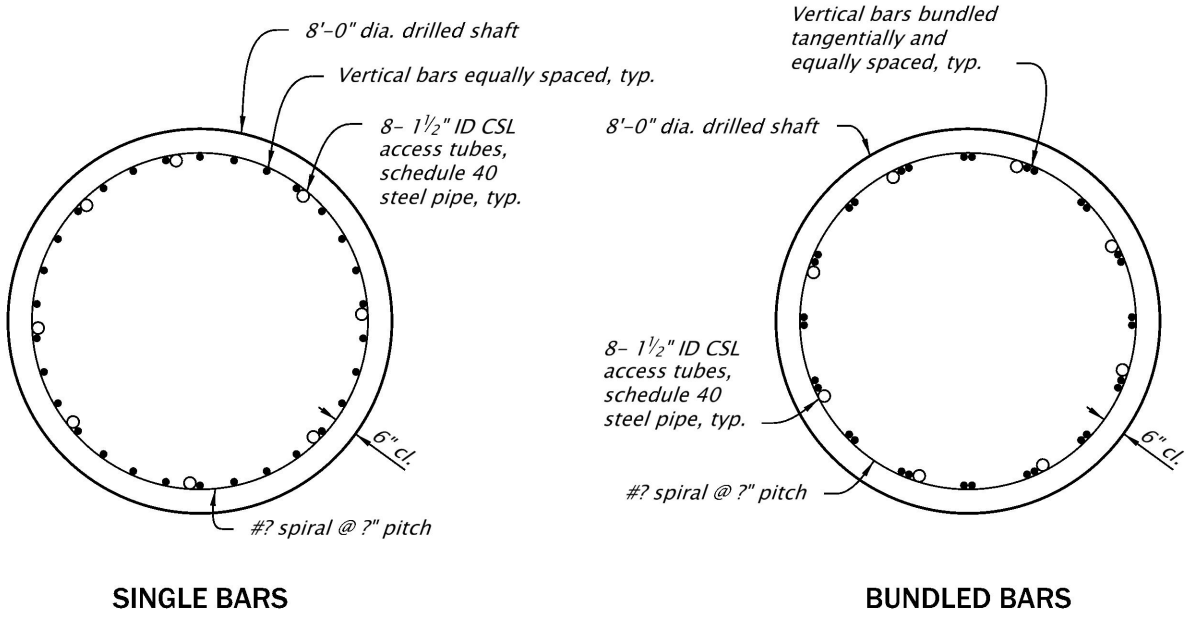
(10) POST INSTALLATION VERIFICATION TESTING

Crosshole Sonic Log (CSL) Testing

In general CSL tubes are installed in all drilled shafts unless otherwise recommended in the Geotechnical Report. Consult with the geotechnical designer regarding the CSL testing that should be performed on the project.

The rule of thumb is one CSL tube per foot diameter of shaft, with no less than four tubes and rounding up. They are equally spaced around the shaft as shown in *Figure 1.10.5.5.1-3*. See *GDM 20.6.3.4* for additional information.

Figure 1.10.5.5.1-3 CSL Tubes in Drilled Shaft



Thermal Integrity Profile Testing

Thermal Integrity Profile (TIP) testing is a post installation integrity verification test and should be considered in addition to CSL testing in some cases. TIP should be used for drilled shafts which are non redundant, or large, or constructed underwater. Unlike CSL, TIP is able to provide rebar cage cover in addition to integrity of the drilled shaft core. TIP thermal sensors

are spaced at 1 foot intervals. Quality control specifications for the use of TIP are being developed, contact the Senior Bridge Geotechnical Engineer for additional guidance.

(11) SHAFT REINFORCEMENT

Normally, a larger diameter is specified for shaft compared to the column due to constructability reasons and the capacity design requirement.

Detail shaft reinforcing to minimize congestion and facilitate concrete placement. Space both longitudinal and transverse reinforcement to provide 5 inches minimum and 9 inches maximum clear spacing between bars. Transverse shaft reinforcement may include spiral bars, hoops or bundled hoop pairs.

Ensure shaft transverse reinforcement within the non-contact splice region meets the requirements in subsection (2) of this section and in *LRFD 5.10.8.4.2a*.

