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7.1 General Substructure Considerations

Note that in the following guidelines where reference is made to AASHTO LRFD the item can be found in the current AASHTO *LRFD Bridge Design Specifications* (LRFD). And for any reference to AASHTO Seismic, the item can be found in the current AASHTO *Guide Specifications for LRFD Seismic Bridge Design* (SEISMIC).

7.1.1 Foundation Design Process

A flowchart is provided in which illustrates the overall design process utilized by the WSDOT Bridge and Structures Office to accomplish an LRFD foundation design. Note this process is also outlined in the [Geotechnical Design Manual](#) Section 8.2. The Bridge and Structures Office (BSO), the Geotechnical Branch (GB) and the Hydraulics Branch (HB) have been abbreviated. The steps in the flowchart are defined as follows:

A. Scoping Level Design

This phase of the design process involves the region requesting initial bridge options and costs for a future project. Depending on the complexity of the project, this phase could include a Type, Size and Location (TS&L) Report.

This design step may result in informal communication between the BSO and the GB and/or HB with the request for preliminary information and recommendations. The level of communication will depend on the available information provided by the region and the complexity of the project. The type of information that may be received from the GB and HB are as follows:

- Anticipated soil site conditions.
- Maximum embankment slopes.
- Possible foundation types and geotechnical hazards such as liquefaction.
- Scour potential for piers if a water crossing.
- Potential for future migration of a stream or river crossing.

In general, these recommendations rely on existing site data. Site borings may not be available and test holes are drilled later. The GB provides enough information to select potential foundation types for an initial scoping level or TS&L level plan and estimate.

B. Develop Site Data and Preliminary Bridge Plan

In the second phase, the BSO obtains site data from the region, see [Section 2.2](#), and develops the preliminary bridge plan. The preliminary pier locations determine soil boring locations at this time. The GB and/or the HB may require the following information to continue their preliminary design.

- Structure type and magnitude of settlement the structure can tolerate (both total and differential).
- At abutments – Approximate maximum top of foundation elevation.

- At interior piers – The initial size, shape and number of columns and how they are configured with the foundation (e.g., whether a single foundation element supports each column, or one foundation element supports multiple columns)
- At water crossings – Pier scour depth, if known, and any potential for migration of the water crossing in the future. Typically, the GB and the BSO should coordinate pursuing this information with the HB.
- Any known structural constraints that affect the foundation type, size, or location.
- Any known constraints that affect the soil resistance (utilities, construction staging, excavation, shoring, and falsework).

C. Preliminary Foundation Design

The third phase is a request by the BSO for a preliminary foundation memorandum. The GB memo will provide preliminary soil data required for structural analysis and modeling. This includes any subsurface conditions and the preliminary subsurface profile.

The concurrent geotechnical work at this stage includes:

- Completion of detailed boring logs and laboratory test data.
- Development of foundation type, soil capacity, and foundation depth.
- Development of static/seismic soil properties and ground acceleration.
- Recommendations for constructability issues.

The BO may also request the HB to provide preliminary scour design recommendations if the structure is located over a water crossing.

D. Structural Analysis and Modeling

In the fourth phase, the BSO performs a structural analysis of the superstructure and substructure using a bridge model and preliminary soil parameters. Through this modeling, the designer determines loads and sizes for the foundation based on the controlling LRFD limit states. Structural and geotechnical design continues to investigate constructability and construction staging issues during this phase.

In order to produce a final geotechnical report, the BSO provides the following structural feedback to the Geotechnical Engineer:

- Foundation loads for service, strength, and extreme limit states.
- Foundation size/diameter and depth required to meet structural design.
- Foundation details that could affect the geotechnical design of the foundations.
- Foundation layout plan.
- Assumed scour depths for each limit state (if applicable)

For water crossings, the BSO also provides the information listed above to the hydraulics engineer to verify initial scour and hydraulics recommendations are still suitable for the site.

(See Chapter 2 for examples of pile design data sheets that shall be filled out and submitted to the Geotechnical Engineer at the early stage of design.)

E. Final Foundation Design

The last phase completes the geotechnical report and allows the final structural design to begin. The preliminary geotechnical assumptions are checked and recommendations are modified, if necessary. The final report is complete to a PS&E format since the project contract will contain referenced information in the geotechnical report, such as:

- All geotechnical data obtained at the site (boring logs, subsurface profiles, and laboratory test data).
- All final foundation recommendations.
- Final constructability and staging recommendations.

The designer reviews the final report for new information and confirms the preliminary assumptions. With the foundation design complete, the final bridge structural design and detailing process continues to prepare the bridge plans. Following final structural design, the structural designer shall follow up with the geotechnical designer to ensure that the design is within the limits of the geotechnical report.

7.1.2 Foundation Design Limit States

The controlling limit states for WSDOT projects for substructure design are described as follows:

Strength I	Relating to the normal vehicular use
Strength III	Relating to the bridge exposed to wind
Strength V	Relating to the normal vehicular use and wind
Extreme-Event I	Relating to earthquake
Service I	Relating to normal operational use and wind

7.1.3 Seismic Design

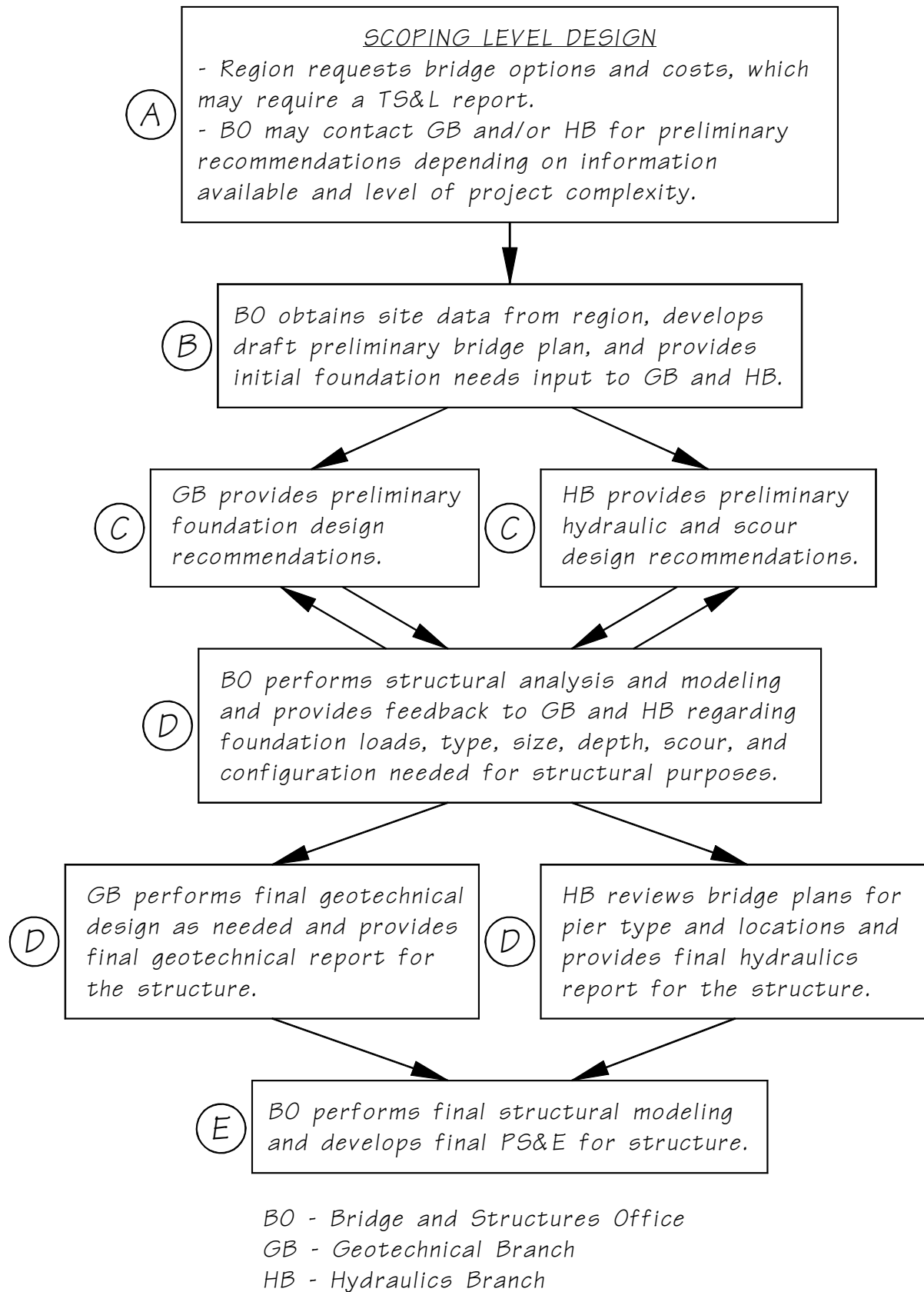
The seismic design of all substructures shall be in accordance with the AASHTO Seismic - except as noted otherwise.

7.1.4 Substructure and Foundation Loads

Figure 7.1.4-1 below provides a common basis of understanding for load location and orientations for substructure design. This figure also shows elevations required for abutment and substructure design. Note that for shaft and some pile foundation designs, the shaft or pile may form the column as well as the foundation element.

Spread footings usually have a design orientation normal to the footing. Since bridge loads are longitudinal and transverse, skewed superstructure loads are converted (using vector components) to normal and parallel footing loads. Deep foundation analysis usually has a normal/parallel orientation to the pier in order to simplify group effects.

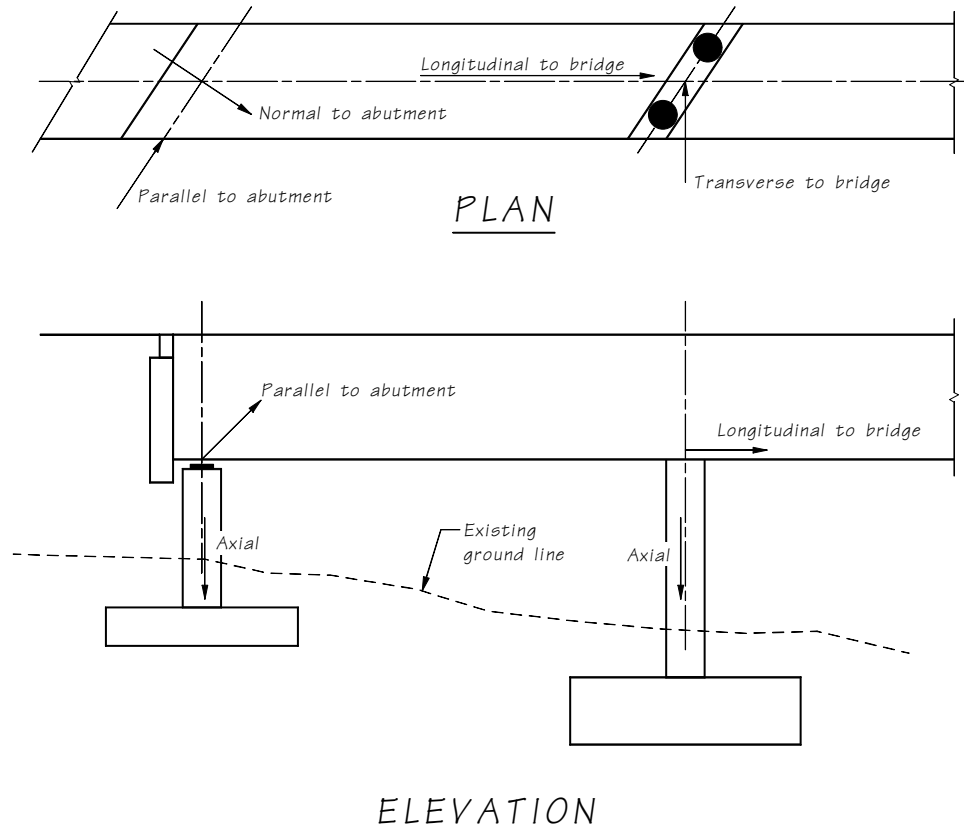
Substructure elements are to carry all of the loads specified in AASHTO Seismic and AASHTO LRFD. Selecting the controlling load conditions requires good judgment to minimize design time. All anticipated dead load (*DC*) conditions shall be accounted for during a substructure design. Sidesway effect shall be included where it tends to increase stresses. For live loads (*LL*), the dynamic allowance (*IM*) shall be applied in accordance with AASHTO LRFD Section 3.6.2 and is not included in the design of buried elements of the substructure. Portions of the abutments in contact with the soil are considered buried elements.



Overall Design Process for LRFD Foundation Design
Figure 7.1.1-1

Bridge design shall consider construction loads to ensure structural stability and prevent members from overstress. For example, temporary construction loads caused by placing all of the precast girders on one side of a crossbeam can overload a single column pier. Construction loads shall also include live loads from potential construction equipment. The plans shall show a construction sequence and/or notes to avoid unacceptable loadings.

On curved bridges, the substructure design shall consider the eccentricity resulting from the difference in girder lengths and the effects of torsion. When superstructure design uses a curved girder theory, such as the V-Load Method, the reactions from such analysis must be included in the loads applied to the substructure.



Substructure Directional Forces

Figure 7.1.4-1

7.1.5 Concrete Class for Substructure

The concrete class for all substructure elements shall be Class 4000. This includes footings, pedestals, massive piers, columns, crossbeams, traffic barriers, and retaining walls, wing walls, and curtain walls connected to the bridge substructure or superstructure. Foundation seals shall be Class 4000W. Shafts and cast-in-place piles shall be Class 5000P. **Concrete Class 4000P may be used for elements other than bridge foundations.**

7.1.6 Foundation Seals

A concrete seal within the confines of a cofferdam permits construction of a pier footing and column in the dry. This type of underwater construction is practical to a water depth of approximately 50 feet.

Seal concrete is placed underwater with the use of a tremie. A tremie is a long pipe that extends to the bottom of the excavation and permits a head to be maintained on the concrete during placement. After the concrete has been placed and has obtained sufficient strength, the water within the cofferdam is removed. In Figure 7.1.6-1, some of the factors that must be considered in designing a seal are illustrated.

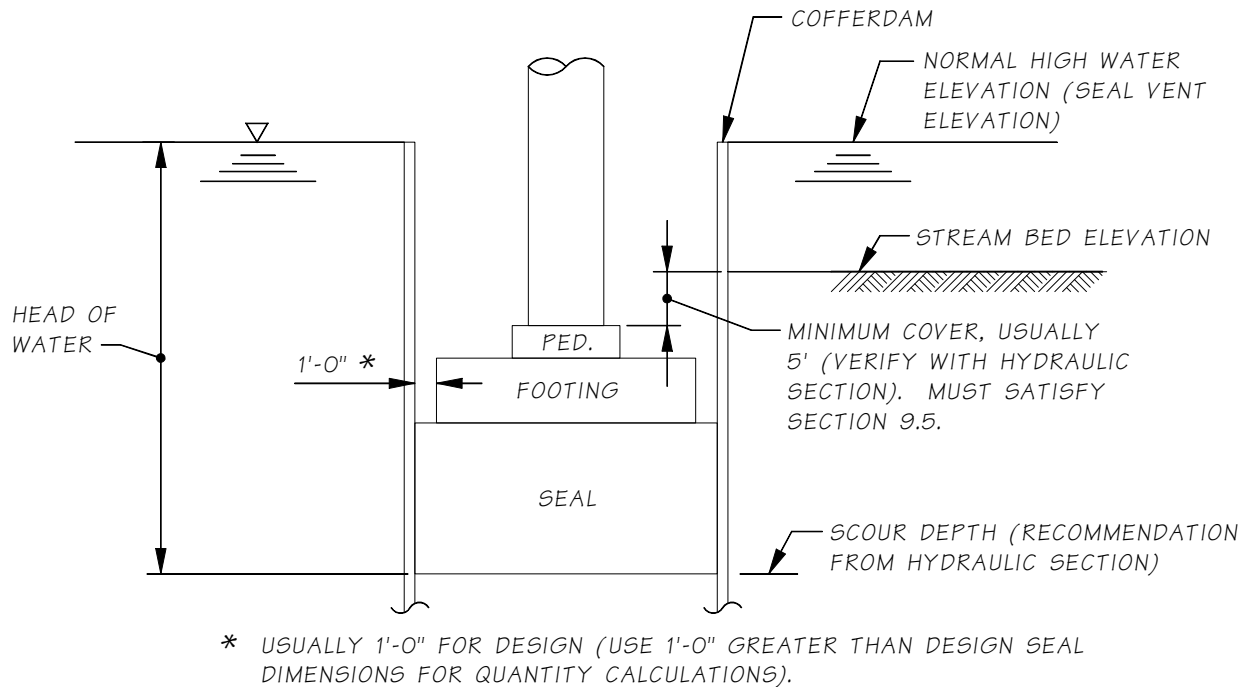


Figure 7.1.6-1

A. General Seal Criteria

The normal high water elevation is defined as the highest water surface elevation that may normally be expected to occur during a given time period. This elevation, on the hydraulics data sheet, is obtained from discussions with local residents or by observance of high water marks at the site. The normal high water is not related to any flood condition.

1. Seal Vent Elevation

The Hydraulics Branch recommends a seal vent elevation in accordance with the following criteria.

i. Construction Time Period Not Known

If the time period of the footing construction is not known, the vent elevation reflects the normal high water elevation that might occur at any time during the year.

ii. Construction Time Period Known

If the time period of the footing construction can be anticipated, the vent elevation reflects the normal high water elevation that might occur during this time period. (If the anticipated time period of construction is later changed, the Hydraulics Branch shall be notified and appropriate changes made in the design.)

2. Scour Depth

The Hydraulics Branch determines the depth of the anticipated scour. The bottom of footing, or bottom of seal if used, shall be no higher than the scour depth elevation. After preliminary footing and seal thicknesses have been determined, the bridge designer shall review the anticipated scour elevation with the Hydraulics Branch to ensure that excessive depths are not used.

3. Foundation Elevation Recommended in Geotechnical Report

Based on the results obtained from test borings at the site, the Geotechnical Engineer determines a foundation elevation, bearing capacity and settlement criteria. If other factors control, such as scour or footing cover, the final footing elevation shall be adjusted as required.

4. Unusual Conditions

Unusual site conditions such as rock formations or deep foundations require special considerations in order to obtain the most optimum design. The proposed foundation design/construction shall be discussed with both the Geotechnical Branch and the Hydraulics Branch prior to final plan preparation.

B. Spread Footing Seals

The Geotechnical Branch will generally recommend whether a foundation seal may or may not be required for construction. Bearing loads are the column moments applied at the base of the footing and vertical load applied at the bottom of the seal. The seal is sized for the soil bearing capacity. Overturning stability need only be checked at the base of the pier footing.

1. When a Seal is Required During Construction

If the footing can be raised without violating cover requirements, the bottom of the seal elevation shall be the lower of the scour elevation or the foundation elevation as recommended by the Geotechnical Engineer. The bottom of the seal may be lower than the scour elevation or foundation elevation due to cover requirements. Spread footing final design shall include the dead load weight of the seal.

2. When a Seal May Not Be Required for Construction

Both methods of construction are detailed in the plans when it is not clear if a seal is required for construction. The plans must detail a footing with a seal and an alternate without a seal. The plan quantities are based on the footing designed with a seal. If the alternate footing elevation is different from the footing with seal, it is also necessary to note on the plans the required changes in rebar such as length of column bars, increased number of ties, etc. Note that this requires the use of either General Special Provision (GSP) 6-02.3(6) B.OPT1.GB6 or 6-02.3(6) B.OPT2.GB6.

C. Pile Footing Seals

The top of footing, or pedestal, is set by the footing cover requirements. The bottom of seal elevation is based on the stream scour elevation determined by the Hydraulics Branch. A preliminary analysis is made using the estimated footing and seal weight, and the column moments and vertical load at the base of the footing to determine the number of piles and spacing. The seal size shall be 1'-0" larger than the footing all around. If the seal is omitted during construction, the bottom of footing shall be set at the scour elevation and an alternate design is made.

In general seal design requires determining a thickness such that the seal weight plus any additional resistance provided by the bond stress between the seal concrete and any piling is greater than the buoyant force (determined by the head of water above the seal). If the bond stress between the seal concrete and the piling is used to determine the seal thickness, the uplift capacity of the piles must be checked against the loads applied to them as a result of the bond stress. The bond between seal concrete and piles is typically assumed to be 10 psi. The minimum seal thickness is 1'-6".

7.2 Foundation Modeling for Seismic Loads

7.2.1 General

Bridge modeling for seismic events shall be in accordance with requirements of the AASHTO Seismic Section 5, “Analytical Models and Procedures.” The following guidance is for elastic dynamic analysis. Refer to AASHTO Seismic Section 5.4 for other dynamic analysis procedures.

7.2.2 Substructure Elastic Dynamic Analysis Procedure

The following is a general description of the iterative process used in an elastic dynamic analysis. **Note:** An elastic dynamic analysis is needed to determine the displacement demand, Δ_D . The substructure elements are first designed using Strength, Service or Extreme II limit state load cases prior to performing the dynamic analysis.

1. Build a Finite Element Model (FEM) to determine initial structure response (EQ+DC). Assume that foundation springs are located at the bottom of the column.

A good initial assumption for fixity conditions of deep foundations (shafts or piles) is to add 10' to the column length in stiff soils and 15' to the column in soft soils.

Use multi-mode response spectrum analysis to generate initial displacements.

2. Determine foundation springs using results from the seismic analysis in the longitudinal and transverse directions. **Note:** The load combinations specified in AASHTO Seismic Section 4.4 shall NOT be used in this analysis.
3. For spread footing foundations, the FEM will include foundation springs calculated based on the footing size as calculated in [Section 7.2.7](#). No iteration is required unless the footing size changes. **Note:** For Site Classes A and B the AASHTO Seismic allows spread footings to be modeled as rigid or fixed.
4. For deep foundation analysis, the FEM and the soil response program must agree or converge on soil/structure lateral response. In other words, the moment, shear, deflection, and rotation of the two programs should be within 10 percent. More iteration will provide convergence much less than 1 percent. The iteration process to converge is as follows:
 - a. Apply the initial FEM loads (moment and shear) to a p-y type soil response program such as LPILE (including LPILE, LPILE-SHAFT and LPILE-GROUP).
 - b. Calculate foundation spring values for the FEM. **Note:** The load combinations specified in AASHTO Seismic Section 4.4 shall not be used to determine foundation springs.
 - c. Re-run the seismic analysis using the foundation springs calculated from the soil response program. The structural response will change. Check to insure the FEM results (M , V , Δ , θ , and spring values) in the transverse and longitudinal direction are within 10 percent of the previous run. This check verifies the linear spring, or soil response (calculated by the FEM) is close to the predicted nonlinear soil behavior (calculated by the soil response program). If the results of the FEM and the soil response program differ by more than 10 percent, recalculate springs and repeat steps (a) thru (c) until the two programs converge to within 10 percent.

Special note for single column/single shaft configuration: The seismic design philosophy requires a plastic hinge in the substructure elements above ground (preferably in the columns). Designers should note the magnitude of shear and moment at the top of the shaft, if the column “zero” moment is close to a shaft head foundation spring, the FEM and soil response program will not converge and plastic hinging might be below grade.

Throughout the iteration process it is important to note that any set of springs developed are only applicable to the loading that was used to develop them (due to the inelastic behavior of the soil in the foundation program). This can be a problem when the forces used to develop the springs are from a seismic analysis that combines modal forces using a method such as the Complete Quadratic Combination (CQC) or other method. The forces that result from this combination are typically dominated by a single mode (in each direction as shown by mass participation). This results in the development of springs and forces that are relatively accurate for that structure. If the force combination (CQC or otherwise) is not dominated by one mode shape (in the same direction), the springs and forces that are developed during the above iteration process may not be accurate.

LPILE may be used for a pile group supported footing. Pile or shaft foundation group effects for lateral loading shall be taken as recommended in the project geotechnical report. The liquefaction option in LPILE shall not be used (the liquefaction option shall be disabled). The “Liquefied Sand” soil type shall not be used in LPILE.

7.2.3 Bridge Model Section Properties

In general, gross section properties may be assumed for all FEM members, except concrete columns.

A. Cracked Properties for Columns

Effective section properties shall be in accordance with the AASHTO Seismic Section 5.6.

B. Shaft Properties

The shaft concrete strength and construction methods lead to significant variation in shaft stiffness described as follows.

For a stiff substructure response:

1. Use f'_c to calculate the modulus of elasticity.
2. Use I_g based on the maximum oversized shaft diameter allowed by [Standard Specifications](#) Section 6-19.
3. When permanent casing is specified, increase shaft I_g using the transformed area of a $\frac{3}{4}$ " thick casing. Since the contractor will determine the thickness of the casing, $\frac{3}{4}$ " is a conservative estimate for design.

For a soft substructure response:

1. Use f'_c to calculate the modulus of elasticity.
2. Use I_g based on the nominal shaft diameter. Alternatively, I_e may be used when it is reflective of the actual load effects in the shaft.

- When permanent casing is specified, increase I using the transformed area of a $\frac{3}{8}$ " thick casing.

Since the contractor will determine the thickness of the casing, $\frac{3}{8}$ " is a minimum estimated thickness for design.

C. Cast-in-Place Pile Properties

For a stiff substructure response:

- Use $1.5 f'_c$ to calculate the modulus of elasticity. Since aged concrete will generally reach a compressive strength of at least 6 ksi when using a design strength of 4 ksi, the factor of 1.5 is a reasonable estimate for an increase in stiffness.
- Use the pile I_g plus the transformed casing moment of inertia.

$$I_{pile} = I_g + (n)(I_{shell}) + (n - 1)(I_{reinf}) \quad (7.2.3-1)$$

Where:

$$n = E_s/E_c$$

Use a steel casing thickness of $\frac{1}{4}$ " for piles less than 14" in diameter, $\frac{3}{8}$ " for piles 14" to 18" in diameter, and $\frac{1}{2}$ " for larger piles.

Note: These casing thicknesses are to be used for analysis only, the contractor is responsible for selecting the casing thickness required to drive the piles.

For a soft substructure response:

- Use $1.0 f'_c$ to calculate the modulus of elasticity.
- Use pile I_g , neglecting casing properties.

7.2.4 Bridge Model Verification

As with any FEM, the designer should review the foundation behavior to ensure the foundation springs correctly imitate the known boundary conditions and soil properties. Watch out for mismatch of units.

All finite element models must have dead load static reactions verified and boundary conditions checked for errors. The static dead loads must be compared with hand calculations or another program's results. For example, span member end moment at the supports can be released at the piers to determine simple span reactions. Then hand calculated simple span *dead load* or PGsuper *dead load* and *live load* is used to verify the model.

Crossbeam behavior must be checked to ensure the superstructure *dead load* is correctly distributing to substructure elements. A 3D bridge line model concentrates the superstructure mass and stresses to a point in the crossbeam. Generally, interior columns will have a much higher loading than the exterior columns. To improve the model, crossbeam I_g should be increased to provide the statically correct column *dead load* reactions. This may require increasing I_g by about 1000 times. Many times this is not visible graphically and should be verified by checking numerical output. Note that most finite element programs have the capability of assigning constraints to the crossbeam and superstructure to eliminate the need for increasing the I_g of the crossbeam.

Seismic analysis may also be verified by hand calculations. Hand calculated fundamental mode shape reactions will be approximate; but will ensure design forces are of the same magnitude.

Designers should note that additional mass might have to be added to the bridge FEM for seismic analysis. For example, traffic barrier mass and crossbeam mass beyond the last column at piers may contribute significant weight to a two-lane or ramp structure.

7.2.5 Deep Foundation Modeling Methods

A designer must assume a foundation support condition that best represents the foundation behavior. Deep foundation elements attempt to imitate the non-linear lateral behavior of several soil layers interacting with the deep foundation. The bridge FEM then uses the stiffness of the element to predict the seismic structural response. Models using linear elements that are not based on non-linear soil-structure interaction are generally considered inaccurate for soil response/element stress and are not acceptable. There are three methods used to model deep foundations (FHWA Report No. 1P-87-6). Of these three methods the Bridge and Structures Office prefers Method II for the majority of bridges.

A. Method I – Equivalent Cantilever Column

This method assumes a point of fixity some depth below the bottom of the column to model the stiffness of the foundation element. This shall only be used for a preliminary model of the substructure response in SDC C and D.

B. Method II – Equivalent Base Springs

This method models deep foundations by using a $\{6 \times 6\}$ matrix. There are two techniques used to generate the stiffness coefficients for the foundation matrix. The equivalent stiffness coefficients assessed are valid only at the given level of loading. Any changes of the shaft-head loads or conditions will require a new run for the program to determine the new values of the equivalent stiffness coefficients. These equivalent stiffness coefficients account for the nonlinear response of shaft materials and soil resistance.

Technique I – The matrix is generated, using superposition, to reproduce the non-linear behavior of the soil and foundation at the maximum loading. With Technique I, 10 terms are produced, 4 of these terms are “cross couples.” Soil response programs analyze the non-linear soil response. The results are then used to determine the equivalent base springs. See Appendix 7-B1 for more information.

Technique II – The equivalent stiffness matrix generated using this technique uses only the diagonal elements (no cross coupling stiffnesses). This technique is recommended to construct the foundation stiffness matrix (equivalent base springs).

In Technique II the “cross couple” effects are internally accounted for as each stiffness element and displacement is a function of the given Lateral load (P) and Moment (M). Technique II uses the total response ($\Delta_{t(P,M)} \theta_{t(P,M)}$) to determine displacement and equivalent soil stiffness, maintaining a nonlinear analysis.

Technique I requires superposition by adding the individual responses due to the lateral load and moment to determine displacement and soil stiffness. Using superposition to combine two nonlinear responses results in errors in displacement

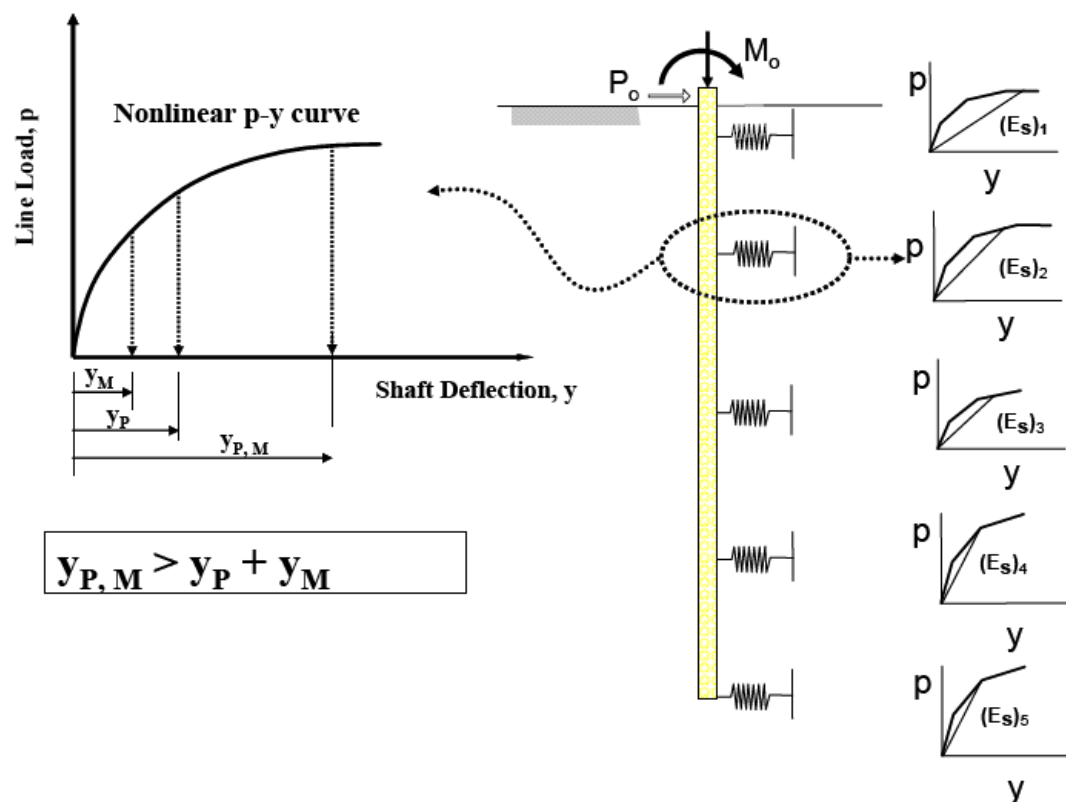
and stiffness for the total response as seen in the [Figure 7.2.5-1](#). As illustrated, the total response due to lateral load (P) and moment (M) does not necessarily equal the sum of the individual responses.

C. Method III – Non-Linear Soil Springs

This method attaches non-linear springs along the length of deep foundation members in a FEM model. See Appendix 7-B2 for more information. This method has the advantage of solving the superstructure and substructure seismic response simultaneously. The soil springs must be nonlinear PY curves and represent the soil/structure interaction. This cannot be done during response spectrum analysis with some FEM programs.

D. Spring Location (Method II)

The preferred location for a foundation spring is at the bottom of the column. This includes the column mass in the seismic analysis. For design, the column forces are provided by the FEM and the soil response program provides the foundation forces. Springs may be located at the top of the column. However, the seismic analysis will not include the mass of the columns. The advantage of this location is the soil/structure analysis includes both the column and foundation design forces.



Limitations on the Technique I (Superposition Technique)

Figure 7.2.5-1

Designers should be careful to match the geometry of the FEM and soil response program. If the location of the foundation springs (or node) in the FEM does not match the location input to the soil response program, the two programs will not converge correctly.

E. Boundary Conditions (Method II)

To calculate spring coefficients, the designer must first identify the predicted shape, or direction of loading, of the foundation member where the spring is located in the bridge model. This will determine if one or a combination of two boundary conditions apply for the transverse and longitudinal directions of a support.

A fixed head boundary condition occurs when the foundation element is in double curvature where translation without rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the opposite direction of applied moment. This is a common assumption applied to both directions of a rectangular pile group in a pile supported footing.

A free head boundary condition is when the foundation element is in single curvature where translation and rotation is the dominant behavior. Stated in other terms, the shear causes deflection in the same direction as the applied moment. Most large diameter shaft designs will have a single curvature below ground line and require a free head assumption. The classic example of single curvature is a single column on a single shaft. In the transverse direction, this will act like a flagpole in the wind, or free head. What is not so obvious is the same shaft will also have single curvature in the longitudinal direction (below the ground line), even though the column exhibits some double curvature behavior. Likewise, in the transverse direction of multi-column piers, the columns will have double curvature (frame action). The shafts will generally have single curvature below grade and the free head boundary condition applies. The boundary condition for large shafts with springs placed at the ground line will be free head in most cases.

The key to determine the correct boundary condition is to resolve the correct sign of the moment and shear at the top of the shaft (or point of interest for the spring location). Since multi-mode results are always positive (CQC), this can be worked out by observing the seismic moment and shear diagrams for the structure. If the sign convention is still unclear, apply a unit load in a separate static FEM run to establish sign convention at the point of interest.

The correct boundary condition is critical to the seismic response analysis. For any type of soil and a given foundation loading, a fixed boundary condition will generally provide soil springs four to five times stiffer than a free head boundary condition.

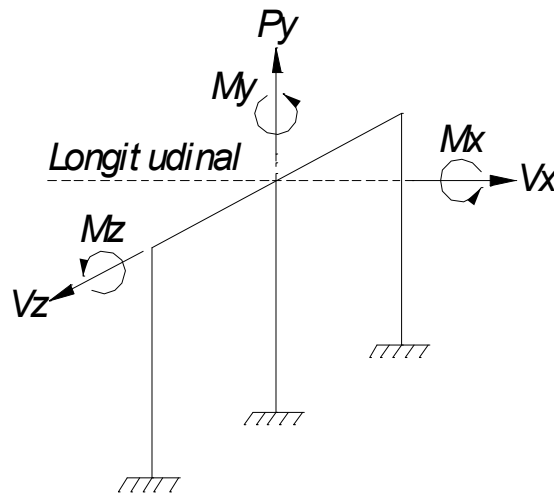
F. Spring Calculation (Method II)

The first step to calculate a foundation spring is to determine the shear and moment in the structural member where the spring is to be applied in the FEM. Foundation spring coefficients should be based on the maximum shear and moment from the applied longitudinal or transverse seismic loading. The combined load case (1.0L and 0.3T) shall be assumed for the design of structural members, and NOT applied to determine foundation response. For the simple case of a bridge with no skew, the longitudinal shear and moment are the result of the seismic longitudinal load, and the transverse components are ignored. This is somewhat unclear for highly skewed piers or curved structures with rotated springs, but the principle remains the same.

G. Matrix Coordinate Systems (Method II)

The Global coordinate systems used to demonstrate matrix theory are usually similar to the system defined for substructure loads in [Section 7.1.3](#), and is shown in [Figure 7.2.5-2](#). This is also the default Global coordinate system of GT STRUDL. This coordinate system applies to this Section to establish the sign convention for matrix terms. Note vertical axial load is labeled as P , and horizontal shear load is labeled as V .

Also note the default Global coordinate system in CSI BRIDGE uses Z as the vertical axis (gravity axis). When imputing spring values in CSI BRIDGE the coefficients in the stiffness matrix will need to be adjusted accordingly. CSI BRIDGE allows you to assign spring stiffness values to support joints. By default, only the diagonal terms of the stiffness matrix can be assigned, but when selecting the advanced option, terms to a symmetrical $\{6 \times 6\}$ matrix can be assigned.



Global Coordinate System
Figure 7.2.5-2

H. Matrix Coefficient Definitions (Method II)

The stiffness matrix containing the spring values and using the standard coordinate system is shown in [Figure 7.2.5-3](#). (Note that cross-couple terms generated using Technique I are omitted). For a description of the matrix generated using Technique I see [Appendix 7-B1](#). The coefficients in the stiffness matrix are generally referred to using several different terms. Coefficients, spring or spring value are equivalent terms. Lateral springs are springs that resist lateral forces. Vertical springs resist vertical forces.

$$\begin{Bmatrix} V_x \\ P_y \\ V_z \\ M_x \\ M_y \\ M_z \end{Bmatrix} = \begin{bmatrix} & V_x & P_y & V_z & M_x & M_y & M_z \\ V_x & K_{11} & 0 & 0 & 0 & 0 & 0 \\ P_y & 0 & K_{22} & 0 & 0 & 0 & 0 \\ V_z & 0 & 0 & K_{33} & 0 & 0 & 0 \\ M_x & 0 & 0 & 0 & K_{44} & 0 & 0 \\ M_y & 0 & 0 & 0 & 0 & K_{55} & 0 \\ M_z & 0 & 0 & 0 & 0 & 0 & K_{66} \end{bmatrix} \times \begin{Bmatrix} \text{Disp.} \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta_x \\ \theta_y \\ \theta_z \end{Bmatrix} = \begin{Bmatrix} \text{Force} \\ V_x \\ P_y \\ V_z \\ M_x \\ M_y \\ M_z \end{Bmatrix}$$

Standard Global Matrix
Figure 7.2.5-3

Where the linear spring constants or K values are defined as follows, using the Global Coordinates:

K11 = Longitudinal Lateral Stiffness (kip/in)

K22 = Vertical or Axial Stiffness (kip/in)

K33 = Transverse Lateral Stiffness (kip/in)

K44 = Transverse Bending or Moment Stiffness (kip-in/rad)

K55 = Torsional Stiffness (kip-in/rad)

K66 = Longitudinal Bending or Moment Stiffness (kip-in/rad)

The linear lateral spring constants along the diagonal represent a point on a non-linear soil/structure response curve. The springs are only accurate for the applied loading and less accurate for other loadings. This is considered acceptable for Strength and Extreme Event design. For calculation of spring constants for Technique I see [Appendix 7-B1](#).

I. Group Effects

When a foundation analysis uses LPILE or an analysis using PY relationships, group effects will require the geotechnical properties to be reduced before the spring values are calculated. The geotechnical report will provide transverse and longitudinal multipliers that are applied to the PY curves. This will reduce the pile resistance in a linear fashion. The reduction factors for lateral resistance due to the interaction of deep foundation members is provided in the [Geotechnical Design Manual](#) Section 8.12.2.5.

J. Shaft Caps and Pile Footings

Where pile supported footings or shaft caps are entirely below grade, their passive resistance should be utilized. In areas prone to scour or lateral spreading, their passive resistance should be neglected.

7.2.6 Lateral Analysis of Piles and Shafts

A. Determination of Tip Elevations

Lateral analysis of piles and shafts involves determination of a shaft or pile tip location sufficient to resist lateral loads in both orthogonal directions. In many cases, the shaft or pile tip depth required to resist lateral loads may be deeper than that required for bearing or uplift. However, a good starting point for a tip elevation is the depth required for bearing or uplift. Another good “rule-of-thumb” starting point for shaft tips is an embedment depth of 6 diameters (6D) to 8 diameters (8D). Refer also to the geotechnical report minimum tip elevations provided by the Geotechnical Engineer.

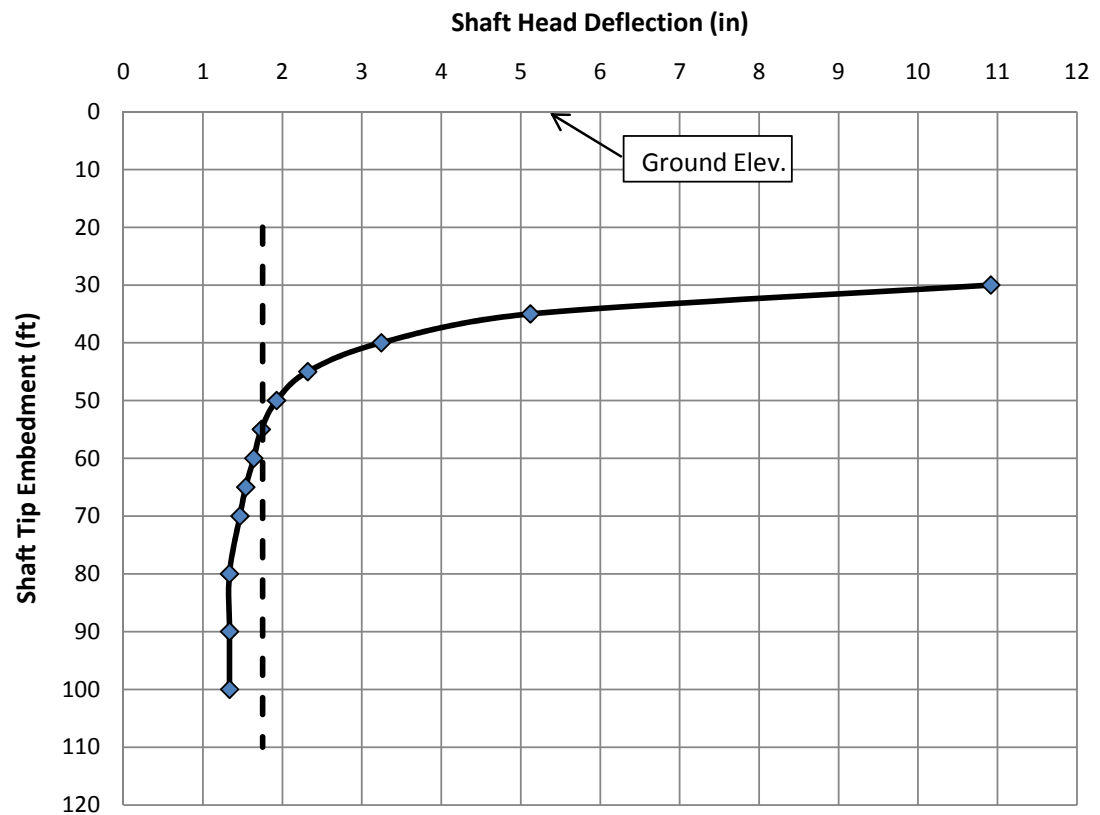
A parametric study or analysis should be performed to evaluate the sensitivity of the depth of the shaft or pile to the displacement of the structure (i.e. the displacement of the shaft or pile head) in order to determine the depth required for stable, proportionate lateral response of the structure. Determination of shaft or pile tip location requires engineering judgment, and consideration should be given to the type of soil, the confidence in the soil data (proximity of soil borings) and the potential variability in the soil profile. Arbitrarily deepening shaft or pile tips may be conservative but can also have significant impact on constructability and cost.

The following is a suggested approach for determining appropriate shaft or pile tip elevations that are located in soils. Other considerations will need to be considered when shaft or pile tips are located in rock, such as the strength of the rock. This approach is based on the displacement demand seismic design procedures specified in the *AASHTO Seismic Specifications*.

1. Size columns and determine column reinforcement requirements for Strength and Service load cases.
2. Determine the column plastic over-strength moment and shear at the base of the column using the axial dead load and expected column material properties. A program such as XTRACT or CSI BRIDGE may be used to help compute these capacities. The plastic moments and shears are good initial loads to apply to a soil response program. In some cases, Strength or other Extreme event loads may be a more appropriate load to apply in the lateral analysis. For example, in eastern Washington seismic demands are relatively low and elastic seismic or Strength demands may control.
3. Perform lateral analysis using the appropriate soil data from the Geotechnical report for the given shaft or pile location. If final soil data is not yet available, consult with the Geotechnical Engineer for preliminary values to use for the site.

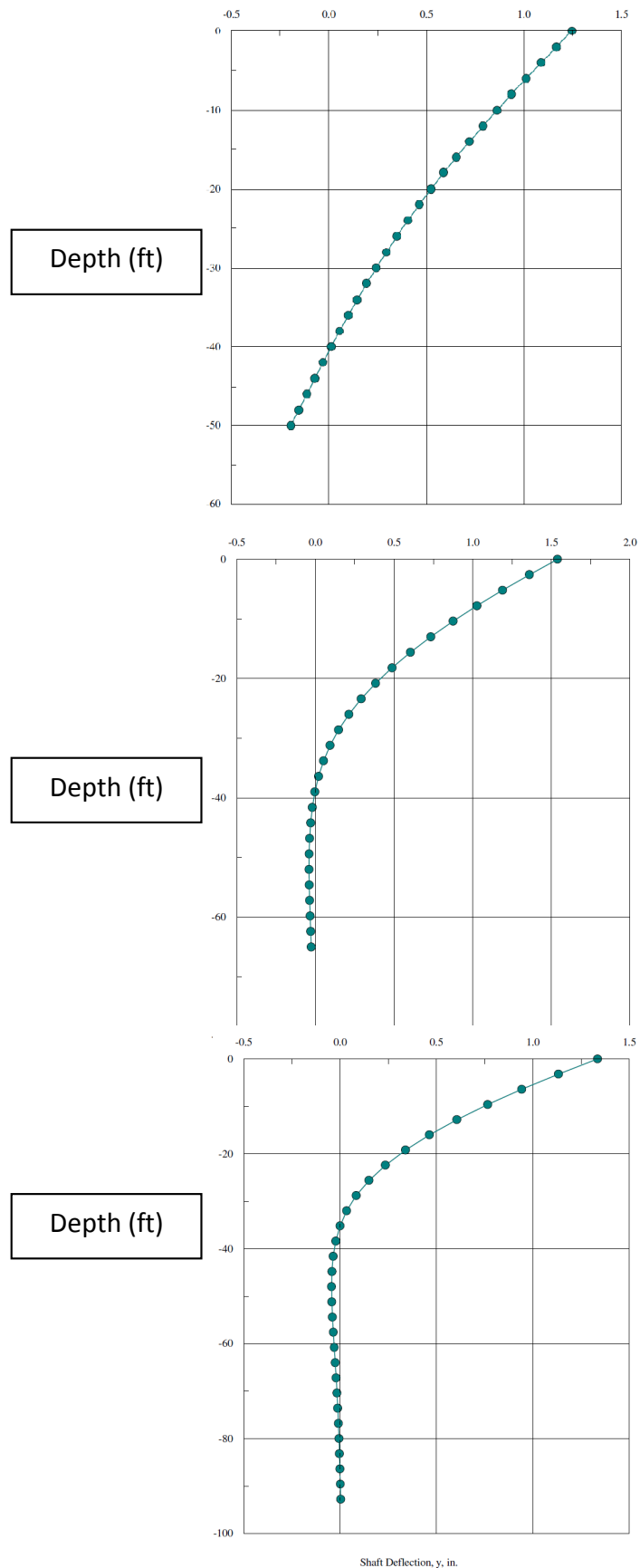
Note: Early in the lateral analysis it is wise to obtain moment and shear demands in the shaft or pile and check that reasonable reinforcing ratios can be used to resist the demands. If not, consider resizing the foundation elements and restart the lateral analysis.