For drilled shaft designs controlled by seismic loading, provide reinforcement detailing according to *AASHTO Guide Specifications Section 8.6 and 8.8.12* as amended by *BDM 1.17.2.6*. The maximum shaft moment depends on the soil-structure interaction, and it is generally larger below the ground surface than on the top of shaft.

(12) SHAFT CONCRETE

Use Class 4000 – 3/8 concrete in all drilled shafts. Concrete for drilled shafts should generally have a high slump and relatively small aggregate size in order to properly flow through the shaft reinforcement and provide the required fluid pressures against the sides of the bore hole. This is necessary to develop the desired friction resistance. Placement of concrete may be by free fall (in dry holes) or by tremie pipe (in dry or wet holes). At the present time, free fall placement of concrete in dry holes is allowed to unlimited depths. Refer to the report *Effects of Free Fall Concrete in Drilled Shafts (ADSC Report No. TL112*) for more information.

(13) REINFORCEMENT CONNECTIONS

Specify and detail hooked longitudinal bars at the top of the shaft (extending into footings or caps) to not conflict with temporary casing removal or create concrete placement difficulty. Design and detail reinforcement considering the requirements of temporary casing and concrete placement.

(14) REINFORCEMENT SPLICING

When splicing of shaft reinforcement is required, provide splicing details on the plans. Stagger splice locations. Where bundled longitudinal bars are used, also stagger splice locations within the bundle group. It is acceptable that the minimum clear spacing between shaft bars at a splice location is reduced by one bar diameter or the extra width of mechanical couplers.

For shafts where the length cannot be determined prior to construction (such as locations where a minimum penetration into the rock or hard strata is required and the elevation of the top of rock is uncertain), consider adding additional length of reinforcement to avoid the need to extend the reinforcement cage in the field. Where appropriate, include boilerplate *Special Provision*

00512.45(e) or similar language in the contract special provisions. Once the final tip elevation is determined, any remaining rebar length can be easily cut off and removed. Extending reinforcing cages in the field is undesirable because it usually results in a contract change order for additional material and potential delay of the shaft concrete pour.

(15) SHAFT ELEVATIONS

Show or list the **Top of Shaft** elevation on the plans for each drilled shaft. This elevation is the top of the drilled shaft concrete. Also show or list shaft tip elevations. When shaft tip elevations are anticipated to vary due to uncertainties in the top of the bearing strata then label these as **Estimated Tip Elevations** and show the required penetration depth into the bearing strata.

(16) PERMANENT CASING

The use of permanent casing may be beneficial in locations especially where the top of shafts are constructed in open water such as rivers or lakes. The use of permanent casing can simplify construction by eliminating the need for any temporary casing and forms. When permanent casing is desired it should be taken into account in the structural analysis of the bridge because it increases the stiffness and strength of the shaft and may significantly affect the overall response of a bridge subject to large lateral loads. It also affects the geotechnical side resistance. Consult with the geotechnical designer when permanent casing is to be used.

When permanent casing is specified remember to take OSHA requirements into account when determining casing lengths. OSHA may require casing to extend at least 2 feet above the ground surface during construction. This additional length may later be cut off and removed after the shaft is constructed.

When permanent casing is required, provide casing diameters, thicknesses and lengths in the special provisions.

(17) SHAFT DIAMETER FOR SEISMIC ANALYSIS

Drilled shafts are generally constructed slightly larger than the nominal diameter shown. For example, in soil conditions where casing is required, a 6 foot diameter shaft cannot be drilled inside a 6 foot diameter casing. A larger size casing diameter is required. Discuss with the geotechnical designer whether or not casing may be required and a larger shaft size should be checked in the structure stiffness analysis (i.e., seismic analysis). An oversize of 6 inches is recommended for shafts up to 6 feet diameter and 12 inches is recommended for larger diameter shafts.

(18) DRILLED SHAFT PRECONSTRUCTION MEETING

Preconstruction meetings are held prior to beginning drilled shaft construction. This meeting should be attended by the structural designer who designed the shaft.