4. Develop a plot of embedment depth of shaft or pile versus lateral deflection of the top of shaft or pile. The minimum depth, or starting point, shall be the depth required for bearing or uplift or as specified by the geotechnical report. An example plot of an 8' diameter shaft is shown in [Figure 7.2.6-1](#) and illustrates the sensitivity of the lateral deflections versus embedment depth. Notice that at tip depths of approximately 50' (roughly 6D) the shaft head deflections begin to increase substantially with small reductions in embedment depth. The plot also clearly illustrates that tip embedment below 70' has no impact on the shaft head lateral deflection.
5. From the plot of embedment depth versus lateral deflection, choose the appropriate tip elevation. In the example plot in [Figure 7.2.6-1](#), the engineer should consider a tip elevation to the left of the dashed vertical line drawn in the Figure. The final tip elevation would depend on the confidence in the soil data and the tolerance of the structural design displacement. For example, if the site is prone to variability in soil layers, the engineer should consider deepening the tip; say 1 to 3 diameters, to ensure that embedment into the desired soil layer is achieved. The tip elevation would also depend on the acceptable lateral displacement of the structure. To assess the potential variability in the soil layers, the Geotechnical Engineer assigned to the project should be consulted.



Shaft Tip Elevation vs Shaft Head Deflection
Figure 7.2.6-1

6. With the selected tip elevation, review the deflected shape of the shaft or pile, which can be plotted in LPILE. Examples are shown in [Figure 7.2.6-2](#). Depending on the size and stiffness of the shaft or pile and the soil properties, a variety of deflected shapes are possible, ranging from a rigid body (fence post) type shape to a long slender deflected shape with 2 or more inflection points. Review the tip deflections to ensure they are reasonable, particularly with rigid body type deflected shapes. Any of the shapes in the Figure may be acceptable, but again it will depend on the lateral deflection the structure can tolerate.



Various Shaft Deflected Shapes
Figure 7.2.6-2

The engineer will also need to consider whether liquefiable soils are present and/or if the shaft or pile is within a zone where significant scour can occur. In this case the analysis needs to be bracketed to envelope various scenarios. It is likely that a liquefiable or scour condition case may control deflection. In general, the WSDOT policy is to not include scour with Extreme Event I load combinations. In other words, full seismic demands or the plastic over-strength moment and shear, are generally not applied to the shaft or pile in a scoured condition. However, in some cases a portion of the anticipated scour will need to be included with the Extreme Event I load combination limit states. When scour is considered with the Extreme Event I limit state, the soil resistance up to a maximum of 25 percent of the scour depth for the design flood event (100 year) shall be deducted from the lateral analysis of the pile or shaft. In all cases where scour conditions are anticipated at the bridge site or specific pier locations, the Geotechnical Engineer and the Hydraulics Branch shall be consulted to help determine if scour conditions should be included with Extreme Event I limit states.

If liquefaction can occur, the bridge shall be analyzed using both the static and liquefied soil conditions. The analysis using the liquefied soils would typically yield the maximum bridge deflections and will likely control the required tip elevation, whereas the static soil conditions may control for strength design of the shaft or pile.

Lateral spreading is a special case of liquefied soils, in which lateral movement of the soil occurs adjacent to a shaft or pile located on or near a slope. Refer to the *Geotechnical Design Manual* M 46-03 for discussion on lateral spreading. Lateral loads will need to be applied to the shaft or pile to account for lateral movement of the soil. There is much debate as to the timing of the lateral movement of the soil and whether horizontal loads from lateral spread should be combined with maximum seismic inertia loads from the structure. Most coupled analyses are 2D, and do not take credit for lateral flow around shafts, which can be quite conservative. The AASHTO *Seismic Specifications* permits these loads to be uncoupled; however, the Geotechnical Engineer shall be consulted for recommendations on the magnitude and combination of loads. See *Geotechnical Design Manual* Sections 6.4.2.7 and 6.5.4 for additional guidance on combining loads when lateral spreading can occur.

B. Pile and Shaft Design for Lateral Loads

The previous section provides guidelines for establishing tip elevations for shafts and piles. Sensitivity analyses that incorporate both foundation and superstructure kinematics are often required to identify the soil conditions and loadings that will control the tip, especially if liquefied or scoured soil conditions are present. Several conditions will also need to be analyzed when designing the reinforcement for shafts and piles to ensure the controlling case is identified. All applicable strength, service and extreme load cases shall be applied to each condition. A list of these conditions includes, but is not limited to the following:

1. Static soil properties with both stiff and soft shaft or pile properties. Refer to Sections 7.2.3(B) and 7.2.3(C) for guidelines on computing stiff and soft shaft or pile properties.

2. Dynamic or degraded soil properties with both stiff and soft shaft or pile properties.
3. Liquefied soil properties with both stiff and soft shaft or pile properties.
 - a. When lateral spreading is possible, an additional loading condition will need to be analyzed. The Geotechnical Engineer shall be consulted for guidance on the magnitude of seismic load to be applied in conjunction with lateral spreading loads. See *Geotechnical Design Manual* Sections 6.4.2.7 and 6.5.4 for additional guidance on combining loads when lateral spreading can occur.
4. Scour condition with stiff and soft shaft or pile properties. The scour condition is typically not combined with Extreme Event I load combinations, however the designer shall consult with the Hydraulics Branch and Geotechnical Engineer for recommendations on load combinations. If scour is considered with the Extreme Event I limit state, the analysis should be conducted assuming that the soil in the upper 25 percent of the estimated scour depth for the design (100 year) scour event has been removed to determine the available soil resistance for the analysis of the pile or shaft.

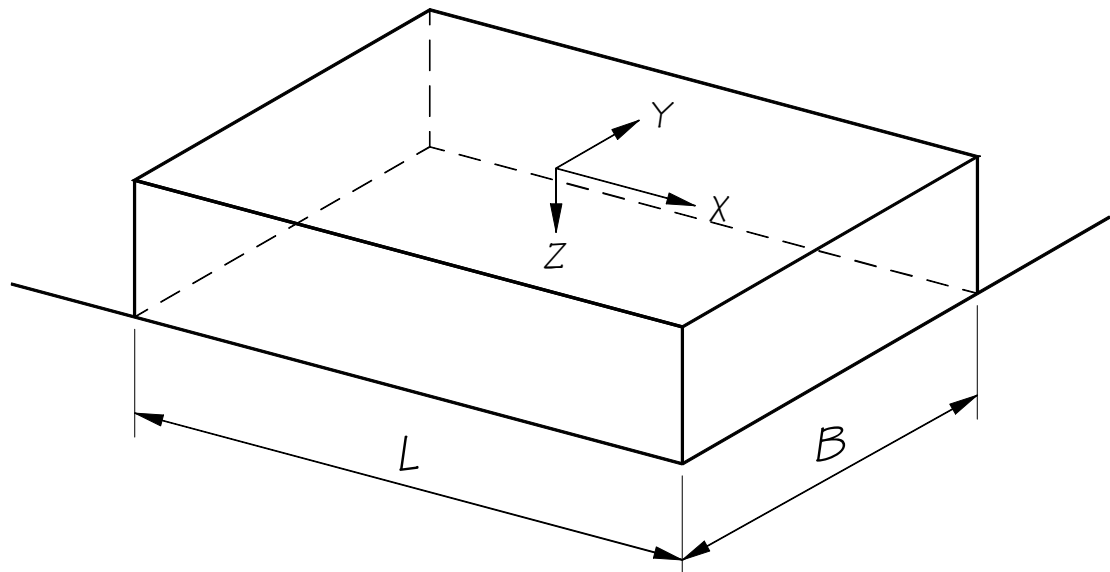
Note: Often, the highest acceleration the bridge sees is in the first cycles of the earthquake, and degradation and/or liquefaction of the soil tends to occur toward the middle or end of the earthquake. Therefore, early in the earthquake, loads are high, soil-structure stiffness is high, and deflections are low. Later in the earthquake, the soil-structure stiffness is lower and deflections higher. This phenomenon is normally addressed by bracketing the analyses as discussed above.

However, in some cases a site specific procedure may be required to develop a site specific design response spectrum. A site specific procedure may result in a reduced design response spectrum when compared to the general method specified in the AASHTO Seismic 3.4. Section 3.4 requires the use of spectral response parameters determined using [USGS/AASHTO Seismic Hazard Maps](#). The AASHTO *Seismic Specifications* limits the reduced site specific response spectrum to two-thirds of what is produced using the general method. Refer to the *Geotechnical Design Manual* [Chapter 6](#) for further discussion and consult the Geotechnical Engineer for guidance.

Refer to Section 7.8 Shafts and Chapter 4 for additional guidance/requirements on design and detailing of shafts and Section 7.9 Piles and Piling and Chapter 4 for additional guidance/requirements on design and detailing of piles.

7.2.7 Spread Footing Modeling

For a first trial footing configuration, Strength column moments or column plastic hinging moments may be applied to generate footing dimensions. Soil spring constants are developed using the footing plan area, thickness, embedment depth, Poisson's ratio ν , and shear modulus G . The Geotechnical Branch will provide the appropriate Poisson's ratio and shear modulus. Spring constants for shallow rectangular footings are obtained using the following equations developed for rectangular footings. This method for calculating footing springs is referenced in [ASCE 41-06](#), Section 4.4.2.1.2. (**Note:** [ASCE 41-06](#) was developed from [FEMA 356](#).)



Orient axes such that $L > B$.
If $L = B$ use x-axis equations for both x-axis and y-axis.

Figure 7.2.7-1

Where:

$$K = \beta K_{sur}$$

K = Translation or rotational spring

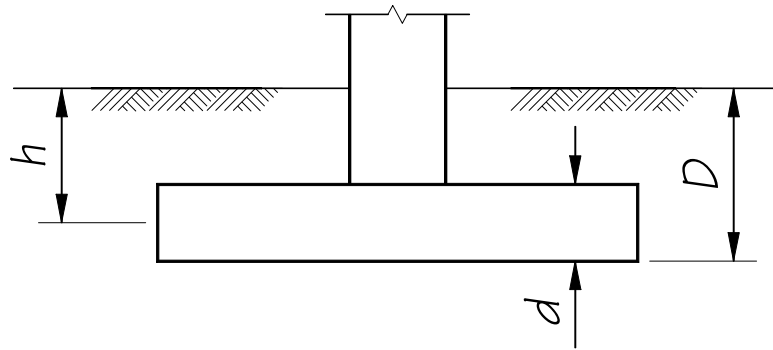
K_{sur} = Stiffness of foundation at surface, see [Table 7.2.7-1](#)

β = Correction factor for embedment, see [Table 7.2.7-2](#)

Degree of Freedom	K_{sur}
Translation along x-axis	$\frac{GB}{2-\nu} \left[3.4 \left(\frac{L}{B} \right)^{0.65} + 1.2 \right]$
Translation along y-axis	$\frac{GB}{2-\nu} \left[3.4 \left(\frac{L}{B} \right)^{0.65} + 0.4 \frac{L}{B} + 0.8 \right]$
Translation along z-axis	$\frac{GB}{1-\nu} \left[1.55 \left(\frac{L}{B} \right)^{0.75} + 0.8 \right]$
Rocking about x-axis	$\frac{GB^3}{1-\nu} \left[0.4 \left(\frac{L}{B} \right) + 0.1 \right]$
Rocking about y-axis	$\frac{GB^3}{1-\nu} \left[0.47 \left(\frac{L}{B} \right)^{2.4} + 0.034 \right]$
Torsion about z-axis	$GB^3 \left[0.53 \left(\frac{L}{B} \right)^{2.45} + 0.51 \right]$

Stiffness of Foundation at Surface

Table 7.2.7-1



Where:

d = Height of effective sidewall contact (may be less than total foundation height if the foundation is exposed).

h = Depth to centroid of effective sidewall contact.

Figure 7.2.7-2

Degree of Freedom	β
Translation along x-axis	$\left(1 + 0.21\sqrt{\frac{D}{B}}\right) \left[1 + 1.6\left(\frac{hd(B+L)}{BL^2}\right)^{0.4}\right]$
Translation along y-axis	$\left(1 + 0.21\sqrt{\frac{D}{L}}\right) \left[1 + 1.6\left(\frac{hd(B+L)}{LB^2}\right)^{0.4}\right]$
Translation along z-axis	$\left[1 + \frac{1}{21}\frac{D}{B}\left(2 + 2.6\frac{B}{L}\right)\right] \cdot \left[1 + 0.32\left(\frac{d(B+L)}{BL}\right)^{\frac{2}{3}}\right]$
Rocking about x-axis	$1 + 2.5\frac{d}{B} \left[1 + \frac{2d}{B}\left(\frac{d}{D}\right)^{-0.2} \sqrt{\frac{B}{L}}\right]$
Rocking about y-axis	$1 + 1.4\left(\frac{d}{L}\right)^{0.6} \left[1.5 + 3.7\left(\frac{d}{L}\right)^{1.9} \left(\frac{d}{D}\right)^{-0.6}\right]$
Torsion about z-axis	$1 + 2.6\left(1 + \frac{B}{L}\right)\left(\frac{d}{B}\right)^{0.9}$

Correction Factor for Embedment

Table 7.2.7-2

7.3 Column Design

7.3.1 General Design Considerations

The preliminary plan stage determines the initial column size, column spacing, and bridge span length based on a preliminary analysis. Columns are spaced to give maximum structural benefit except where aesthetic considerations dictate otherwise. Piers normally are spaced to meet the geometric and aesthetic requirements of the site and to give maximum economy for the total structure. Good preliminary engineering judgment results in maximum economy for the total structure.

The designer may make changes after the preliminary plan stage. The Design Unit Manager will need to review all changes, and if the changes are more than minor dimension adjustments, the Bridge Project Support Engineer and the State Bridge and Structures Architect will also need to be involved in the review.

Tall piers spaced farther apart aesthetically justify longer spans. Difficult and expensive foundation conditions will also justify longer spans. Span lengths may change in the design stage if substantial structural improvement and/or cost savings can be realized. The designer should discuss the possibilities of span lengths or skew with the Design Unit Manager as soon as possible. Changes in pier spacing at this stage can have significant negative impacts to the geotechnical investigation.

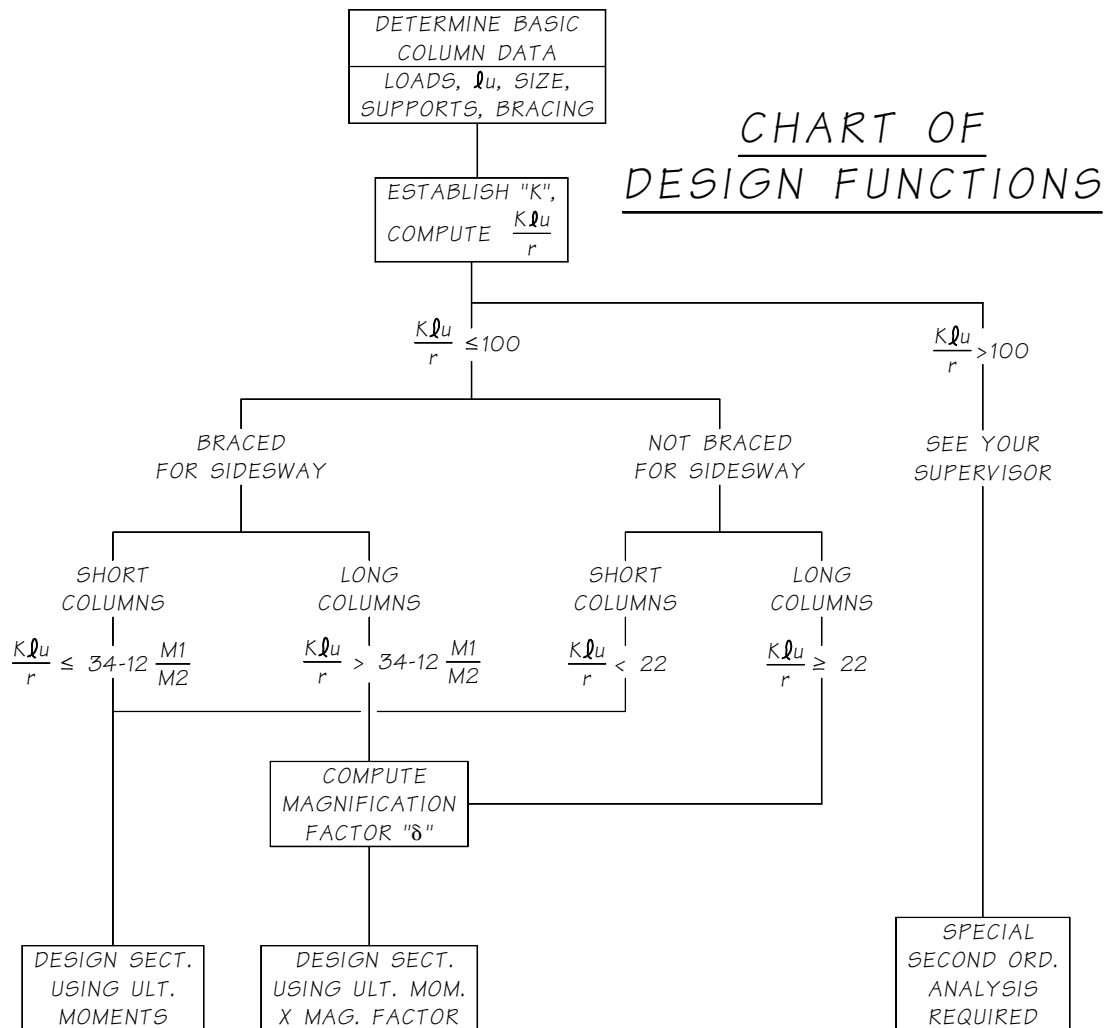
Column spacing should minimize column dead load moments. Multiple columns are better suited for handling lateral loads due to wind and/or earthquake. The designer may alter column size or spacing for structural reasons or change from a single-column pier to a multicolumn pier. Columns should be designed so that construction is as simple and repetitive as possible. The diameter of circular columns should be a multiple of one foot; however increments of 6 inches may be appropriate in some cases. Rectangular sections shall have lengths and widths that are multiples of 3 inches. Long rectangular columns are often tapered to reduce the amount of column reinforcement required for strength. Tapers should be linear for ease of construction.

For long columns, it may be advantageous to reduce the amount of reinforcement as the applied loads decrease along the column. In these cases, load combinations need to be generated at the locations where the reinforcement is reduced.

Bridge plans shall show column construction joints at the top of footing or pedestal and at the bottom of crossbeam. Optional construction joints with roughened surfaces should be provided at approximately 30-foot vertical spacing.

7.3.2 Slenderness Effects

This section supplements and clarifies AASHTO LRFD. The goal of a slenderness analysis is to estimate the additional bending moments in the columns that are developed due to axial loads acting upon a deflected structure. Two primary analysis methods exist: the moment magnifier method and the second-order analysis. The designer must decide which method to use based upon the slenderness ratio (kL_u/r) of the column(s). Figure 7.3.2-1 below illustrates the basic steps in the column design process for evaluating the effects of slenderness on columns and methods for computing magnification of moments on columns.



Column Design Flowchart for Non-Seismic Design

Figure 7.3.2-1

A. Moment Magnification Method

The moment magnification method is described in AASHTO LRFD Section 4.5.3.2.2. In general, if magnification factors computed using the AASHTO LRFD are found to exceed about 1.4, then a second-order analysis may yield substantial benefits.

In a member with loads applied at the joints, any significant lateral deflection indicated the member is unbraced. The usual practice is to consider the pier columns as unbraced in the transverse direction. The superstructure engages girder stops at the abutment and resists lateral sidesway due to axial loads. However, pier lateral deflections are significant and are considered unbraced. Short spanned bridges may be an exception. Most bridge designs provide longitudinal expansion bearings at the end piers. Intermediate columns are considered unbraced because they must resist the longitudinal loading. The only time a column is braced in the longitudinal direction is when a framed bracing member does not let the column displace more than $L/1500$. L is the total column length. In this case, the bracing member must be designed to take all of the horizontal forces.

B. Second-Order Analysis

A second-order analysis that includes the influence of loads acting on the deflected structure is required under certain circumstances, and may be advisable in others. It can lead to substantial economy in the final design of many structures. Computations of effective length factors, k , and buckling loads, P_c , are not required for a second-order analysis, though they may be helpful in establishing the need for such an analysis. The designer should discuss the situation with the Design Unit Manager before proceeding with the analysis. The ACI Building Code ([ACI 318-08](#)), should be consulted when carrying out a second-order analysis.

1. Design Methods for a Second-Order Analysis

For columns framed together, the entire frame should be analyzed as a unit. Analyzing individual columns result in overly conservative designs for some columns and non- conservative results for others. The preferred method for performing a second-order analysis of an entire frame or isolated single columns is to use a nonlinear finite element program with appropriate stiffness and restraint assumptions. The factored group loads are applied to the frame, including the self-weight of the columns. The model is then analyzed using the nonlinear option. The final design moments are obtained directly from the analysis.

$P\Delta$ moments are added to the applied moments using an iterative process until stability is reached. The deflections should converge within 5 percent of the total deflection. Analysis must include the effect of the column weight; therefore, the axial dead load must be adjusted as follows:

$$P_u = P_u + \frac{1}{3} (\text{factored column weight}) \quad (7.3.2-1)$$

2. Applying Factored Loads

For a second-order analysis, loads are applied to the structure and the analysis results in member forces and deflections. It must be recognized that a second-order analysis is non-linear and the commonly assumed principle of superposition may not be applicable. The loads applied to the structure should be the entire set of factored loads for the load group under consideration. The analysis must be repeated for each group load of interest. The problem is complicated by the fact that it is often difficult to predict in advance which load groups will govern.

For certain loadings, column moments are sensitive to the stiffness assumptions used in the analysis. For example, loads developed as a result of thermal deformations within a structure may change significantly with changes in column, beam, and foundation stiffness. Accordingly, upper and lower bounds on the stiffness should be determined and the analysis repeated using both sets to verify the governing load has been identified.

3. Member Properties

As with a conventional linear elastic frame analysis, various assumptions and simplifications must be made concerning member stiffness, connectivity, and foundation restraint. Care must be taken to use conservative values for the slenderness analysis. Reinforcement, cracking, load duration, and their variation along the members are difficult to model while foundation restraint will be modeled using soil springs.

7.3.3 Shear Design

Shear design should follow the “Simplified Procedure for Nonprestressed Sections” in AASHTO LRFD Section 5.8.3.4.1.

7.3.4 Column Silos

Column silos are an acceptable technique to satisfy the balanced stiffness and frame geometry requirements of Section 4.2.7 and the [AASHTO Seismic Specifications](#). Due to the construction and inspection complications of column silos, designers are encouraged to meet balanced stiffness and frame geometry requirements by the other methods suggested in Section 4.1.4 of the [AASHTO Seismic Specifications](#) prior to use of column silos.

A. General Design and Detailing Requirements

1. Column silo plans, specifications, and estimates shall be included in the Contract Documents.
2. Column silos shall be designed to resist lateral earth and hydrostatic pressure, including live load surcharge if applicable, for a 75-year minimum service life.
3. Column silos are not permitted for in-water locations such as in rivers and lakes.
4. Clearance between the column and the column silo shall be adequate for column lateral displacement demands, construction and post-earthquake inspection, but shall not be less than 1'-6".

5. A 6" minimum clearance shall be provided from the top of column silo to ground level.
6. Maximum depth of column silos shall not exceed 15 feet.
7. Column silos shall be watertight when located below the highest expected groundwater elevation. Silo covers need not be liquid tight.
8. Column silos shall be positively attached to the foundation element.

B. Column Silos Formed From Extending Shaft Casing

Designers shall determine a minimum steel casing thickness sufficient to resist lateral loads and shall provide it in the Contract Documents. This thickness shall include a sacrificial steel area as recommended in AASHTO *LRFD Specification* Section C10.7.5 for corrosion resistance. The actual steel casing size and materials shall be determined by the Contractor as delineated in *Standard Specifications* Section 6-19 and 9-36. Appropriate detailing, as shown in [Figure 7.3.4-1](#), shall be provided. The designer shall check that the minimum column-to-silo horizontal clearance is provided even if the permanent casing is constructed with a smaller diameter slip casing.

C. Column Silos Formed by Other Methods

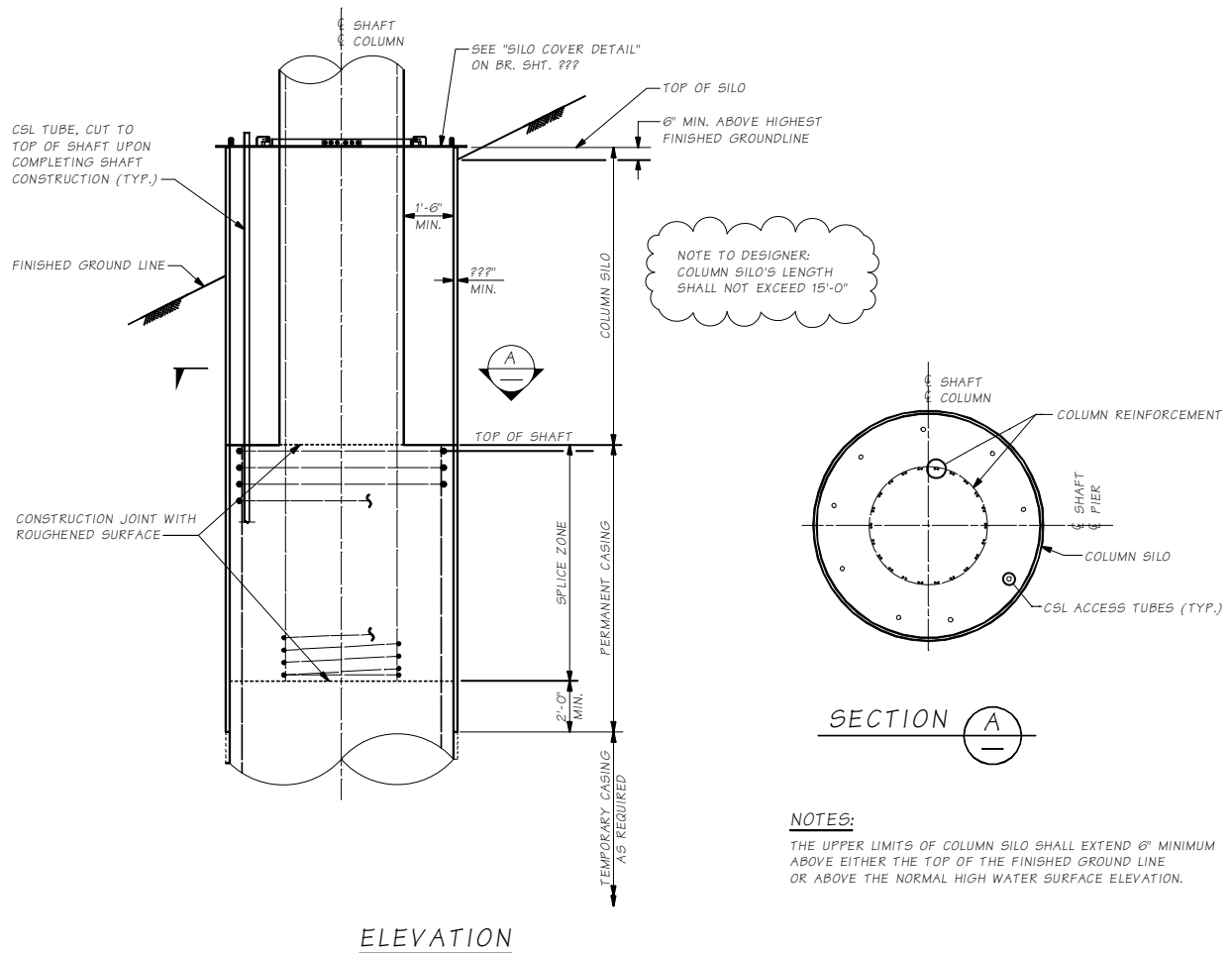
Column silos formed by other methods, such as corrugated metal pipes, may be considered if the general requirements above are satisfied.

D. Column Silo Covers and Access Hatches

A column silo cover, including access hatches, shall be specified in the Contract Plans as shown in Appendix 7.3-A1-1. Column silo covers and access hatches shall be painted in accordance with *Standard Specifications* Section 6-07.3(9).

Column silo covers shall be protected from vehicular loading. Column silo covers shall be capable of sliding on top of the column silo and shall not restrain column lateral displacement demands. Obstructions to the column silo cover sliding such as barriers or inclined slopes are not allowed adjacent to the column silo where they may interfere with column lateral displacement demands. Column silo covers and tops of column silos shall be level.

Sufficient access hatches shall be provided in the column silo cover so that all surfaces of the column and the column silo can be inspected. Access hatches shall include a minimum clear opening of 1'-0" × 1'-0" to accommodate the lowering of pumping and inspection equipment into the column silo. Access hatches for direct personnel access shall have a minimum clear opening of 2'-0" square. Column silo covers shall be designed to be removable by maintenance and inspection personnel. Public access into the column silo shall be prevented.



Column Silo on Shaft Foundation
Figure 7.3.4-1

7.3.5 Column Reinforcement

A. Reinforcing Bar Material

Steel reinforcing bars for all bridge substructure elements (precast and cast-in-place) shall be in accordance with Section 5.1.2.

B. Longitudinal Reinforcement

The reinforcement ratio is the steel area divided by the gross area of the section (A_s/A_g). The maximum reinforcement ratio shall be 0.04 in SDCs A, B, C and D. The minimum reinforcement ratio shall be 0.007 for SDC A, B, and C and shall be 0.01 for SDC D.

For bridges in SDC A, if oversized columns are used for architectural reasons, the minimum reinforcement ratio of the gross section may be reduced to 0.005, provided all loads can be carried on a reduced section with similar shape and the reinforcement ratio of the reduced section is equal to or greater than 0.01 and $0.133f'_c/f_y$. The column dimensions are to be reduced by the same ratio to obtain the similar shape.

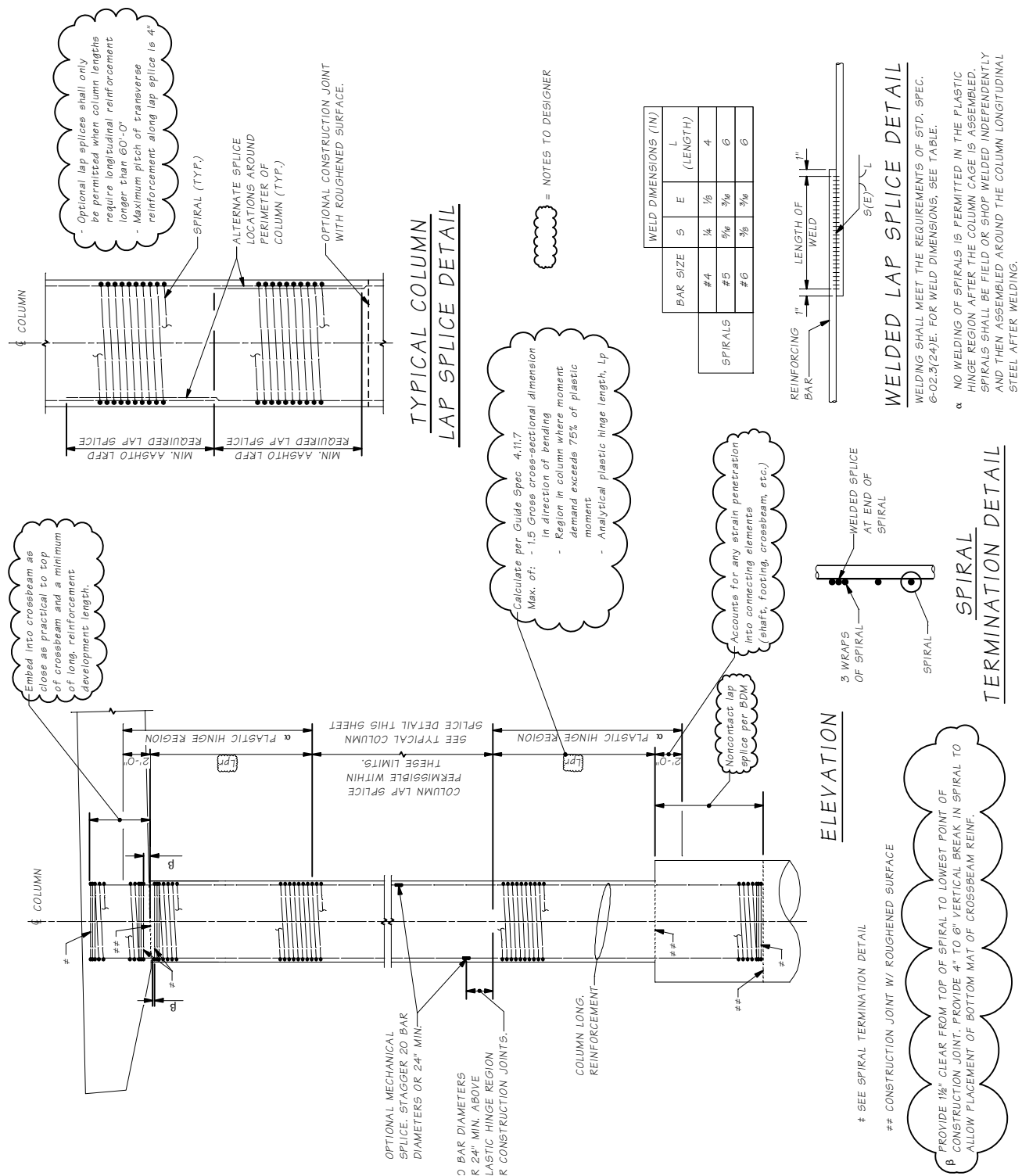
The reinforcement shall be evenly distributed and symmetric within the column.

C. Longitudinal Splices

In general, no splices are allowed when the required length of longitudinal reinforcement is less than the conventional mill length (typically 60-feet). Splicing of longitudinal reinforcement shall be outside the plastic hinge regions. But in SDC A, splices need only be located a minimum of 1.5 times the column diameter from the top and bottom of the column. The bridge plans shall clearly identify the limits of the permissible splice zone. [Figure 7.3.5-1](#) shows standard column reinforcement details.

For bridges in SDC's A and B, no lap splices shall be used for #14 or #18 bars. Either lap or mechanical splices may be used for #11 bars and smaller. Lap splices shall be detailed as Class B splices. The smaller bars in the splice determine the length of lap splice required. When space is limited, #11 bars and smaller can use welded splices, an approved mechanical butt splice, or the upper bars can be bent inward (deformed by double bending) to lie inside and parallel to the lower bars. The spacing of the transverse reinforcement over the length of a lap splice shall not exceed 4-inches or one-quarter of the minimum member dimension.

For bridge in SDC's C and D, bars shall be spliced using mechanical splices meeting the requirements of [Standard Specifications](#) Section 6-02.3(24)F. Splices shall be staggered. The distance between splices of adjacent bars shall be greater than the maximum of 20 bar diameters or 24-inches.



Column Splice and Plastic Hinge Region Details
Figure 7.3.5-1

D. Longitudinal Development

1. Crossbeams

Development of longitudinal reinforcement shall be in accordance with AASHTO Seismic, Sections 8.8.4 and 8.8.8. Column longitudinal reinforcement shall be extended into crossbeams as close as practically possible to the opposite face of the crossbeam (below the bridge deck reinforcement).

For precast prestressed concrete girder bridges in SDC A and B with fixed diaphragms at intermediate piers, column longitudinal reinforcement may be terminated at top of lower crossbeam, provided that adequate transfer of column forces is provided.

For precast prestressed concrete girder bridges in SDC C and D with two-stage fixed diaphragms at intermediate piers, all column longitudinal reinforcement should extend to the top of the cast-in-place concrete diaphragm (upper crossbeam) above the lower crossbeam. Careful attention should be given that column reinforcement does not interfere with extended strands projecting from the end of the prestressed concrete girders. In case of interference, column longitudinal reinforcement obstructing the extended strands may be terminated at top of the lower crossbeam, and shall be replaced with equivalent full-height stirrups extending from the lower to upper crossbeam within the effective width as shown in [Figure 7.3.5-2](#). All stirrups within the effective zone, based on an approximate strut-and-tie model, may be used for this purpose. The effective zone shall be taken as column diameter plus depth of lower crossbeam provided that straight column bars are adequately developed into the lower crossbeam. The effective zone may be increased to the column diameter plus two times depth of lower crossbeam if headed bars are used for column longitudinal reinforcement.

If the depth of lower crossbeam is less than 1.25 times the tension development length required for column reinforcement, headed bars shall be used. Heads on column bars terminated in the lower crossbeam are preferable from a structural perspective. However, extra care in detailing during design and extra care in placement of the column reinforcement during construction is required. Typically the heads on the column bars will be placed below the lower crossbeam top mat of reinforcement. Headed reinforcement shall conform to the requirements of ASTM 970 Class HA.

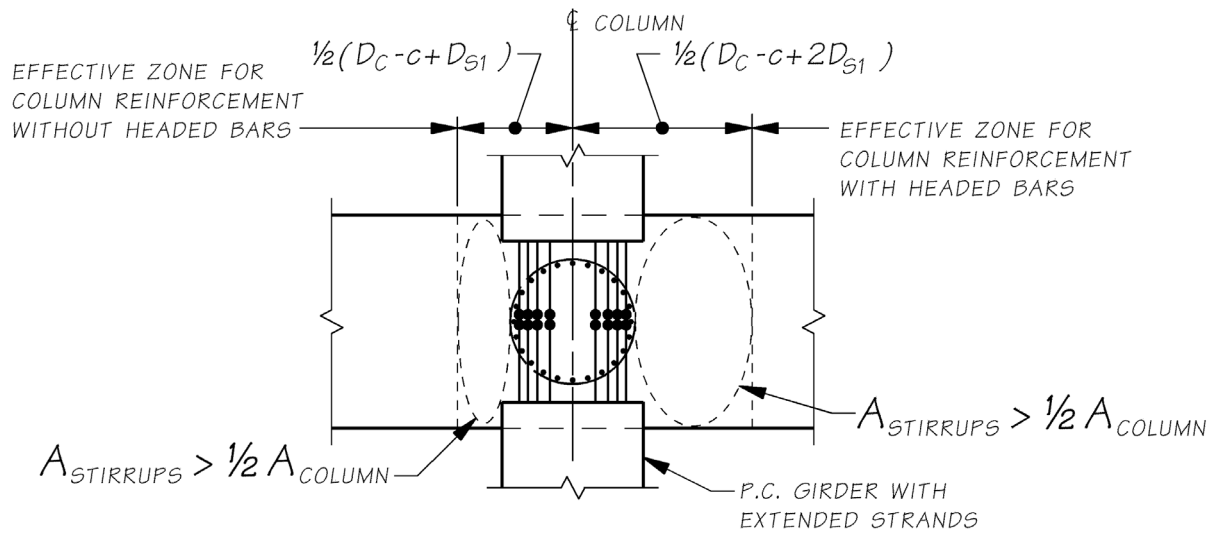
Transverse column reinforcement only needs to extend to the top of the lower crossbeam just below the top longitudinal steel. However, when the joint shear principal tension is less than $0.11\sqrt{f'_c}$, minimum cross tie reinforcement shall be placed acting across the upper cross beam in accordance with the AASHTO Seismic, Sections 8.13.3. The minimum cross tie reinforcement shall provide at least as much confining pressure at yield as the column spiral can provide at yield. This pressure may be calculated assuming hydrostatic conditions. If the joint shear principal tension exceeds $0.11\sqrt{f'_c}$, then additional joint reinforcement as outlined by AASHTO Seismic, Sections 8.13.3 shall be provided. With the exception of J-bars, the additional reinforcement shall be placed in the upper and lower crossbeam. The cross tie reinforcement may be placed with a lap splice in the center of the joint.

Large columns or columns with high longitudinal reinforcement ratios may result in closely spaced stirrups with little clear space left for proper concrete consolidation outside the reinforcement. In such cases, either hanger reinforcement comprised of larger bars with headed anchors may be used in the effective zone shown in [Figure 7.3.5-2](#) or supplemental stirrups may be placed beyond the effective zone. Hanger reinforcement in the effective zone is preferred.

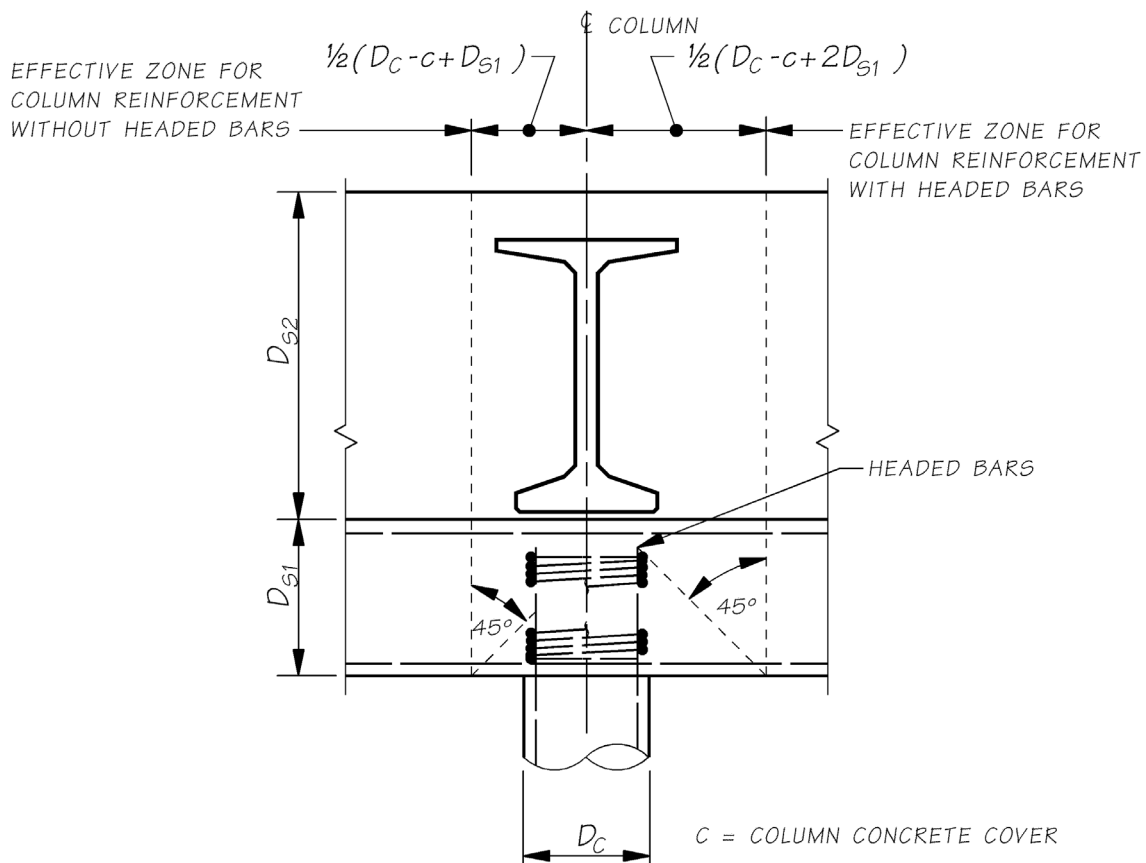
The designer is encouraged to include interference detail/plan views of the crossbeam reinforcement in relation to the column steel in the contract drawings. Suggested plans include the views at the lower stage crossbeam top reinforcement and the upper crossbeam top reinforcement.

2. Footings

Longitudinal reinforcement at the bottom of a column should extend into the footing and rest on the bottom mat of footing reinforcement with standard 90° hooks. In addition, development of longitudinal reinforcement shall be in accordance with AASHTO Seismic, Sections 8.8.4 and AASHTO LRFD Sections 5.11.2.1. Headed bars may be used for longitudinal reinforcement at the bottom of columns. The head shall be placed at least 3-inches below the footing bottom mat reinforcement. This may require the footing to be locally thickened in the region of the column to provide cover for the bottom of the headed bars. The head of the rebar placed below the footing bottom reinforcement mat shall not contribute to the compressive capacity of the rebar.



PLAN



ELEVATION

Longitudinal Development Into Crossbeams

Figure 7.3.5-2

3. Shafts

Column longitudinal reinforcement in shafts is typically straight. Embedment shall be a minimum length equal to $l_{ns} = l_s + s$ (per TRAC Report WA- RD 417.1 titled “*Noncontact Lap Splices in Bridge Column-Shaft Connections*”).

Where:

l_s = the larger of $1.7 \times l_{ac}$ or $1.7 \times l_d$

l_{ac} = development length from the AASHTO Seismic 8.8.4 for the column longitudinal reinforcement.

l_d = tension development length from AASHTO LRFD Section 5.11.2.1 for the column longitudinal reinforcement.

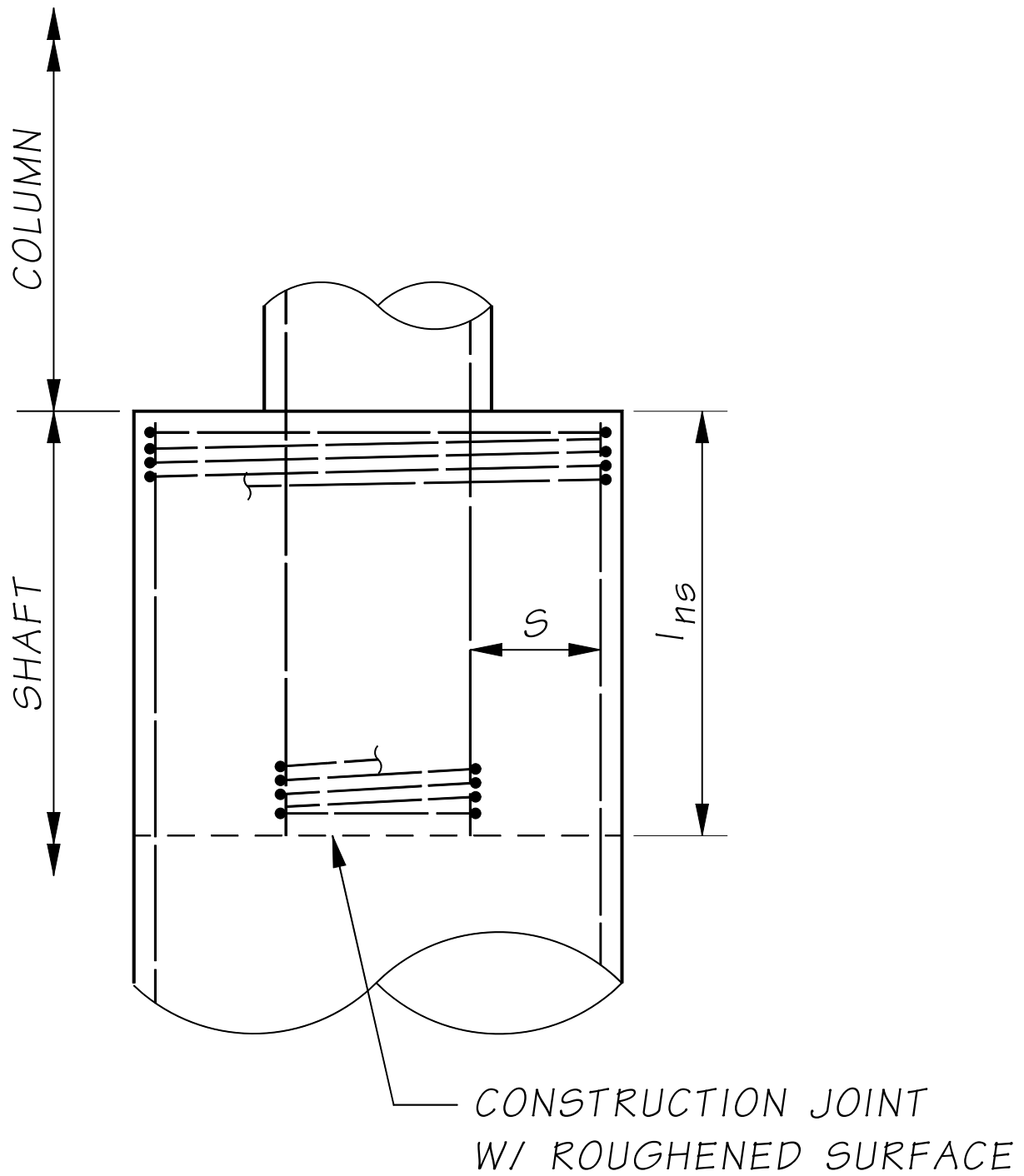
s = distance between the shaft and column longitudinal reinforcement

The requirements of the AASHTO Seismic, Section 8.8.10 for development length of column bars extended into oversized pile shafts for SDC C and D shall not be used.

The factor of 1.7 used in determining l_s represents a Class C lap splice modification factor from previous versions of AASHTO LRFD. Although the concept of a Class C splice is no longer applicable, the factor is still necessary to match the recommendations of TRAC Report WA-RD 417.1.

The modification factor in 5.11.2.1.3 that allows l_d to be decreased by the ratio of (As required)/(As provided), shall not be used. Using this modification factor would imply that the reinforcement does not need to yield to carry the ultimate design load. This may be true in other areas. However, our shaft/column connections are designed to form a plastic hinge, and therefore the reinforcement shall have adequate development length to allow the bars to yield.