1.10.5.6 Seismic Foundation Recommendations

The geotechnical designer shall provide the seismic ground motion values for the Cascadia Subduction Zone Earthquake and the 1000-year return period earthquake. Liquefaction potential is addressed along with recommendations regarding estimated lateral deformations of embankments and dynamic settlement and downdrag potential. Downdrag loads resulting from liquefaction or dynamic compaction (settlement) will be provided. Liquefaction mitigation measures and recommendations are addressed if necessary (see *BDM 1.17.4* for Liquefaction Mitigation Procedure).

1.10.5.7 Overall Stability Evaluation

The geotechnical designer shall evaluate the overall stability of the approach fills leading up to the bridge and all other unstable ground conditions, such as landslides or rockslides, that may affect the structure. This analysis shall include both static and dynamic analysis of slope stability as related to the service and extreme limit state designs. This analysis is to determine potential impacts to the bridge and approach fills which may result from embankment instability, landslide movements, settlement or other potential ground movements. A thorough geotechnical investigation, focused on slope instability, should be conducted in accordance with *the ODOT Geotechnical Design Manual (GDM).* Methods for evaluating overall stability and for estimating settlements and displacements are also described in the *GDM*. The overall stability analysis should include both non liquefiable and liquefiable foundation soil conditions as appropriate. This evaluation should be completed as early as possible in the design process to allow for changes in location and modifications to the bridge design to accommodate slope instability conditions. Coordinate with the geotechnical designer to resolve any slope instability issues that will affect the final bridge design.

For the Service Limit State, the overall stability of bridge approach fills not supporting abutment spread footings should provide a minimum factor of safety of 1.3, (roughly equivalent to a resistance factor of 0.75). A factor of safety of 1.5 against overall stability should be provided for end bent spread footings supported directly on embankments or bridge retaining walls. For bridges that are located in landslide areas, or in areas that could be affected by slide movements, the slide should be stabilized to the same factors of safety as stated above for approach fills or as determined by the Region Tech Center Geotechnical Section and Senior Bridge Geotechnical Engineer.

For Extreme Limit State I (seismic loading) conditions, the overall stability and displacement of the approach fills should be evaluated. In addition, other potentially unstable ground conditions, such as landslides or rockfalls, should also be investigated and evaluated for their potential impacts on the structure due to earthquake forces. A minimum factor of safety of 1.1 should be provided for the pseudo static analysis of bridge approach fills, landslides and any other potentially unstable ground conditions that may affect the structure. This criterion applies to sites with or without liquefiable foundation soils. In addition to this requirement,

ground displacements (lateral and vertical) should be estimated and evaluated in terms of meeting the seismic design performance criteria described in *BDM 1.17.1*. This performance criterion also applies to liquefiable or non-liquefiable foundation soil conditions. The Newmark approach is recommended for estimating the lateral displacements of approach fills, adjacent slopes, landslide masses or other ground features that may affect the structure. Other methods for estimating lateral ground deformations under seismic loading are presented in the *ODOT Geotechnical Design Manual*. When estimated ground displacements result in excessive deformation or damage to the bridge such that the performance criteria cannot be met, then mitigation measures should be pursued. The limits of liquefaction mitigation described in *BDM 1.17.4* also apply to all non-liquefiable soil conditions that require mitigation measures to meet the specified performance criteria.

1.10.5.8 Scour Design Requirements

Design bridge foundations according to the scour design criteria in *LRFD 2.6.4.4* except as modified below. Coordinate and resolve differences related to the structural, hydraulic, and geotechnical aspects of foundation design prior to approval of TS&L.

Modify *LRFD Table 2.6.4.4.4-1* as stated below:

- Replace the Yearly Mean Discharge with Q_2 for Extreme Event II Case B.

For bridges that overtop below the 100 year flood event, coordinate with the hydraulics designer to determine appropriate limit case scenarios.

Coordinate debris criteria with the project team when applicable.

State the type of countermeasure being used on the bridge plans: abutment scour countermeasure or embankment protection. See *Figures 1.10.5.8-2* and *1.10.5.8-3* for an example of these countermeasures.

State the critical scour elevation for each foundation element on the foundation data sheet of the contract plans.