See [Figure 7.3.5-3](#) for an example of longitudinal development into shafts.



Longitudinal Development Into Shafts
Figure 7.3.5-3

E. Transverse Reinforcement

All transverse reinforcement in columns shall be deformed. Although allowed in the AASHTO LRFD, plain bars or plain wire may not be used for transverse reinforcement.

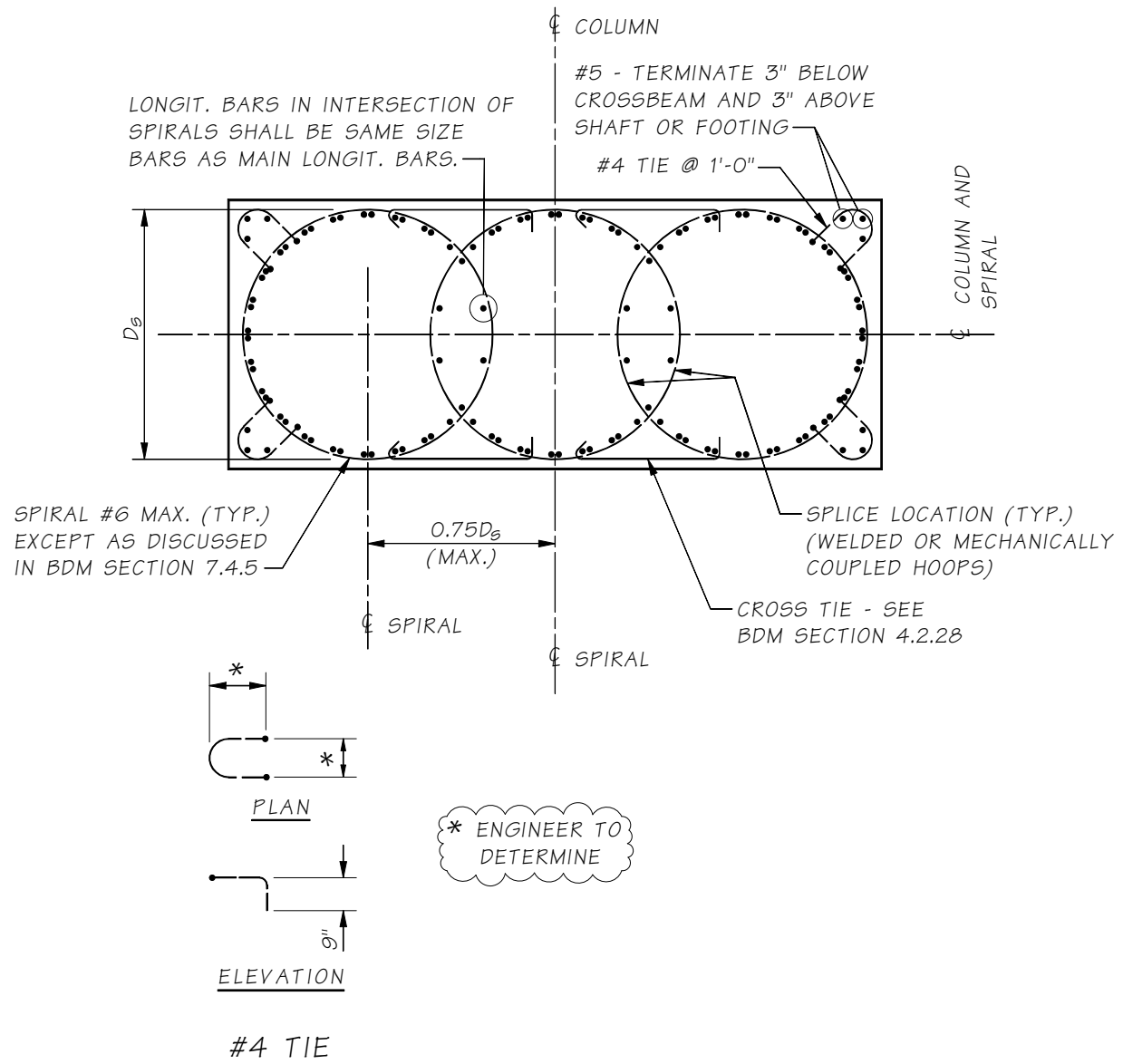
Columns in SDC A may use spirals, circular hoops, or rectangular hoops and crossties. Spirals are the preferred confinement reinforcement and shall be used whenever a #6 spiral is sufficient to satisfy demands. When demands require reinforcement bars greater than #6, circular hoops of #7 through #9 may be used. Bundled spirals shall not be used for columns or shafts. Also, mixing of spirals and hoops within the same column is not permitted by the AASHTO Seismic

Specifications. [Figure 7.3.5-4](#) and [7.3.5-5](#) show transverse reinforcement details for rectangular columns in high and low seismic zones, respectively.

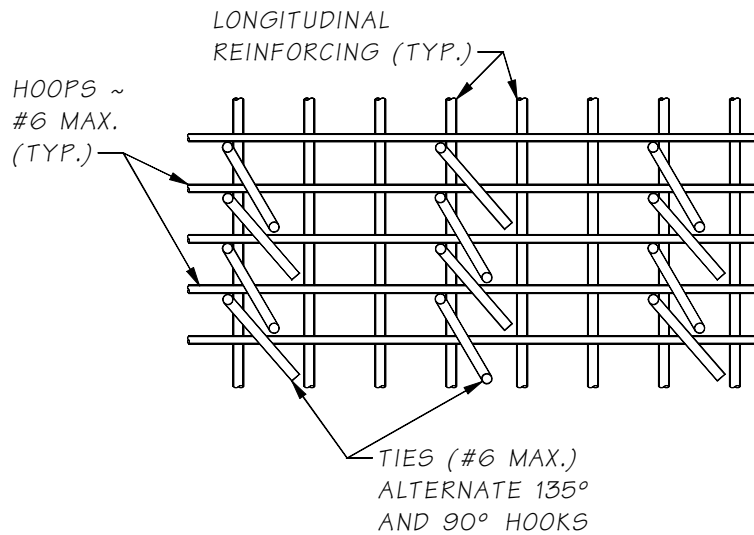
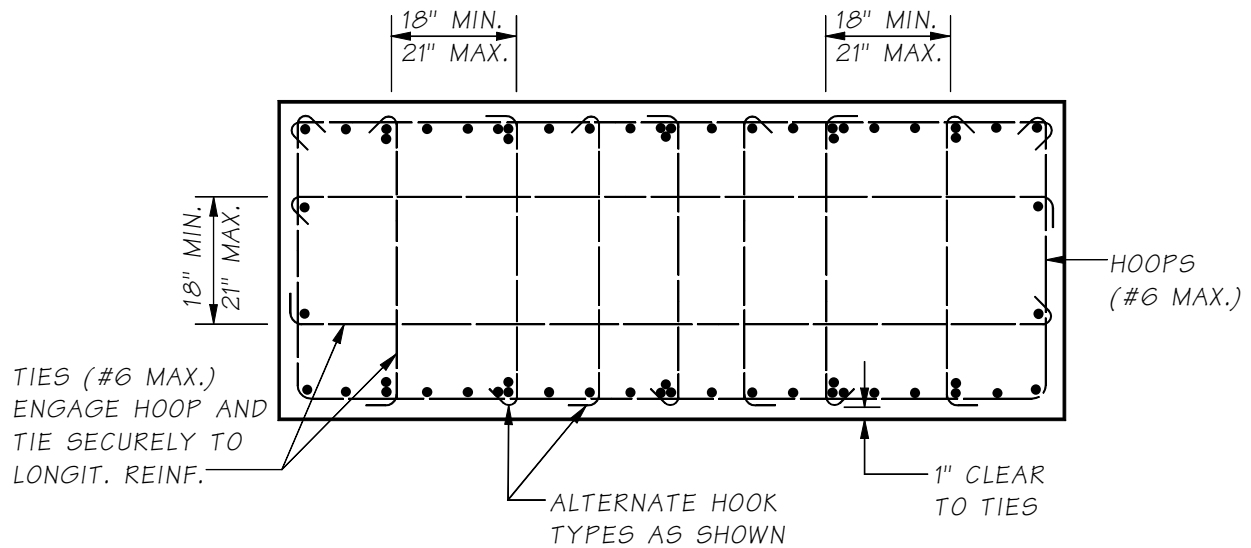
Columns in SDCs C and D shall use hoop reinforcement. Hoop reinforcement shall be circular where possible, although rectangular hoops with ties may be used when large, odd shaped column sections are required. Where the column diameter is 3-feet or less, the WSDOT Steel Specialist shall be contacted regarding the constructability of smaller diameter welded hoops.

When rectangular hoops with ties are used, consideration shall be given to column constructability. Such considerations can include, but are not limited to a minimum of 2'-6" by 3'-0" open rectangle to allow access for the tremie tube and construction workers for concrete placement, in-form access hatches, and/or external vibrating.

A larger gap between transverse reinforcement should be provided at the top of columns to allow space for the crossbeam longitudinal reinforcement to pass. In SDC's C & D, the gap shall not exceed the maximum spacing for lateral reinforcement in plastic hinge regions specified in AASHTO Seismic, Section 8.8.9. This can be of particular concern in bridge decks with large superelevation cross slopes.



Constant and Tapered Rectangular Column Section SDCs C and D
Figure 7.3.5-4



Constant and Tapered Rectangular Column Section SDCs A and B
Figure 7.3.5-5

F. Spiral Splices and Hoops

Welded laps shall be used for splicing and terminating spirals and shall conform to the details shown in [Figure 7.3.5-6](#). Only single sided welds shall be used, which is the preferred method in construction. Spirals or butt-welded hoops are required for plastic hinge zones of columns. Lap spliced hoops are not permitted in columns in any region.

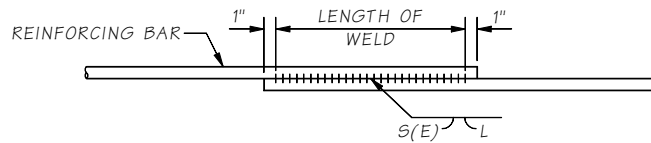
Although hooked lap splices are structurally acceptable, and permissible by AASHTO LRFD for spirals or circular hoops, they shall not be allowed due to construction challenges. While placing concrete, tremies get caught in the protruding hooks, making accessibility to all areas and its withdrawal cumbersome. It is also extremely difficult to bend the hooks through the column cage into the core of the column.

When welded hoops are used, the plans shall show a staggered pattern around the perimeter of the column so that no two adjacent welded splices are located at the same location. Also, where interlocking hoops are used in rectangular or non-circular columns, the splices shall be located in the column interior.

Circular hoops for columns shall be shop fabricated using a manual direct butt weld or resistance butt weld. Currently, a Bridge Special Provision has been developed to cover the fabrication requirements of hoops for columns and shafts, which may eventually be included in the [Standard Specifications](#). Manual direct butt welded hoops require radiographic nondestructive examination (RT), which may result in this option being cost prohibitive at large quantities.

Columns with circular hoop reinforcement shall have a minimum 2" concrete cover to the hoops to accommodate resistance butt weld "weld flash" that can extend up to ½" from the bar surface.

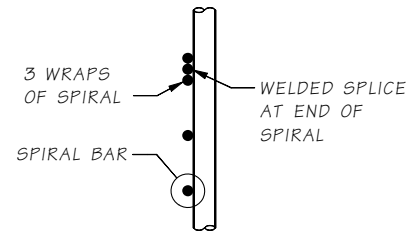
Field welded lap splices and termination welds of spirals of any size bar are not permitted in the plastic hinge region including a zone extending 2'-0" into the connected member and should be clearly designated on the contract plans. If spirals are welded while in place around longitudinal steel reinforcement, there is a chance that an arc can occur between the spiral and longitudinal bar. The arc can create a notch that can act as a stress riser and may cause premature failure of the longitudinal bar when stressed beyond yield. Because high strains in the longitudinal reinforcement can penetrate into the connected member, welding is restricted in the first 2'-0" of the connected member as well. It would be acceptable to field weld lap splices of spirals off to the side of the column and then slide into place over the longitudinal reinforcement.



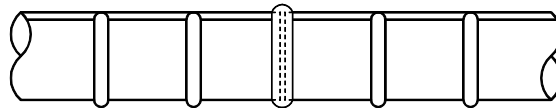
WELDED LAP SPLICE DETAIL

WELDED LAP SPLICE IS SUITABLE FOR SPIRALS IN COLUMNS AND SHAFTS UP TO BAR SIZE #6. LAP SPLICE FOR BAR SIZES #7 TO #9 ARE ONLY INTENDED FOR SHAFT HOOPS. WELDING SHALL MEET THE REQUIREMENTS OF STD. SPEC. 6-02.3(24)E. FOR WELD DIMENSIONS, SEE TABLE BELOW.

		WELD DIMENSIONS (IN)		
	BAR SIZE	S	E	L (LENGTH)
SPIRALS	#4	1/4	1/8	4
	#5	5/16	3/16	6
	#6	3/8	3/16	6
HOOPS FOR SHAFTS	#7	7/16	1/4	7
	#8	1/2	1/4	8
	#9	9/16	5/16	8

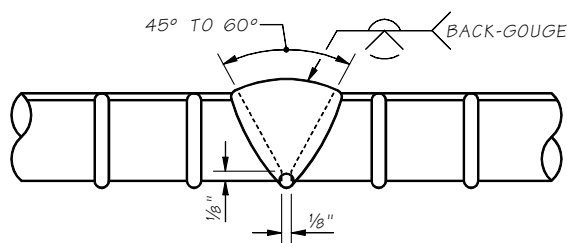


SPIRAL TERMINATION DETAIL

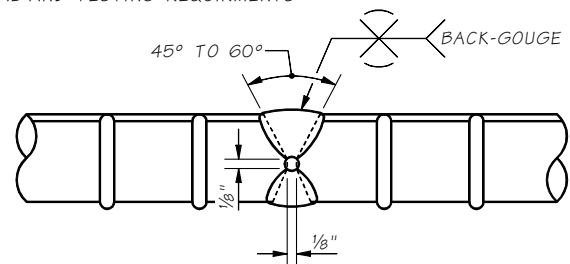


RESISTANCE BUTT JOINT DETAIL

SEE SPECIAL PROVISIONS FOR APPROVAL AND TESTING REQUIREMENTS



SINGLE V-GROOVE WELD
AWS D1.4 FIGURE 3.2(A)



DOUBLE V-GROOVE WELD
AWS D1.4 FIGURE 3.2(B)

MANUAL DIRECT BUTT JOINT DETAILS

ALL BACKING SHALL BE REMOVED.
SEE SPECIAL PROVISIONS FOR RT TESTING FREQUENCY

Welded Spiral Splice and Butt Splice Details
Figure 7.3-5-6

7.3.6 Column Hinges

Column hinges of the type shown in [Figure 7.3.6-1](#) were built on past WSDOT bridges. Typically they were used above a crossbeam or wall pier. These types of hinges are suitable when widening an existing bridge crossbeam or wall pier with this type of detail. The area of the hinge bars in square inches is as follows:

$$A_s = \frac{\frac{P_u}{2} + \sqrt{\left(\frac{P_u}{4}\right)^2 + V_u^2}}{0.85 * F_y * \cos \theta} \quad (7.3.6-1)$$

Where:

P_u is the factored axial load

V_u is the factored shear load

F_y is the reinforcing yield strength (60 ksi)

θ is the angle of the hinge bar to the vertical

The development length required for the hinge bars is 1.25 l_d . All applicable modification factors for development length in AASHTO LRFD Section 5.11.2 may be used when calculating l_d . Tie and spiral spacing shall conform to AASHTO LRFD confinement and shear requirements. Ties and spirals shall not be spaced more than 12" (6" if longitudinal bars are bundled). Premolded joint filler should be used to assure the required rotational capacity. There should also be a shear key at the hinge bar location.

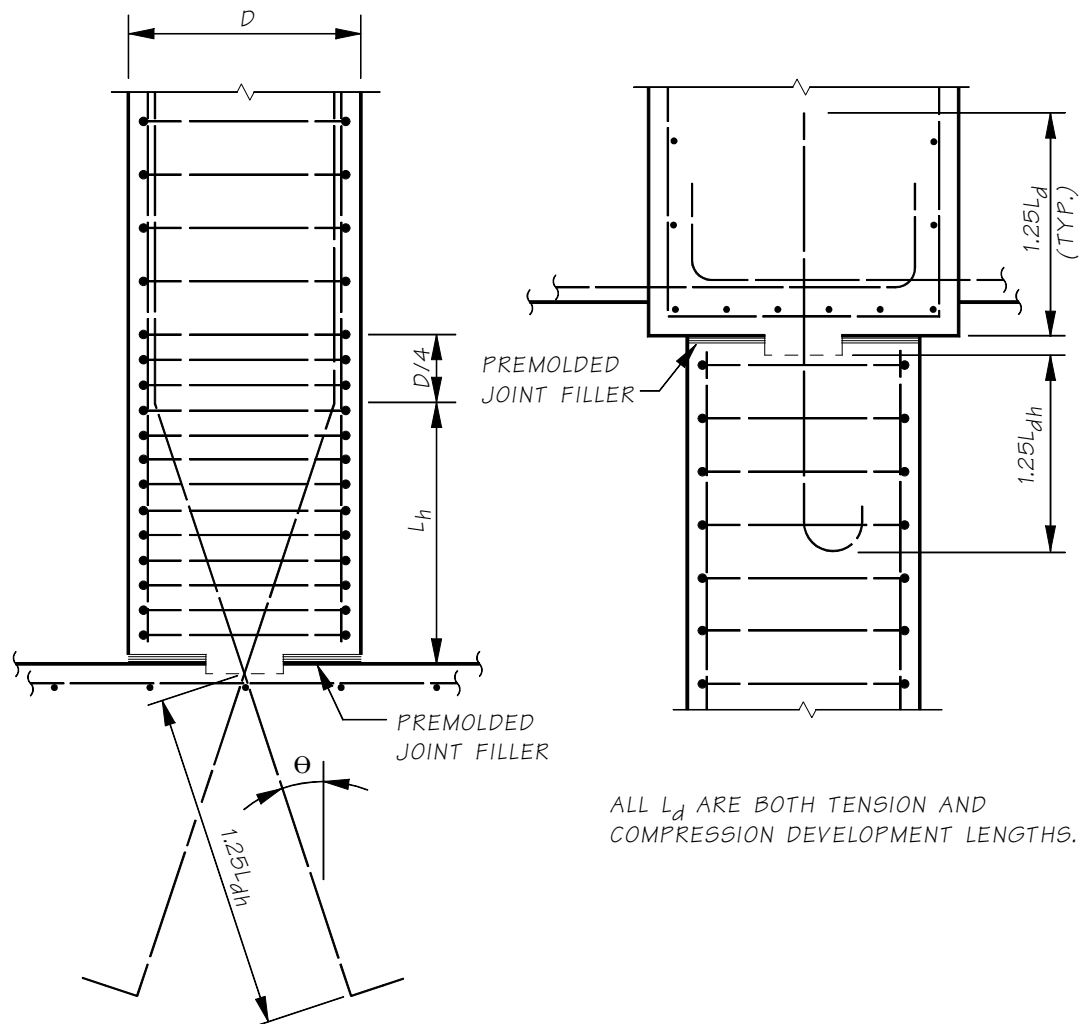
When the hinge reinforcement is bent, additional confinement reinforcing may be necessary to take the horizontal component from the bent hinge bars. The maximum spacing of confinement reinforcing for the hinge is the smaller of that required above and the following:

$$S_{max} = \frac{A_v F_y}{\left[\frac{P_u \tan \theta}{.85 l_h} + \frac{V_s}{d} \right]} \quad (7.3.6-2)$$

Where:

A_v , V_s , and d are as defined in AASHTO Article "Notations" l_h is the distance from the hinge to where the bend begins

Continue this spacing one-quarter of the column width (in the plane perpendicular to the hinge) past the bend in the hinge bars.



Hinge Details
Figure 7.3.6-1

7.3.7 Reduced Column Section

Reduced column sections decrease overstrength plastic demands into the foundation. Traditional column designs are preferred over this detail, but this may be used if it is determined that traditional details will not satisfy the design code requirements due to architectural, balanced stiffness, or other project specific requirements. The reduction at the base of the column shall be designed as described below and detailed as shown in [Figure 7.3.7-1](#). The concept is shown in [Figure 7.3.7-1](#) for a spread footing foundation, but could be used for shaft and pile supported foundations also. Similar checks are required if the reduced section is placed at the crossbeam, along with any additional checks required for those sections. One such additional check is joint shear in the crossbeam based on the overstrength plastic capacity of the reduced column section.

The design and detail at the top of columns, for architectural flares, is similar.

A. Inner Concrete Column

1. Longitudinal Reinforcement

- a. The longitudinal inner column reinforcement shall extend a distance of L_{ns} into the column and shall be set on top of bottom mat reinforcement of foundation with standard 90° hooks.

$$L_{ns} = L_s + sc + L_p \quad (7.3.7-1)$$

Where:

L_s = The larger of $1.7 \times L_{ac}$ or $1.7 \times L_d$ (for Class C lap splice)

L_{ac} = Development length of bar from the AASHTO Seismic Section 8.8.4

L_d = Tension development length from AASHTO LRFD Section 5.11.2.1
(Note: All applicable modification factors for L_d may be used except for the reduction specified in Section 5.11.2.2.2 for A_s required/ A_s provided)

sc = Distance from longitudinal reinforcement of outer column to inner column.

L_p = Analytical Plastic Hinge Length defined in the AASHTO Seismic Section 4.11.6-3.

- b. The longitudinal reinforcing in the inner column shall meet all the design checks in the AASHTO Seismic and AASHTO LRFD. Some specific checks of the inner column (inner core) will be addressed as follows:
 - i. A shear friction check shall be met using the larger of the overstrength plastic shear (V_{po}) or the ultimate shear demand from strength load cases at the hinge location. The area of longitudinal inner column reinforcement, A_{st} , in excess of that required in the tensile zone for flexural resistance (usually taken as $\frac{1}{2}$ the total longitudinal bars) may be used for the required shear friction reinforcement, A_{vf} .
 - ii. The flexural capacity of the inner column shall be designed to resist the strength load cases and meet cracking criteria of the service load cases. Special consideration shall be given to construction staging load cases where the column stability depends on completion of portions of the superstructure.

- iii. The axial resistance of the inner column shall meet the demands of strength load cases assuming the outer concrete has cracked and spalled off. The gross area, A_g , shall be the area contained inside the spiral reinforcement.
- iv. The inner core shall be designed and detailed to meet all applicable requirements of AASHTO Seismic Section 8.

2. Transverse Reinforcement

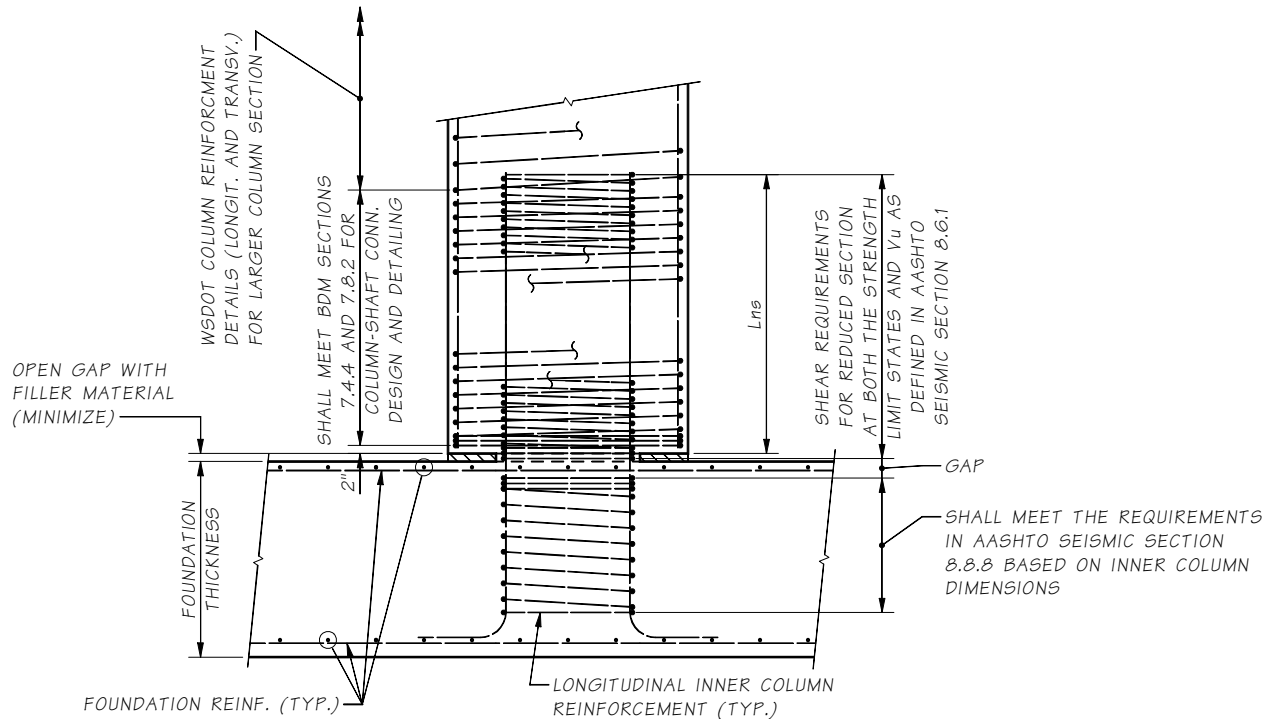
- a. The portion of the transverse reinforcement for the inner core, inside the larger column dimension (above the foundation), shall meet all the requirements of the AASHTO Seismic and AASHTO LRFD. The demand shall be based on the larger of the overstrength plastic shear demand (V_{po}) of the inner column or the ultimate shear demand from strength load cases at the hinge location. The transverse reinforcement shall be extended to the top of the longitudinal reinforcement for the inner column (L_{ns}).
- b. The portion of transverse reinforcement for the inner core, in the foundation, shall meet the minimum requirements of the AASHTO Seismic, Section 8.8.8, for compression members, based on the dimensions of the inner column. This reinforcement shall be extended to the bend radius of the of the longitudinal inner column reinforcement for footings or as required for column-shaft connections.
- c. A gap in the inner column transverse reinforcement shall be sized to allow the foundation top mat reinforcement and foundation concrete to be placed prior to setting the upper portion of the transverse inner column reinforcement. This gap shall be limited to 5"; a larger gap will require the WSDOT Bridge Design Engineer's approval. The spiral reinforcement above the footing shall be placed within 1" of the top of footing to reduce the required gap size. The WSDOT Spiral termination details will be required at each end of this gap, the top of the upper transverse reinforcement, but not the bottom of the lower transverse reinforcement with spread footings.

3. Analytical Plastic Hinge Region

- a. The analytical plastic hinge length of the reduced column section shall be based on horizontally isolated flared reinforced concrete columns, using Equation 4.11.6-3 of the AASHTO *Seismic Specifications*.
- b. The end of the column which does not have a reduced column section shall be based on Equation 4.11.6-1 of the AASHTO *Seismic Specifications*.

B. Outer Concrete Column

1. The WSDOT Bridge and Structures Office normal practices and procedures shall be met for the column design, with the following exceptions:
 - i. The end with the reduced column shall be detailed to meet the seismic requirements of a plastic hinge region. This will ensure that if a plastic hinge mechanism is transferred into the large column shape, it will be detailed to develop such hinge. The plastic shear this section shall be required to resist shall be the same as that of the inner column section.
 - ii. The WSDOT spiral termination detail shall be placed in the large column at the reduced section end, in addition to other required locations.
 - iii. In addition to the plastic hinge region requirements at the reduced column end, the outer column spiral reinforcement shall meet the requirements of the WSDOT *Noncontact Lap Splices in Bridge Column-Shaft Connections*. The k factor shall be taken as 0.5 if the column axial load, after moment distribution, is greater than $0.10f'cAg$ and taken as 1.0 if the column axial load is in tension. Ag shall be taken as the larger column section. Linear interpolation may be used between these two values.
2. The column end without the reduced column section shall be designed with WSDOT practices for a traditional column, but shall account for the reduced overstrength plastic shear, applied over the length of the column, from the overstrength plastic capacities at each column end.

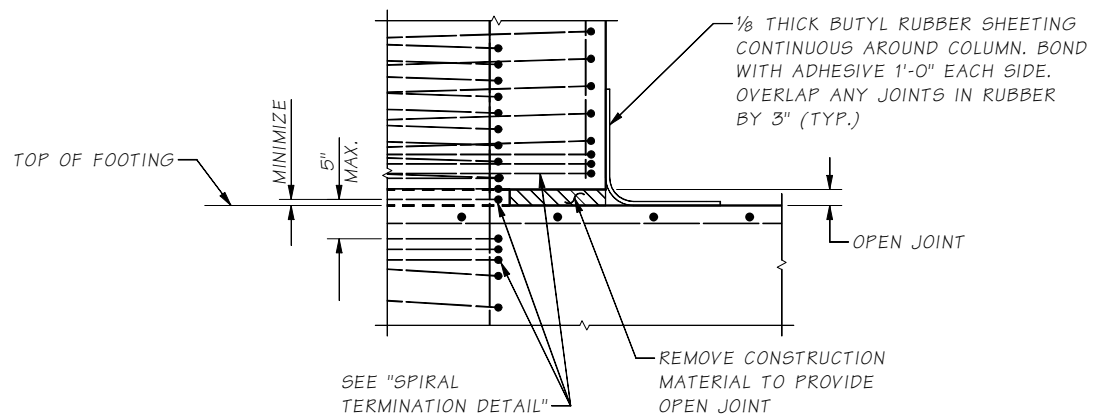


Reduced Column Section at Bottom of Column
Figure 7.3.7-1

C. Gap in Concrete at Reduced Column Section

The gap shall be minimized, but shall not be less than 2". It shall also be designed to accommodate the larger of 1.5 times the calculated service, strength or extreme event elastic rotation demand or the plastic rotation capacity, as determined from an inelastic pushover analysis. In no loading condition shall the edge of the larger column section contact the footing.

The gap shall be constructed with a material sufficiently strong to support the wet concrete condition. The material in the gap must keep soil or debris out of the gap for the life of the structure. This is especially important if the gap is to be buried and inspection access is difficult. If a filler material is used in this gap which can transfer compressive forces, then the gap shall be increased to account for this compressive force. If a filler material can meet construction and service requirements, it can be left in place after construction. Otherwise the gap shall be cleared and covered, or the gap shall be filled with a material that meets the service requirements. See [Figure 7.3.7-2](#).



Open Gap Detail
Figure 7.3.7-2

7.4 Crossbeams

7.4.1 General Design

The following is the recommended procedure for strength design and load rating of a two-stage, integral, non-prestressed crossbeam at multicolumn intermediate piers supporting precast superstructures. The procedure is based on beam theory for a tension-controlled element, which is an acceptable method for design and load rating. The strut and tie method is also an acceptable procedure.

A. Stage I Design

1. Obtain load effects on the lower stage I crossbeam. The dead loads on the crossbeam typically include the self-weight of the lower crossbeam, girders, diaphragms, bridge deck, and the dead load from the upper stage II portion of the crossbeam. Additionally:
 - A construction load equal to 15 psf over the entire deck area shall be included in the Strength I and III load combinations. This construction load is intended to account for formwork, work decks, miscellaneous materials and equipment, and any construction related live loads (Bidwell finishing machine, etc.). The load factor for the construction load shall be in accordance with [Section 3.6](#).
 - Strength IV, which is the load combination relating to very high dead load to live load force effects, need not be considered for this condition as clarified in the 7th Edition of the AASTHO *LRFD Bridge Design Specifications*.
 - Torsion due to unbalanced loading on the stage I crossbeam shall be considered. The unbalanced cases shall include at a minimum, the case where all girders are set on one side of the pier and the case where all girders are in place on both sides but the deck is only placed on one side of the pier.
2. Design the longitudinal reinforcement in the top and bottom of the lower crossbeam for the controlling strength load case.
3. Design the transverse reinforcement in the lower crossbeam considering the controlling strength shear demands, including the construction load previously described. Only the transverse reinforcement that is fully enclosed and anchored within the stage I crossbeam shall be considered to be effective.
4. Check minimum flexural and shear reinforcement, crack control by distribution of flexural steel, and temperature and shrinkage requirements.

B. Stage II Design

1. The stage II crossbeam is full depth and fully composite with the stage I lower portion. Apply the total DC, DW, LL, TU, and all other applicable load effects on the stage II crossbeam, including loads which were applied to the stage I crossbeam. This is a simplified procedure assuming the entire full depth crossbeam is cast monolithic, and it may imply some load redistribution. The construction load of 15 psf need not be considered in the stage II analysis.
2. Design the top and bottom longitudinal reinforcement for the controlling strength load case. For the bottom reinforcement, use the largest required steel area for either the stage I or stage II case. The crossbeam may either be treated as a rectangular section, or the effective width of the deck can be considered as a "T" section. The top longitudinal reinforcement in the stage I lower crossbeam is typically ignored.
3. Design the transverse reinforcement for the combination of the controlling strength load case. For most crossbeams, torsion in the stage II analysis can be ignored. The concrete shear resistance shall be computed assuming the full depth section. The transverse steel shear resistance shall be based on the sum of V_{s1} and V_{s2} . The V_{s1} resistance shall be based on the shear steel and depth d_v from the stage I crossbeam. The V_{s2} resistance shall be based on the shear steel that runs full depth and a d_v value corresponding to the full depth of the crossbeam. Each vertical leg of transverse reinforcement shall only be considered in either V_{s1} or V_{s2} , never both. In the zones where girders are located, the outer rows of full depth shear reinforcement is terminated. For inner rows of full depth shear reinforcement, the spacing may be increased where there is congestion due to extended girder strands, however the designer shall maintain as much shear reinforcement in these zones as practicably possible.

In the regions where shear reinforcement spacing varies across the shear failure plane, the resistance may be determined based on the average shear reinforcement area per unit length within the shear failure plane. The average shear reinforcement area per unit length may be determined as follows:

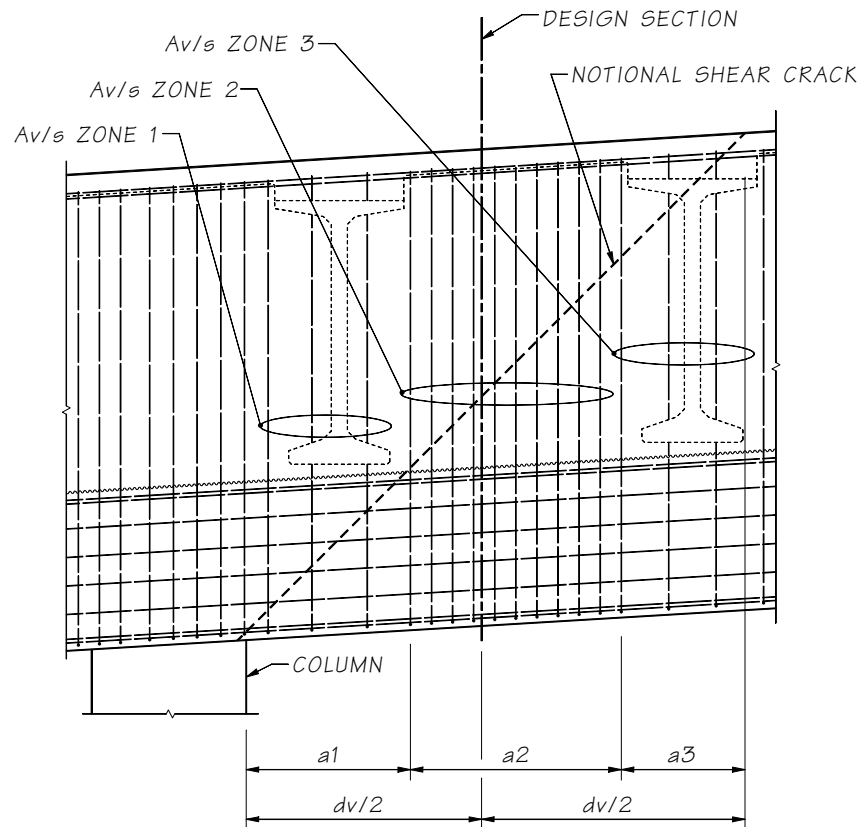
$$\left(\frac{A_v}{s}\right)_{avg} = \frac{\sum \left(\frac{A_v}{s}\right)_i a_i}{d_v \cot \theta} \quad 7.4.1-1$$

Where:

- A_v = area of shear reinforcement
- s = spacing of shear reinforcement
- a_i = horizontal distance of shear plane crossing the stirrup zone i
- d_v = effective shear depth
- $\cot \theta$ = for the simplified method, $\theta = 45$ degrees and $\cot \theta = 1.0$

See Figure 7.4.1-1. For deep girders, the shear failure plane at 45 degrees will typically run beyond the girder width and will intersect shear reinforcement on either side of the girder. For smaller depth girders, such as WF42Gs, this average shear reinforcement method may not suffice and more steel will need to be placed in the girder zones, regardless of congestion constraints.

The concrete shear resistance, V_c , shall be based on the depth d_v from the stage II crossbeam.



Effective Crossbeam Shear Reinforcement

Figure 7.4.1-1

4. Check minimum flexural and shear reinforcement, crack control by distribution of flexural steel, and temperature and shrinkage requirements.

C. Other Crossbeam Types

A special case of two stage integral crossbeam is the single column “hammer head” type crossbeams. A similar methodology can be employed for these crossbeams. The top flexural reinforcement in the stage I section is designed for the applicable stage I loads. In the stage II analysis, the stage I and stage II loads are applied to the full depth section and the top reinforcement in the full depth composite section shall be designed for the total stage I and II demands. The stage I top reinforcement can be considered in the stage II analysis, although it may not be very effective. The section can be considered rectangular or as a “T” section by including deck steel reinforcement within the effective width.

Although not discussed here, capacity protection of integral crossbeams will likely control the final design. The analysis should treat the full depth crossbeam as if it was cast monolithic. Various analytical methods can be utilized to obtain the demands in the crossbeam from the column plastic moment and shear, and they shall be determined by the designer.

The analysis and design for prestressed girder expansion pier crossbeams and most crossbeams for steel plate and box girder bridges is straightforward and consists of only one stage. All dead and live loads are applied directly to the crossbeam. The designer should consider conditions where torsion may be induced by unbalanced loading during the construction and/or permanent phases. The 15 psf construction load shall not be included with any vehicle live load combinations.

Load rating of the integral 2-stage crossbeams shall follow the methods described for the Stage II analysis using the applicable load rating live loads.

7.5 Abutment Design and Details

7.5.1 General

A. Abutment Types

There are five abutment types described in the following section that have been used by the Bridge and Structures Office. Conventional stub and cantilever abutments on spread footings, piles, or shafts are the preferred abutment type for WSDOT bridges. The representative types are intended for guidance only and may be varied to suit the requirements of the bridge being designed.

Significant measures may be required to accommodate bridge security and deter inappropriate access to the bridge abutment areas. Designs may include steel security fences or concrete curtain walls. Configuring the land form at the abutment or increasing the stem wall height to deter access may also be considered. Where required, coordinate with the State Bridge and Structures Architect during final design.

1. Stub Abutments

Stub abutments are short abutments where the distance from the girder seat to top of footing is less than approximately 4 feet, see [Figure 7.5.1-1](#). The footing and wall can be considered as a continuous inverted T-beam. The analysis of this type abutment shall include investigation into both bending and shear stresses parallel to centerline of bearing. If the superstructure is relatively deep, earth pressure combined with longitudinal forces from the superstructure may become significant.

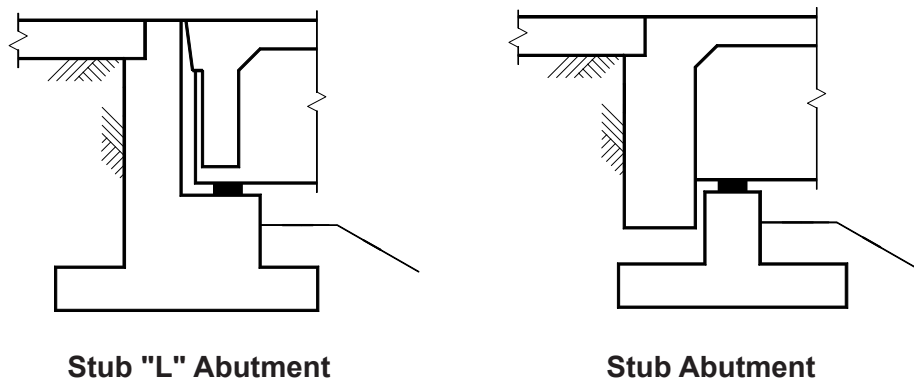


Figure 7.5.1-1

2. Cantilever Abutments

If the height of the wall from the bearing seat down to the bottom of the footing exceeds the clear distance between the girder bearings, the assumed 45° lines of influence from the girder reactions will overlap, and the dead load and live load from the superstructure can be assumed equally distributed over the abutment width. The design may then be carried out on a per-foot basis. The primary structural action takes place normal to the abutment, and the bending moment effect parallel to the abutment may be neglected in most cases. The wall is assumed to be a cantilever member fixed at the top of the footing and subjected to axial, shear, and bending loads see [Figure 7.5.1-2](#).

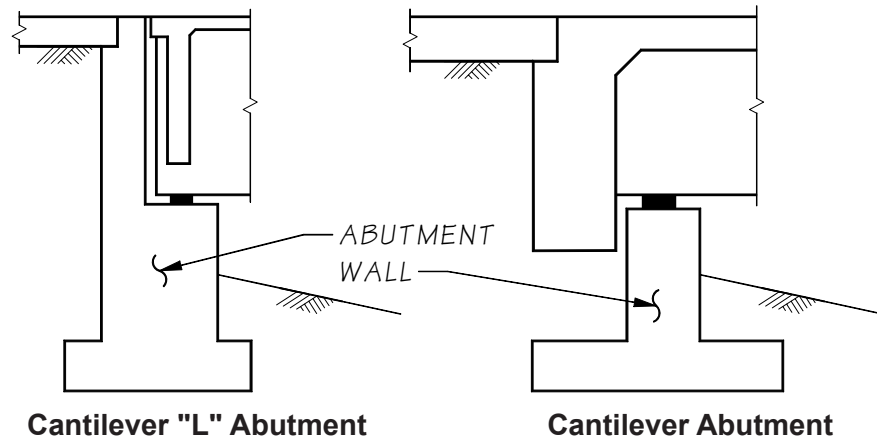
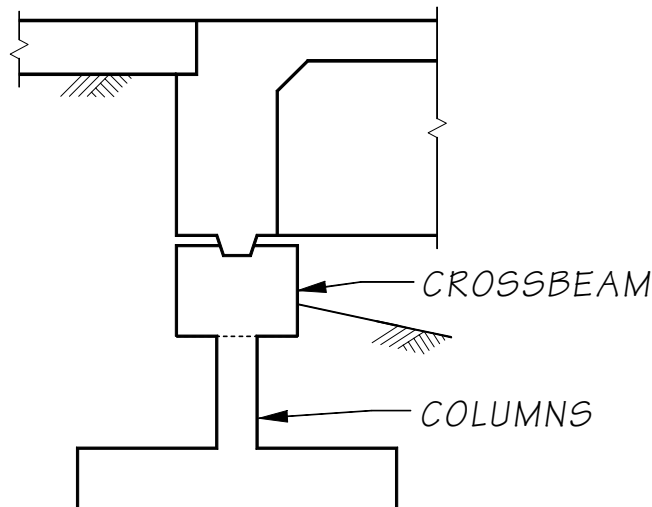


Figure 7.5.1-2

3. Spill-Through Abutments

The analysis of this type of abutment is similar to that of an intermediate pier, see [Figure 7.5.1-3](#). The crossbeam shall be investigated for vertical loading as well as earth pressure and longitudinal effects transmitted from the superstructure. Columns shall be investigated for vertical loads combined with horizontal forces acting transversely and longitudinally. For earth pressure acting on rectangular columns, assume an effective column width equal to 1.5 times the actual column width. Short, stiff columns may require a hinge at the top or bottom to relieve excessive longitudinal moments.

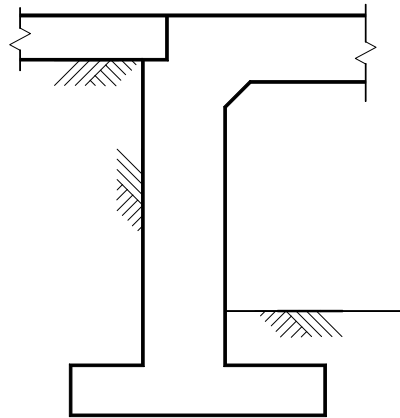


Spill-Through Abutment

Figure 7.5.1-3

4. Rigid Frame Abutments

Abutments that are part of a rigid frame are generically shown in [Figure 7.5.1-4](#). At-Rest earth pressures (EH) will apply to these structures. The abutment design should include the live load impact factor from the superstructure. However, impact shall not be included in the footing design. The rigid frame itself should be considered restrained against sidesway for live load only. AASHTO LRFD Chapter 12 addresses loading and analysis of rigid frames that are buried (box culverts).



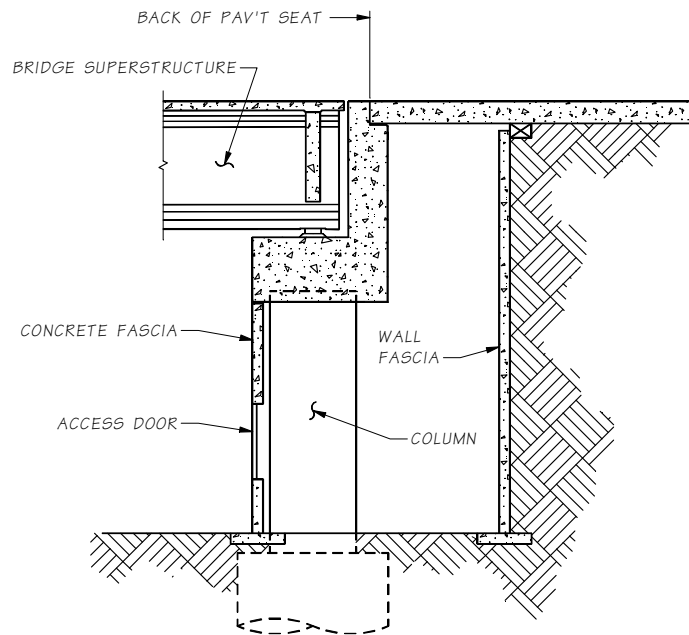
Rigid Frame Abutment
Figure 7.5.1-4

5. Bent-Type Abutments

An abutment that includes a bent cap supported on columns or extended piles or shafts is shown in [Figure 7.5.1-5](#). For structural reasons it may be required to construct a complete wall behind a bridge abutment prior to bridge construction. Bent-type abutments may be used where the abutment requires protection from lateral and vertical loads and settlement. This configuration shall only be used with the approval of the WSDOT Bridge Design Engineer. It shall not be used where initial construction cost is the only determining incentive.

A bridge approach slab shall span a maximum of 6'-0" between the back of pavement seat and the face of the approach embankment wall. The approach slab shall be designed as a beam pinned at the back of pavement seat. The approach slab shall support traffic live loads and traffic barrier reactions. The approach embankment wall shall support the vertical live load surcharge. The approach slab shall not transfer loads to the approach embankment wall facing.

An enclosing fascia wall is required to prohibit unwanted access with associated public health, maintenance staff safety, and law enforcement problems. The design shall include a concrete fascia enclosing the columns and void. The fascia shall have bridge inspection access. The access door shall be a minimum 3'-6" square with the sill located 2'-6" above finished grade. Contact the State Bridge and Structures Architect for configuration and concrete surface treatments.



Bent-Type Abutment

Figure 7.5.1-5

6. Abutments Supported by Mechanically Stabilized Earth Walls

Bridge abutments may be supported on mechanically-stabilized earth (MSE) walls. Refer to [Section 7.5.11](#) for specific requirements.

7.5.2 Embankment at Abutments

The minimum clearances for the embankment at the front face of abutments shall be as indicated on Standard Plans A-50.10.00 through A-50.40.00. At the ends of the abutment, the fill may be contained with wing walls or in the case of concrete structures, placed against the exterior girders.

The minimum clearance between the bottom of the superstructure and the embankment below shall be 3'-0" for girder bridges and 5'-0" for non-girder, slab, and box girder bridges.

The presence of a horizontal landform shelf beneath the superstructure at the abutment face may constitute an attractive nuisance. Limiting access to superstructure, by increasing the stem wall height, may also be required. Where required, coordinate with the State Bridge and Structures Architect for bridge security issues.

7.5.3 Abutment Loading

In general, bridge abutment loading shall be in accordance with AASHTO LRFD Chapter 3 and 11. The following simplifications and assumptions may be applied to the abutment design. See Section 7.7.4 for a force diagram of typical loads as they are applied to an abutment spread footing.