C1.10.5.8

See *GDM* and *Hydraulics Design Manual (HM)* for additional information on scour design information, scour risk and footing elevation.

This section addresses scour during seismic events but does not address liquefaction. Refer to *BDM 1.17* for additional information on liquefaction and additional seismic related requirements for foundation design.

The scour elevation will be provided by the hydraulic designer in the *Hydraulics Report*. See *Figure 1.10.5.8-1 and 1.10.5.8-2* for a depiction of scour definitions.

When drift or debris is a concern, coordinate with the hydraulic designer to review the bridge inspection reports and contact the District Bridge Supervisor to confirm any ongoing debris issues.

An **abutment scour countermeasure** is one in a scour zone used to retain soil that structurally supports the structure foundation.

Embankment protection is used to protect the embankment or roadway where removal of protected material does not impact the structural integrity of the structure.

The **critical scour elevation** is the point at which scour reaches a level where the bridge's foundation or abutment become unstable, posing a risk of failure.

Do not use countermeasures for scour at interior piers, unless the local scour has the potential to impact the stability of an adjacent countermeasure.

Coordinate with geotechnical designer for lateral loading on foundations affected by scour.

Coordinate with Hydraulics and Geotech to determine impacts of local scour for interior piers. See *LRFD C2.6.4.4.3* for information about rehabilitation at interior piers.

Scour may not always affect structural elements in a uniform way. This may affect pile caps, abutment walls and other structural elements.

1.10.5.8.1 Shallow Foundations

C 1.10.5.8.1

Design shallow foundations according to *LRFD 2.6.4.4.3* except as modified below:

Design and construct shallow foundations on scour resistant rock to maintain the integrity of the supporting rock, which may allow for the top of footing above scour elevation.

When designing shallow foundations on scour resistant rock, ensure the footings are constructed fully on sound rock or a grout leveling pad constructed on sound rock.

See *HM 17.4* for additional spread footing design requirements, including minimum depth.

1.10.5.8.2 Deep Foundations

C 1.10.5.8.2

Design deep foundations according to *LRFD 2.6.4.4.3* except as modified below.

Figures 1.10.5.8-5 and *1.10.5.8-6*, along with the following subsections, provide guidance on foundation design considerations such as the elevation of shaft tops or caps relative to total scour depth and the location of the stream including the presence or absence of channel migration.

Design deep foundations such that:

- The top of the shaft or pile cap is located below the estimated long term degradation plus contraction scour depth for the scour design flood or

- The top of the pile cap may be located above scour elevations. In this case, design the deep foundation to account for the longer unbraced length due to scour.

It is often desirable to locate the pile cap below the critical scour elevation. However, this may not be feasible with project constraints. Locating a pile cap above the scour depth can lead to exposed piles. The alternate locations should be used when not feasible to place cap below long term degradation and contraction scour.

See *GDM Figure 17-1* and *AASHTO C2.6.4.4.3-1* for information on unbraced pile length.

See the *Hydraulics Report* for scour elevations at bridge elements.