A. Dead Load - DC

Approach slab dead load reaction taken as 2 kips/foot of wall applied at the pavement seat.

B. Live Load - LL

Live load impact does not apply to the abutment. For bridge approach slab live load assumptions, see Section 10.6. If bridge approach slabs are not to be constructed in the project (e.g. bridge approach slab details are not included in the bridge sheets of the Plans) a live load surcharge (LS) applies.

C. Earth Pressure - EH, EV

Active earth pressure (EH) and the unit weight of backfill on the heel and toe (EV) will be provided in a geotechnical report. The toe fill shall be included in the analysis for overturning if it adds to overturning.

Passive earth pressure resistance (EH) in front of a footing may not be dependable due to potential for erosion, scour, or future excavation. Passive earth pressure may be considered for stability at the strength limit state only below the depth that is not likely to be disturbed over the structure's life. The Geotechnical Branch should be contacted to determine if passive resistance may be considered. The top two feet of passive earth pressure should be ignored.

D. Earthquake Load - EQ

Seismic superstructure loads shall be transmitted to the substructure through bearings, girder stops or restrainers. As an alternative, the superstructure may be rigidly attached to the substructure. The Extreme Event I load factor for all EQ induced loads shall be 1.0.

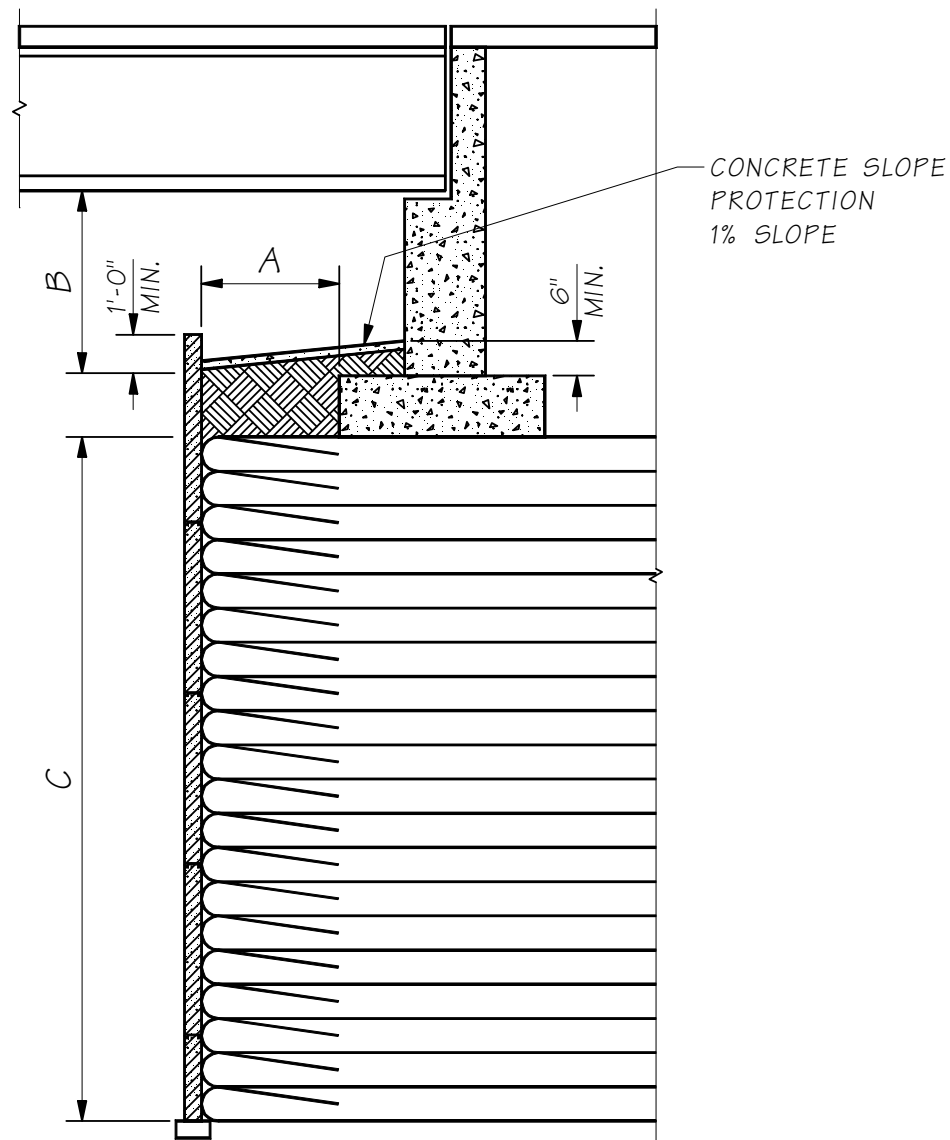
For bearing pressure and wall stability checks, the seismic inertial force of the abutment, P_{IR} , shall be combined with the seismic lateral earth pressure force, P_{AE} , as described in AASHTO LRFD Section 11.6.5.1.

For structural design of the abutment, the seismic inertial force, P_{IR} , shall be combined with the seismic lateral earth pressure force, P_{AE} , as described in AASHTO LRFD Section 11.6.5.1 for stability checks. The inertial force shall include the inertia of the concrete, but need not include the inertia of the soil above the heel.

E. Bearing Forces - TU

For strength design, the bearing shear forces shall be based on $\frac{1}{2}$ of the annual temperature range. This force is applied in the direction that causes the worst case loading.

For extreme event load cases, calculate the maximum friction force (when the bearing slips) and apply in the direction that causes the worst case loading.



- A. 4'-0" MIN. FOR SE WALLS (PRECAST CONCRETE PANEL FACE OR CAST-IN-PLACE CONCRETE FACE) AND 2'-0" MIN. FOR SPECIAL DESIGNED GEOSYNTHETIC RETAINING WALLS WITH WRAPPED FACE.
- B. 3'-0" MIN. FOR GIRDER BRIDGES AND 5'-0" MIN. FOR NON-GIRDER, SLAB, AND BOX GIRDER BRIDGES.
- C. 30'-0" MAXIMUM

Abutment on SE Wall or Geosynthetic Wall
Figure 7.5.2-1

7.5.4 Temporary Construction Load Cases

A. Superstructure Built after Backfill at Abutment

If the superstructure is to be built after the backfill is placed at the abutments, the resulting temporary loading would be the maximum horizontal force with the minimum vertical force. During the abutment design, a load case shall be considered to check the stability and sliding of abutments after placing backfill but prior to superstructure placement. This load case is intended as a check for a temporary construction stage, and not meant to be a controlling load case that would govern the final design of the abutment and footing. This loading will generally determine the tensile reinforcement in the top of the footing heel.

If this load case check is found to be satisfactory, a note shall be added to the general notes in the contract plans and the contractor will not be required to make a submittal requesting approval for early backfill placement. This load case shall include a 2'-0" deep soil surcharge for the backfill placement equipment (LS) as covered by the *Standard Specifications* Section 2-03.3(14)I.

B. Wing Wall Overturning

It is usually advantageous in sizing the footing to release the falsework from under the wing walls after some portion of the superstructure load is applied to the abutment. A note can cover this item, when applicable, in the sequence of construction on the plans.

7.5.5 Abutment Bearings and Girder Stops

All structures shall be provided with some means of restraint against lateral displacement at the abutments due to temperature, shrinkage, wind, earth pressure, and earthquake loads, etc. Such restraints may be in the form of concrete girder stops with vertical elastomeric pads, concrete hinges, or bearings restrained against movement.

All prestressed girder bridges in Western Washington (within and west of the Cascade mountain range) shall have girder stops between all girders at abutments and intermediate expansion piers. This policy is based on fact that the February 28, 2001 Nisqually earthquake caused significant damage to girder stops at bridges where girder stops were not provided between all girders. In cases where girder stops were cast prior to placement of girders and the 3" grout pads were placed after setting the girders, the 3" grout pads were severely damaged and displaced from their original position.

A. Abutment Bearings

Longitudinal forces from the superstructure are normally transferred to the abutments through the bearings. The calculated longitudinal movement shall be used to determine the shear force developed by the bearing pads. The shear modulus of Neoprene at 70°F (21°C) shall be used for determining the shear force. However, the force transmitted through a bearing pad shall be limited to that which causes the bearing pad to slip. Normally, the maximum percentage of the vertical load reaction transferred in shear is assumed to be 6 percent for PTFE sliding bearings and 20 percent for elastomeric bearing pads. For semi-integral abutments, the horizontal earth pressure acting on the end diaphragm is transferred through the bearings.

When the force transmitted through the bearing pads is very large, the designer should consider increasing the bearing pad thickness, using PTFE sliding bearings and/or utilizing the flexibility of the abutment as a means of reducing the horizontal design force. When the flexibility of the abutment is considered, it is intended that a simple approximation of the abutment deformation be made.

For semi-integral abutments with overhanging end diaphragms at the Extreme Event, the designer shall consider that longitudinal force may be transmitted through the end diaphragm. If the gap provided is less than the longitudinal displacement demand, assume the end diaphragm is in contact with abutment wall. In this case, the bearing force shall not be added to seismic earth pressure force.

B. Bearing Seats

The bearing seats shall be wide enough to accommodate the size of the bearings used with a minimum edge dimension of 3" and satisfy the requirements of AASHTO LRFD Section 4.7.4.4. On L abutments, the bearing seat shall be sloped away from the bearings to prevent ponding at the bearings. The superelevation and profile grade of the structure should be considered for drainage protection. Normally, a 1/4" drop across the width of the bearing seat is sufficient.

C. Transverse Girder Stops

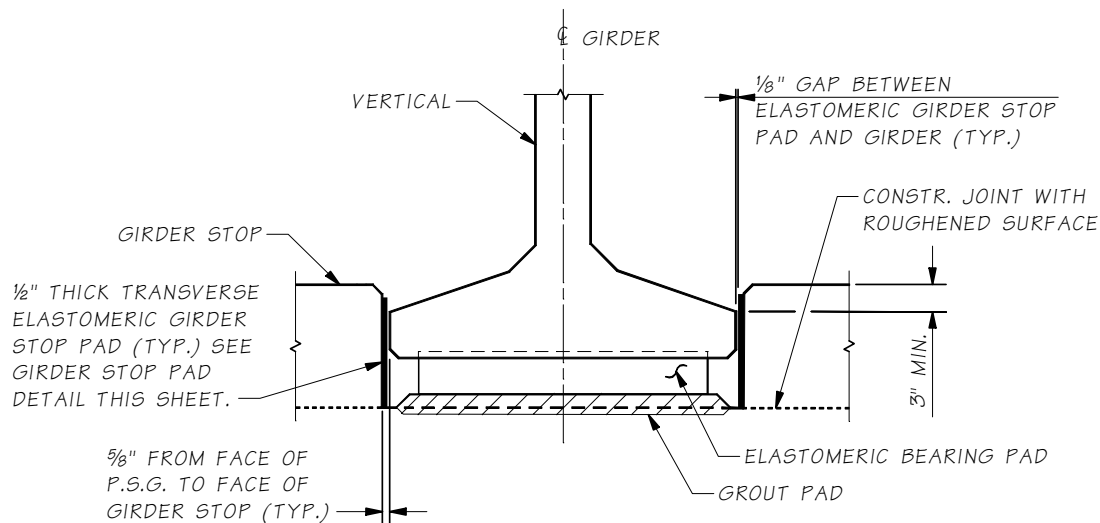
Transverse girder stops are required for all abutments in order to transfer lateral loads from the superstructure to the abutment. Abutments shall normally be considered as part of the Earthquake Resisting System (ERS). Girder stops shall be full width between girder flanges except to accommodate bearing replacement requirements as specified in Chapter 9. The girder stop shall be designed to resist loads at the Extreme Limit State for the earthquake loading, Strength loads (wind etc.) and any transverse earth pressure from skewed abutments, etc. Girder stops are designed using shear friction theory and the shear strength resistance factor shall be $\phi_s = 0.9$. The possibility of torsion combined with horizontal shear when the load does not pass through the centroid of the girder stop shall also be investigated.

In cases where the WSDOT Bridge Design Engineer permits use of ERS #3 described in Section 4.2.2, which includes a fusing mechanism between the superstructure and substructure, the following requirements shall be followed:

- The abutment shall not be included in the ERS system, the girder stops shall be designed to fuse, and the shear strength resistance factor shall be $\phi_s = 1.0$.
- If a girder stop fusing mechanism is used on a pile supported abutment, the combined overstrength resistance of the girder stops per AASHTO Seismic Section 4.14 shall be less than the combined plastic shear resistance of the piles.

The detail shown in [Figure 7.5.5-1](#) may be used for prestressed girder bridges. Prestressed girders shall be placed in their final position before girder stops are cast to eliminate alignment conflicts between the girders and girder stops. Elastomeric girder stop pads shall run the full length of the girder stop. All girder stops shall provide 1/8" clearance between the prestressed girder flange and the elastomeric girder stop pad.

For skewed bridges with semi-integral or end type A diaphragms, the designer shall evaluate the effects of earth pressure forces on the elastomeric girder stop pads. These pads transfer the skew component of the earth pressure to the abutment without restricting the movement of the superstructure in the direction parallel to centerline. The performance of elastomeric girder stop pads shall be investigated at Service Limit State. In some cases bearing assemblies containing sliding surfaces may be necessary to accommodate large superstructure movements.



GIRDER STOP DETAIL

NOTE:

1. GIRDER STOPS SHALL BE CONSTRUCTED AFTER PLACEMENT OF PRESTRESSED GIRDERS.
2. TRANSVERSE ELASTOMERIC GIRDER STOP PADS BETWEEN GIRDER AND GIRDER STOPS SHALL BE PLACED AFTER CONSTRUCTING THE GIRDER STOPS. THE PADS SHALL BE BONDED TO GIRDER STOPS WITH APPROVED ADHESIVE.

Girder Stop Details

Figure 7.5-1

7.5.6 Abutment Expansion Joints

The compressibility of abutment expansion joints shall be considered in the design of the abutment when temperature, shrinkage, and earthquake forces may increase the design load. For structures without abutment expansion joints, the earth pressure against the end diaphragm is transmitted through the superstructure.

7.5.7 Open Joint Details

Vertical expansion joints extending from the top of footings to the top of the abutment are usually required between abutments and adjacent retaining walls to handle anticipated movements. The expansion joint is normally filled with premolded joint filler which is not water tight. There may be circumstances when this joint must be water tight; $\frac{1}{8}$ butyl rubber may be used to cover the joint. The open joint in the barrier shall contain a compression seal to create a watertight joint. [Figure 7.5.7-1](#) shows typical details that may be used. Aesthetic considerations may require that vertical expansion joints between abutments and retaining walls be omitted. This is generally possible if the retaining wall is less than 60 feet long.

The footing beneath the joint may be monolithic or cast with a construction joint. In addition, dowel bars may be located across the footing joint parallel to the wall elements to guard against differential settlement or deflection.

On semi-integral abutments with overhanging end diaphragms, the open joints must be protected from the fill spilling through the joint. Normally butyl rubber is used to seal the openings. See the end diaphragm details in the Appendices of Chapter 5 for details.

7.5.8 Construction Joints

Construction joints should be provided between the footings/caps and stems of abutments. Shear keys shall be provided at construction joints between the footing and the stem, at vertical construction joints and at any construction joint that requires shear transfer. To simplify construction, vertical construction joints are often necessary, particularly between the abutment and adjacent wing walls. The *Standard Specifications* cover the size and placement of shear keys. The location of such joints shall be detailed on the plans. Construction joints with roughened surface can be used at locations with horizontal joints. These should be shown on the plans and labeled “Construction Joint With Roughened Surface.” When construction joints are shown in the Plans for the convenience of the Contractor and are not structurally required, they shall be indicated as optional. When construction joints are located within the face of the abutment wall, a pour strip or an architectural reveal should be used for a clean appearance. Details should be shown in the plans.

7.5.9 Abutment Wall Design

When the primary structural action is parallel to the superstructure or normal to the abutment face, the wall shall be treated as a column subjected to combined axial load and bending moment. Compressive reinforcement need not be included in the design of cantilever walls, but the possibility of bending moment in the direction of the span as well as towards the backfill shall be considered. A portion of the vertical bars may be cut off where they are no longer needed for stress.

A. General

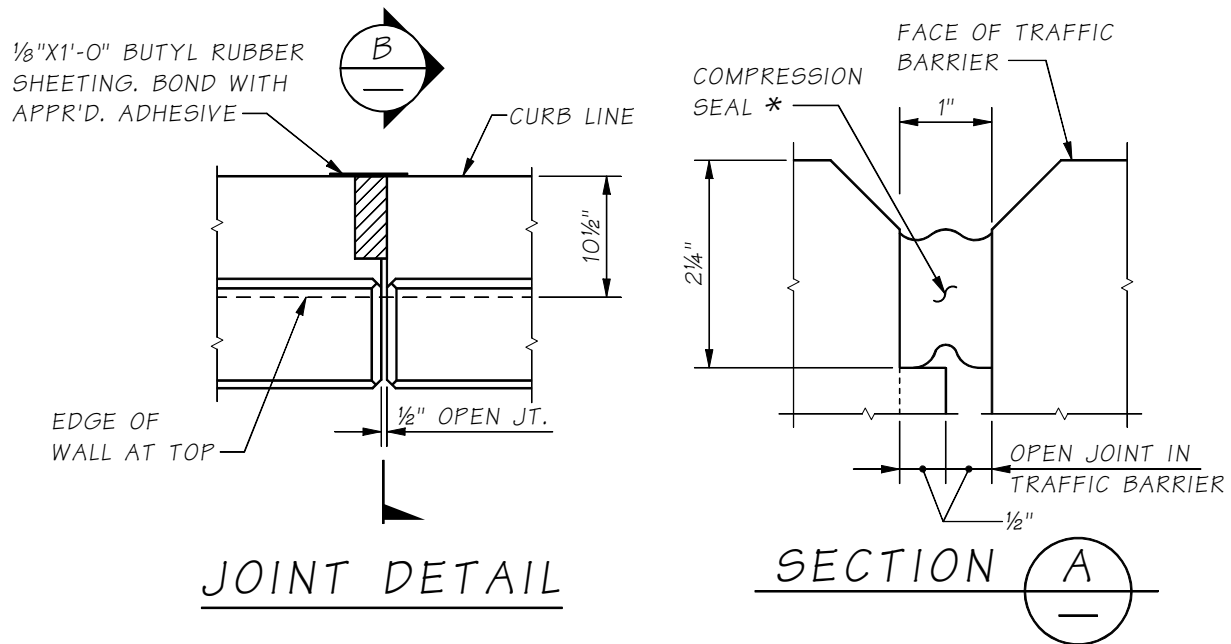
In general, horizontal reinforcement should be placed outside of vertical reinforcement to facilitate easier placement of reinforcement.

B. Temperature and Shrinkage Reinforcement

AASHTO LRFD Section 5.10.8 shall be followed for providing the minimum temperature and shrinkage steel near surfaces of concrete exposed to daily temperature changes and in structural mass concrete. On abutments that are longer than 60', consideration should be given to have vertical construction joints to minimize shrinkage cracks.

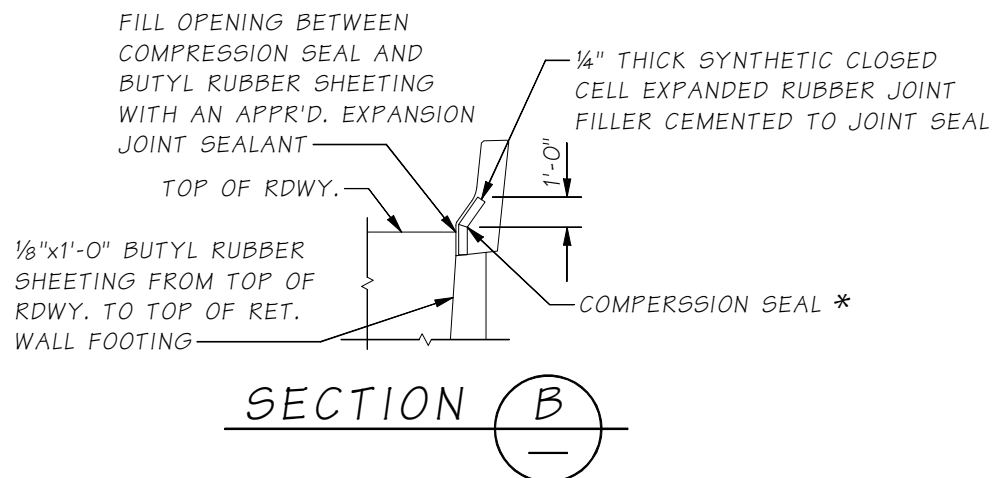
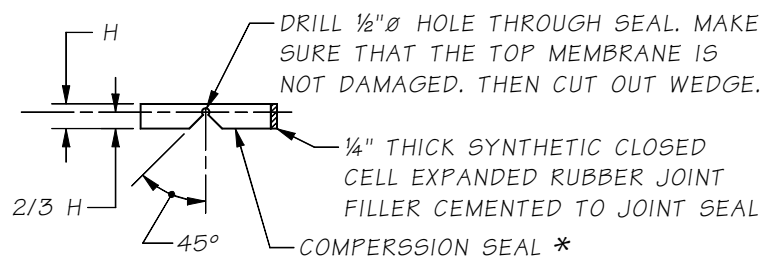
C. Cross Ties

The minimum cross tie reinforcement in abutment walls, except stub abutments, shall be #4 tie bars with 135° hooks, spaced at approximately 2'-0" maximum center-to-center vertically and horizontally, see [Figure 7.5.9-1](#).