Figure 1.10.5.8-1 Scour Components, Stable Channel – No Channel Migration

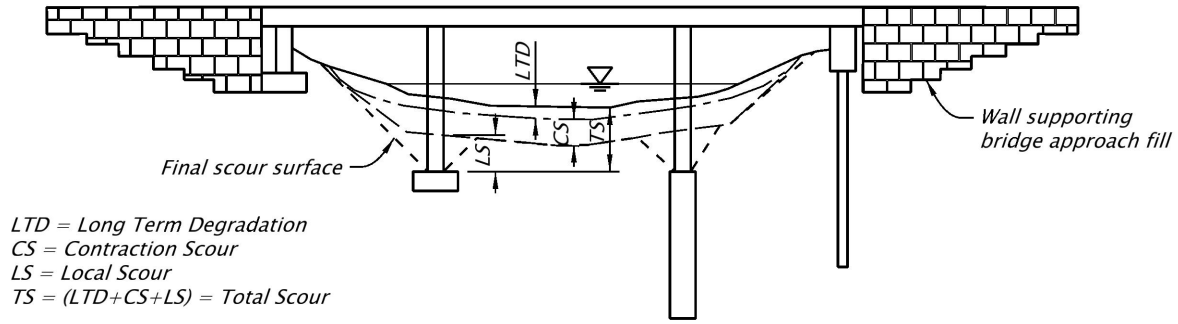


Figure 1.10.5.8-2 Scour Components, Unstable Channel – with Channel Migration

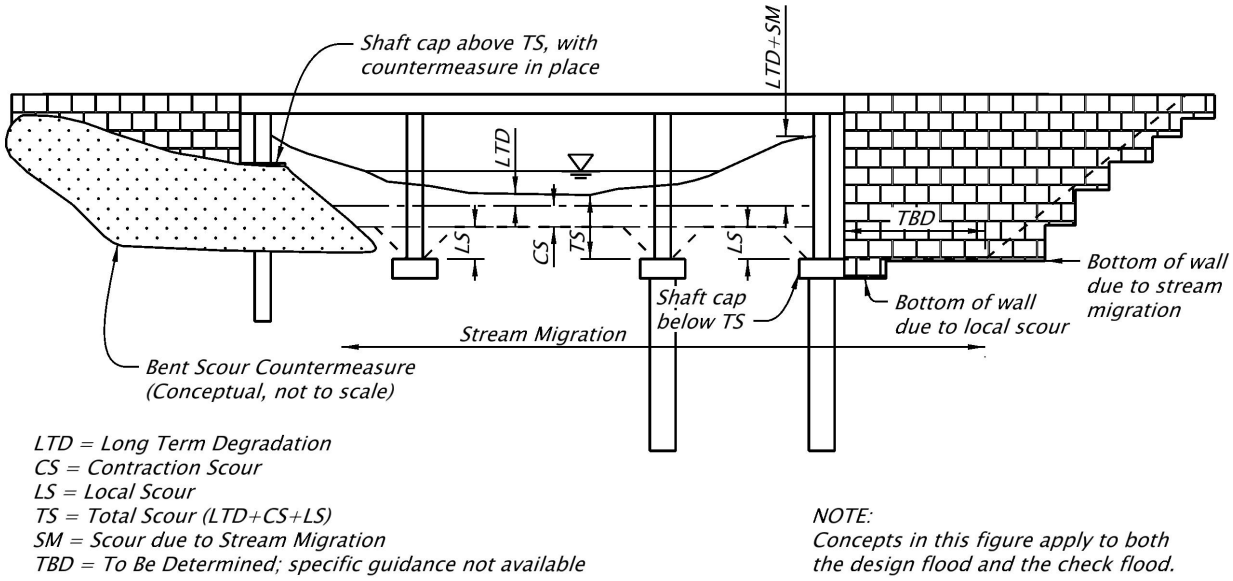


Figure 1.10.5.8-3 Scour Components with Washout

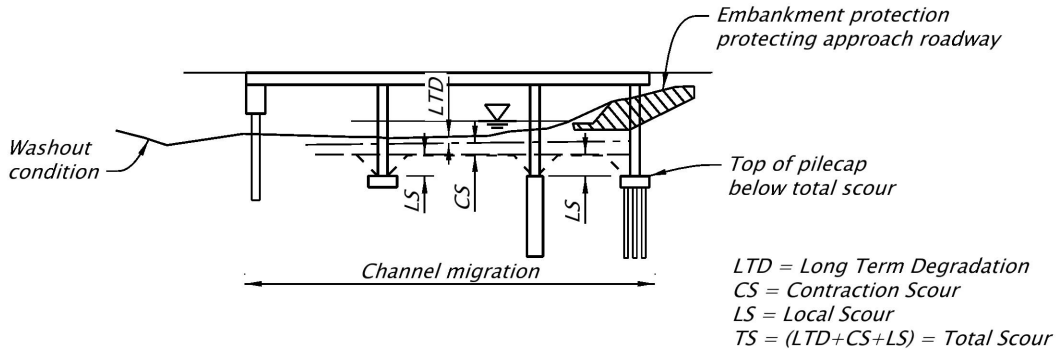


Figure 1.10.5.8-4 Pile Cap End Bent, Single Span, no Channel Migration

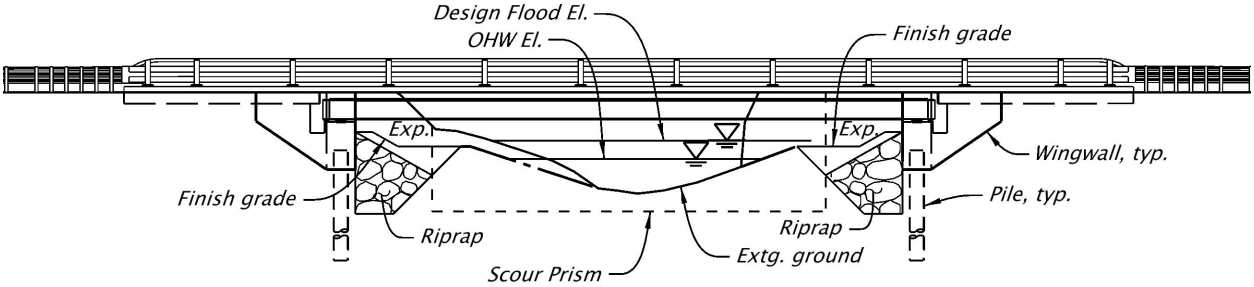
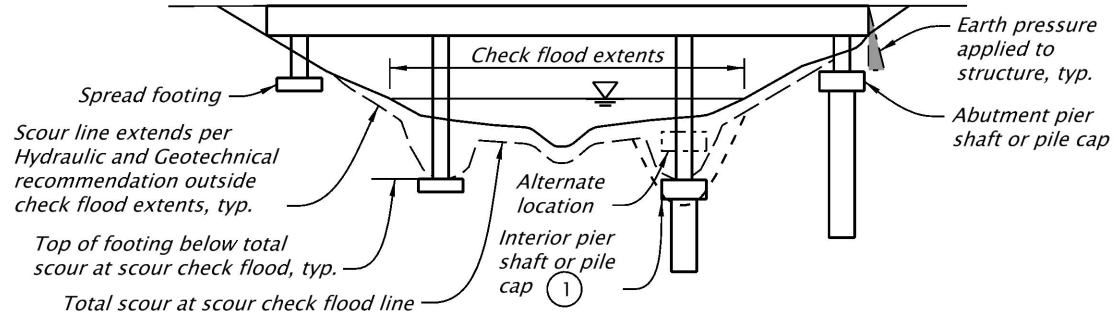
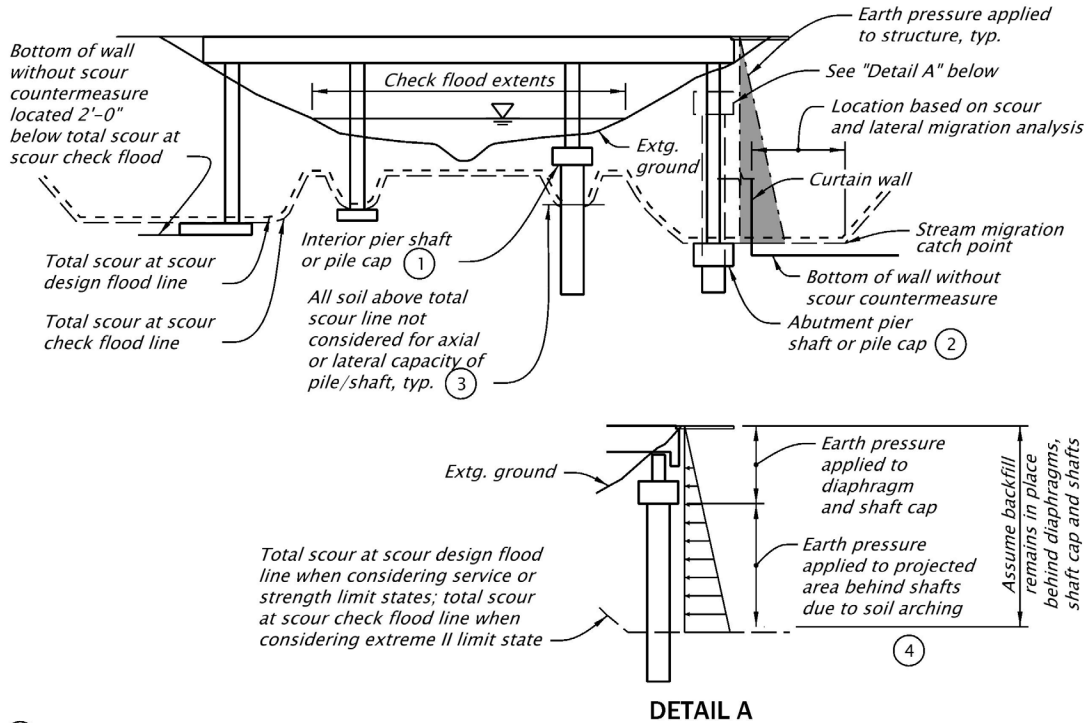


Figure 1.10.5.8-5 Recommended Footing Elevation Relative to Total Scour Depth, no Channel Migration



(1) *Interior pier shaft or pile cap may be located above the total scour at the scour check flood line as long as the foundation is designed for this condition. Account for the exposed foundation geometry in both Hydraulic and Geotechnical recommendations. The effect of the scour prism on the capacity of the shaft may follow LRFD Figure C2.6.4.4.3-1.*

Figure 1.10.5.8-6 Recommended Footing Elevation Relative to Total Scour Depth, with Channel Migration



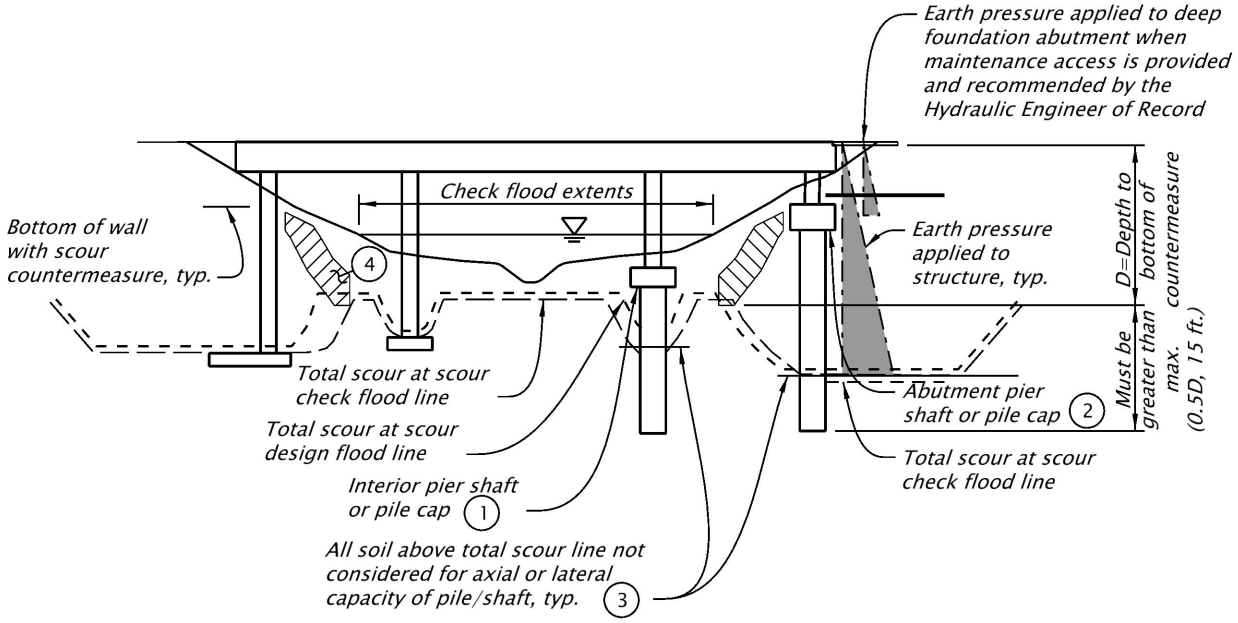
1. *Interior pier shaft or pile cap may be located above the total scour at the scour check flood line as long as the foundation is designed for this condition. Both Hydraulic and Geotechnical recommendations shall account for the exposed foundation geometry. The effect of the scour prism on the capacity of the shaft may follow LRFD Figure C2.6.4.4.3–1.*

2. *Abutment pier shaft cap or pile cap may be located above the total scour at the scour check flood line if a scour countermeasure designed and constructed following the hydraulics manual has been added at the bridge end and the foundation is designed to account for the cap being located at a higher elevation. Both hydraulic and geotechnical recommendations. Account for the exposed foundation geometry in both Hydraulic and Geotechnical recommendations. Use earth pressure on abutment per the figure when scour countermeasure is not recommended to prevent scour by the hydraulic engineer of record or no maintenance access is provided.*

3. *Only the upper half of the total scour for scour design flood is ignored for Extreme Event Case I.*

4. *Earth pressure to be developed in coordination with geotech.*

Figure 1.10.5.8-7 Recommended Footing Elevation Relative to Total Scour Depth, with Channel Migration and Scour Countermeasure at Abutment



1. *Interior pier shaft or pile cap may be located above the total scour at the scour check flood line as long as the foundation is designed for this condition. Both Hydraulic and Geotechnical recommendations shall account for the exposed foundation geometry. The effect of the scour prism on the capacity of the shaft may follow LRFD Figure C2.6.4.4.3-1.*

2. *Abutment pier shaft cap or pile cap may be located above the total scour at the scour check flood line if a scour countermeasure designed and constructed following the hydraulics manual has been added at the bridge end and the foundation is designed to account for the cap being located at a higher elevation. Both hydraulic and geotechnical recommendations shall account for the exposed foundation geometry. Earth pressure on abutment shall be per figure when scour countermeasure is not recommended to prevent scour by the hydraulic engineer of record or no maintenance access is provided.*

3. *Only the upper half of the total scour for scour design flood is ignored for Extreme Event Case I.*

4. *Place scour countermeasures outside the minimum hydraulic width. Limits of the countermeasure are based on scour and lateral migration analysis. Scour countermeasures shown for graphical purposes only. Design and construct scour countermeasures per the hydraulic manual.*

1.10.5.8.3 Channel Migration

Design bridge foundations for potential channel migration according to *LRFD 2.6.4.4.3 and 2.6.4.4.4*.

Under channel migration conditions, neglect all foundation support (vertical and lateral) provided by the embankment material beneath the abutment down to the scour elevation associated with both the Design Flood (base flood) and Check Flood events (excluding local pier scour). Design the foundation to support the bridge dead loads (DW and DC only) under both design conditions as described in the *AASHTO LRFD Bridge Design Specifications*. When the waterway is subject to channel migration, evaluate the structure for stability assuming scour associated with both the Design Flood (base flood) and Check Flood events (excluding local pier scour) has occurred throughout the "waterway".

For single span structures, investigate the bridge for the case of backfill being washed out behind one abutment. For this case, use active soil pressures with no live load surcharge on the abutment.

For channel migration at abutments supported on deep foundations, do not

C 1.10.5.8.3

The potential for channel migration is addressed in the *Bridge Hydraulic Report*. Refer to *BDM 2.14.7.3* for bridge channel migration evaluations.

Consider two key concepts during this evaluation: channel migration and stream stability. See *HEC-20* for definitions of each. The channel migration definition should include vertical and horizontal changes over time. For the purposes of this chapter sticking to channel migration as a whole should be sufficient in addressing the concerns of washout. Alternatively there is stream stability which captures a stable channel vs an unstable channel (once that has the potential for vertical and/or horizontal migration) More information available in *HEC-20*.

Coordinate with the Hydraulics designer to determine whether any scour countermeasures or increase in structure size are recommended.

For sites with potential channel migration, investigate the condition that would feasibly produce the worst case unbalanced loading in the bridge (e.g. an abutment located outside of the main channel could end up within the channel, the abutment should be designed for scour as if in the main channel). This is often the case for strutted abutments where the passive resistance of the abutment backfill material is crucial to the stability of the bridge. Backfill material behind only one abutment could lead to unbalanced loads and failure of the bridge.

include debris loads on the end bent piles or shafts in this analysis.