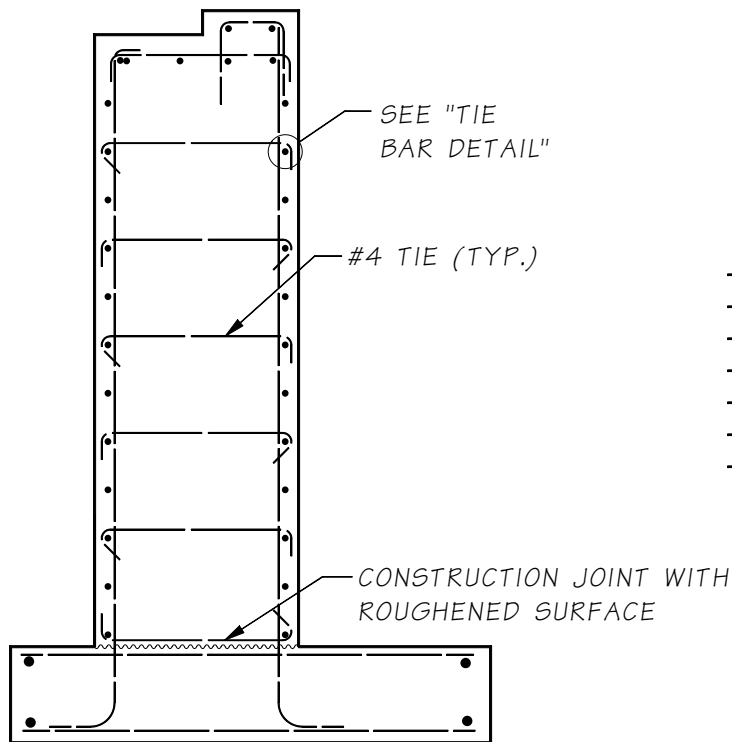


* COMPRESSION SEAL

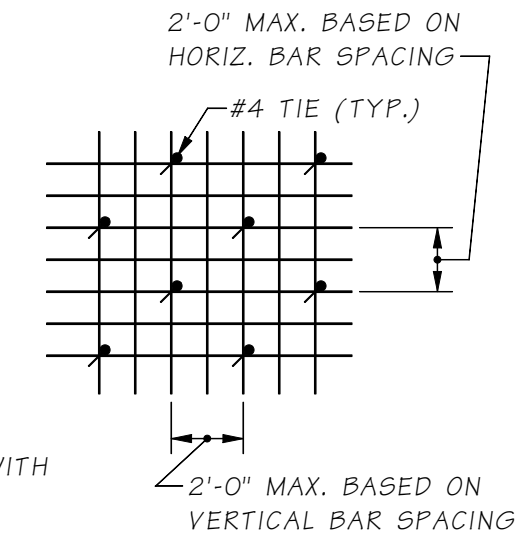
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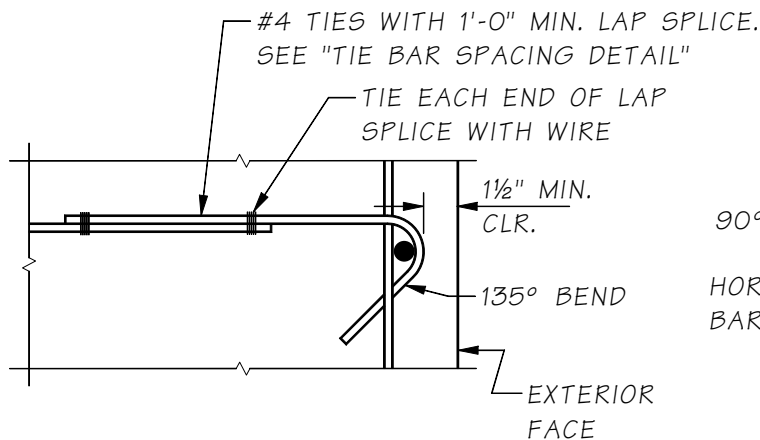
Open Joint Details Between Abutment and Retaining Walls
Figure 7.5.7-1



TYPICAL SECTION

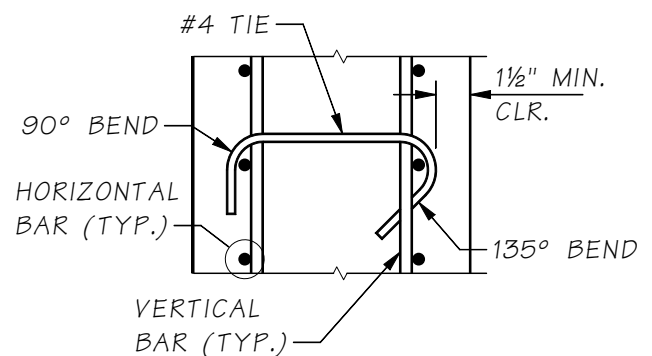


TIE BAR
SPACING DETAIL



ALTERNATE
TIE BAR DETAIL

CONSTANT OR VARIABLE WIDTH SECTION



TIE BAR DETAIL

CONSTANT WIDTH SECTION

Cross Tie Details
Figure 7.5.9-1

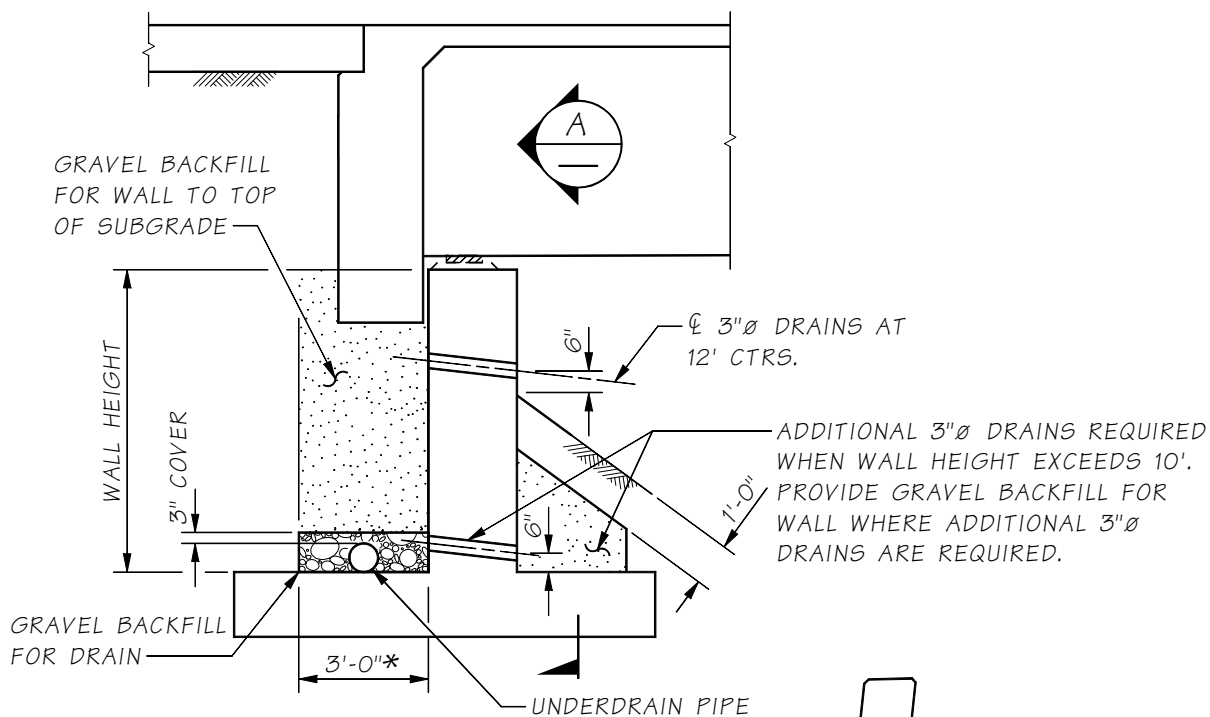
7.5.10 Drainage and Backfilling

3" diameter weep holes shall be provided in all bridge abutment walls. These shall be located 6" above the finish ground line at about 12' on center. In cases where the vertical distance between the top of the footing and the finish groundline is greater than 10', additional weep holes shall be provided 6" above the top of the footing. No weep holes are necessary in cantilever wing walls where a wall footing is not used.

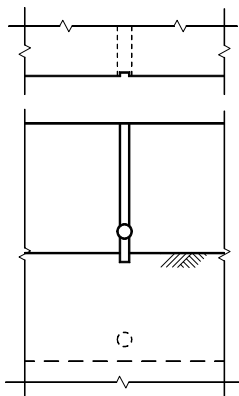
The details for gravel backfill for wall, underdrain pipe and backfill for drain shall be indicated on the plans. The gravel backfill for wall shall be provided behind all bridge abutments. The underdrain pipe and gravel backfill for drain shall be provided behind all bridge abutments except abutments on fills with a stem wall height of 5' or less. When retaining walls with footings are attached to the abutment, a breakout may be required for the underdrain pipe outfall. Cooperation between Bridge and Structures Office and the Design PE Office as to the drainage requirements is needed to guarantee proper breakout locations.

Underdrain pipe and gravel backfill for drain are not necessary behind cantilever wing walls. A 3' thickness of gravel backfill for wall behind the cantilever wing walls shall be shown in the plans.

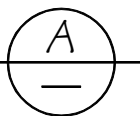
The backfill for wall, underdrain pipe and gravel backfill for drain are not included in bridge quantities, the size of the underdrain pipe should not be shown on the bridge plans, as this is a Design PE Office design item and is subject to change during the design phase. [Figure 7.5.10-1](#) illustrates backfill details.



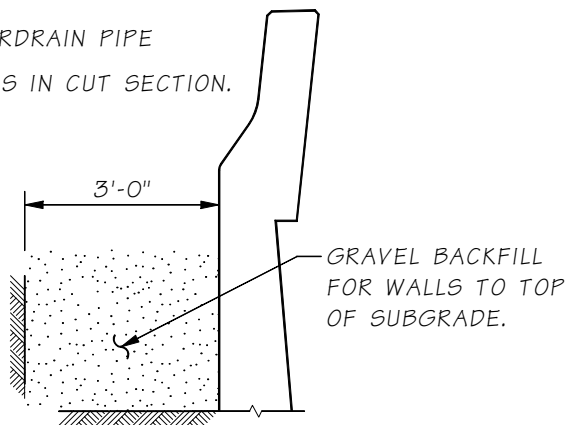
* CONSULT WITH SUPERVISOR FOR ABUTMENTS IN CUT SECTION.



SECTION



WHERE DRAINS ARE USED
WITH RUSTICATION STRIPS
DETAIL SO DRAIN ENDS
ON THE STRIP.



SECTION THROUGH WING WALL

GRAVEL BACKFILL FOR DRAIN, GRAVEL BACKFILL
FOR WALL, AND UNDERDRAIN PIPE NOT INCLUDED
IN BRIDGE QUANTITIES

Drainage and Backfill Details
Figure 7.5.10-1

7.5.11 Abutments Supported By Mechanically-Stabilized Earth Walls

Bridge abutments may be supported on mechanically-stabilize earth (MSE) walls, including geosynthetic retaining walls (with and without structural facing), structural earth walls and reinforced soil. Abutments supported on these walls shall be designed in accordance with the requirements of this manual and the following documents (listed in order of importance):

1. *Geotechnical Design Manual* Section 15.5.3.5.
2. AASHTO LRFD.
3. Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volume I and II, FHWA-NHI-10-024, FHWA-NHI-10-025.

Bridges with MSE supported abutments shall be designed as one of two types described below, and shall satisfy the associated design requirements.

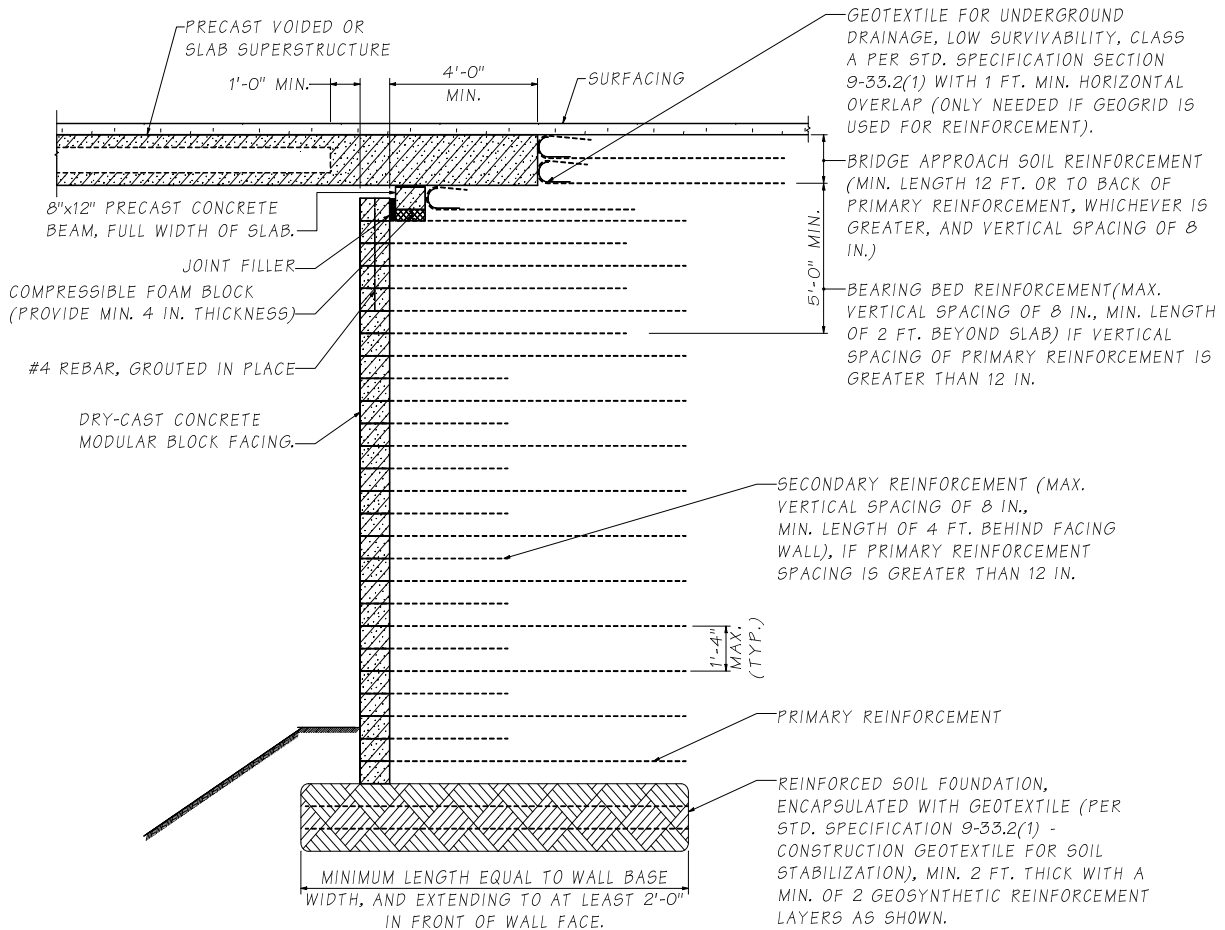
A. Single-span bridges with precast slab superstructures supported directly on reinforced soil

These bridges shall conform to the following requirements:

1. Walls supporting abutments shall be special designed wall systems, and shall be one of two types:
 - a. Geosynthetic walls with a stacked dry-cast modular concrete block facing. The top 3 rows of dry-cast modular concrete blocks shall be grouted with #4 rebar. See [Figure 7.5.11-1](#)
 - b. Geosynthetic and structural earth walls with full-height concrete facing. See [Figure 7.5.11-2](#)
2. The span length shall not exceed 60 feet.
3. The superstructure shall include a 5" thick C.I.P. composite topping.
4. The end of the precast superstructure shall be at least 4 feet from the back face of the MSE wall. Minimum seat width requirements shall be provided on the reinforced soil bearing area.
5. A foam board detail shall be used to create a 1 foot horizontal buffer between the bearing area and the wall facing.
6. The vertical gap between the top of wall facing and the bottom of superstructure shall be 4" or 2 percent of the abutment height, whichever is greater.
7. Prestressing strands in the zone bearing on reinforced soil shall have a minimum concrete cover of 2". Transverse reinforcing steel within this zone shall have a minimum concrete cover of 1½". All prestressing strand shall be removed to a 2" depth from the end of the slab. The voids shall be patched with epoxy grout.
8. Where voided slab superstructures are used, the slab section shall be solid from the end of the slab to at least 1 foot in front of the fascia.

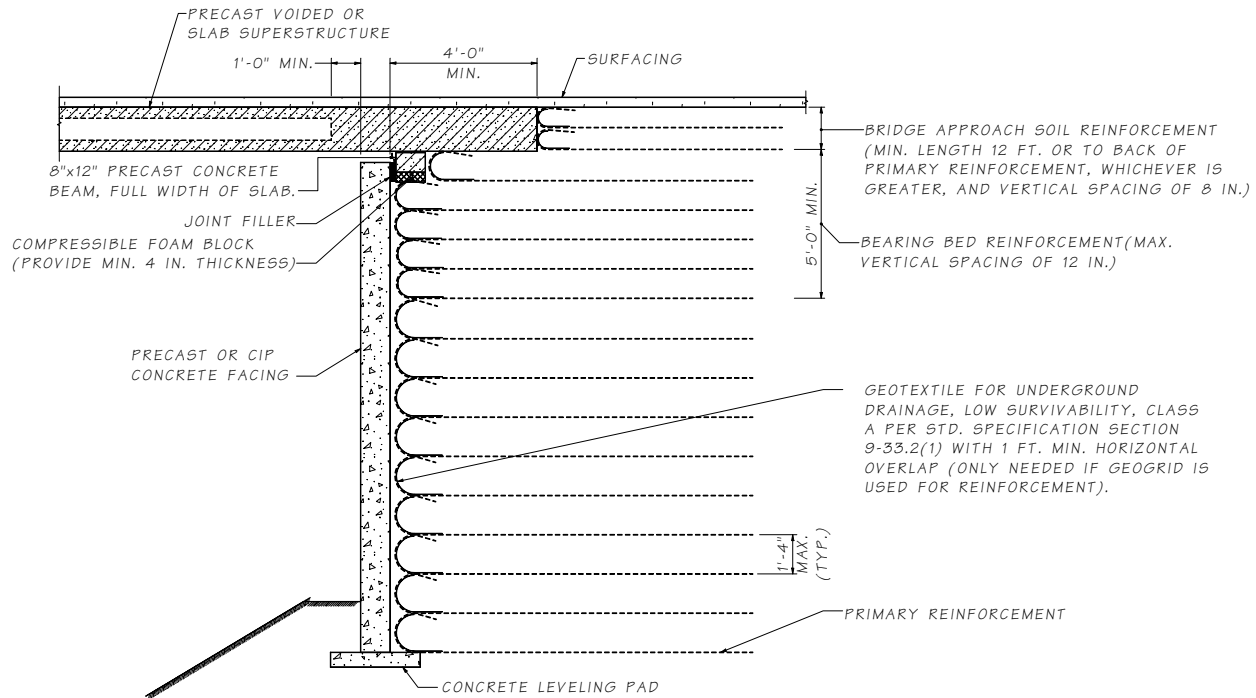
9. The abutment shall be designed for a bearing pressure at service loads not to exceed 2.0 TSF and a factored load at strength and extreme limit states not to exceed 3.5 TSF. The bearing pressure may be increased to 3.0 TSF at service loads and 4.5 TSF at strength and extreme limit states if a vertical settlement monitoring program is conducted in accordance with WSDOT GDM Section 15.5.3.5.

10. Bridge approach slabs may be omitted.



Reinforced Soil Abutment with Dry-Cast Modular Block Facing

Figure 7.5.11-1



Reinforced Soil Abutment with Full-Height Concrete Facing

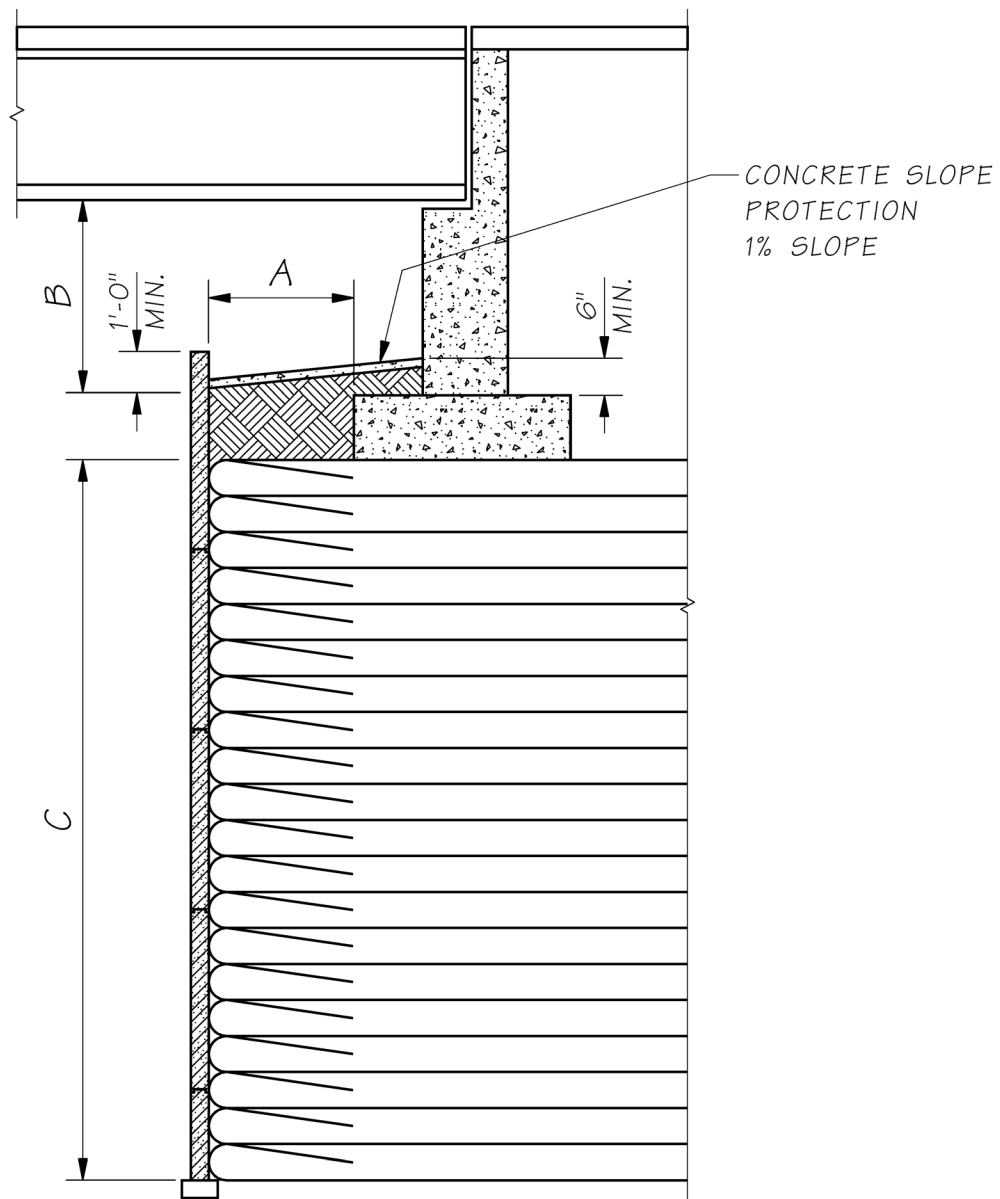
Figure 7.5.11-2

B. Bridges with spread footing abutments supported by a geosynthetic wall or SE wall

These bridges shall conform to the following requirements, see [Figure 7.5.11-3](#):

1. Walls shall be 30 feet or less in total height, which includes the retained soil height up to the bottom of the embedded spread footing.
2. For SE walls, the front edge of the bridge footing shall be placed 4 feet minimum from the back face of the fascia panel. For geosynthetic retaining walls with a wrapped face, the front edge of the bridge footing shall be placed 2 feet minimum from the back face of the fascia panel.
3. The abutment footing shall be covered by at least 6 inch of soil for frost protection.
4. The superstructure of continuous span bridges shall be designed for differential settlement between piers.
5. Abutment spread footings shall be designed for bearing pressure at service loads not to exceed 2.0 TSF and factored load at strength and extreme limit states not to exceed 3.5 TSF. The bearing pressure may be increased to 3.0 TSF at service loads and 4.5 TSF at strength and extreme limit states if a vertical settlement monitoring program is conducted in accordance with the [Geotechnical Design Manual](#) Section 15.5.3.5.
6. Walls supporting bridge abutments shall be special designed wall systems. Walls supporting permanent bridges shall have precast or C.I.P concrete fascia panels. Walls supporting temporary bridges may use dry-cast modular concrete block or welded wire facing.
7. Concrete slope protection shall be provided. Fall protection shall be provided in accordance with *Design Manual* Chapter 730.

Deviations from the design requirements require approval from the State Bridge Design Engineer and the State Geotechnical Engineer.



- A. 4'-0" MIN. FOR SE WALLS (PRECAST CONCRETE PANEL FACE OR CAST-IN-PLACE CONCRETE FACE) AND 2'-0" MIN. FOR SPECIAL DESIGNED GEOSYNTHETIC RETAINING WALLS WITH WRAPPED FACE.
- B. 3'-0" MIN. FOR GIRDER BRIDGES AND 5'-0" MIN. FOR NON-GIRDER, SLAB, AND BOX GIRDER BRIDGES.
- C. 30'-0" MAXIMUM

Spread Footing on SE Wall or Geosynthetic Wall
Figure 7.5.11-3

7.6 Abutment Wing Walls and Curtain Walls

Particular attention should be given to the horizontal reinforcing steel required at fixed corners between abutment and wing/curtain walls. Since wall deflections are zero near the abutment, curtain walls and cantilever wing walls shall assume an at-rest soil pressure. This increased loading can normally be reduced to an Active soil pressure at a distance (from the corner), equal to the average height of the wall under design. At this distance, the wall deflections are assumed large enough to allow the active state soil pressures to be developed. See *Geotechnical Design Manual* Section 15.4.2.7, “Active, Passive, and At-Rest Pressures.”

7.6.1 Traffic Barrier Loads

Traffic barriers shall be rigidly attached to a bridge approach slab that is cantilevered over the top of a wing/curtain wall or Structural Earth wall. The barrier collision load is applied directly to the bridge approach slab. The yield line theory as specified in AASHTO LRFD Appendix A13.3 is primarily for traffic barrier on bridge deck slabs and may not be applicable to traffic barrier on less rigid supports, such as retaining walls.

7.6.2 Wing Wall Design

The following wing wall design items should be addressed in the Plans.

- A. For strength design of wing walls, vertical loads and moments may be distributed over 10' of the abutment wall and footing.
- B. Footing thickness shall be not less than 1'-6".
- C. Exterior girder top flanges should be located (at the least) inside the curb line at the end pier.
- D. For skewed bridges, modify the details on the traffic barrier and approach slab sheet so the expansion joint detailing agree. List appropriate manufacturers and model numbers for the expansion joint system. Generally, a 1" expansion joint with a 1" open joint in the barrier is shown in the plans, unless the bridge expansion joint design dictates otherwise.

7.6.3 Wing Wall Detailing

All wing wall reinforcement should be a vertical grid and not follow a tapered bottom of wall. This allows for the steel to be placed in two layers that fits better with abutment reinforcing.

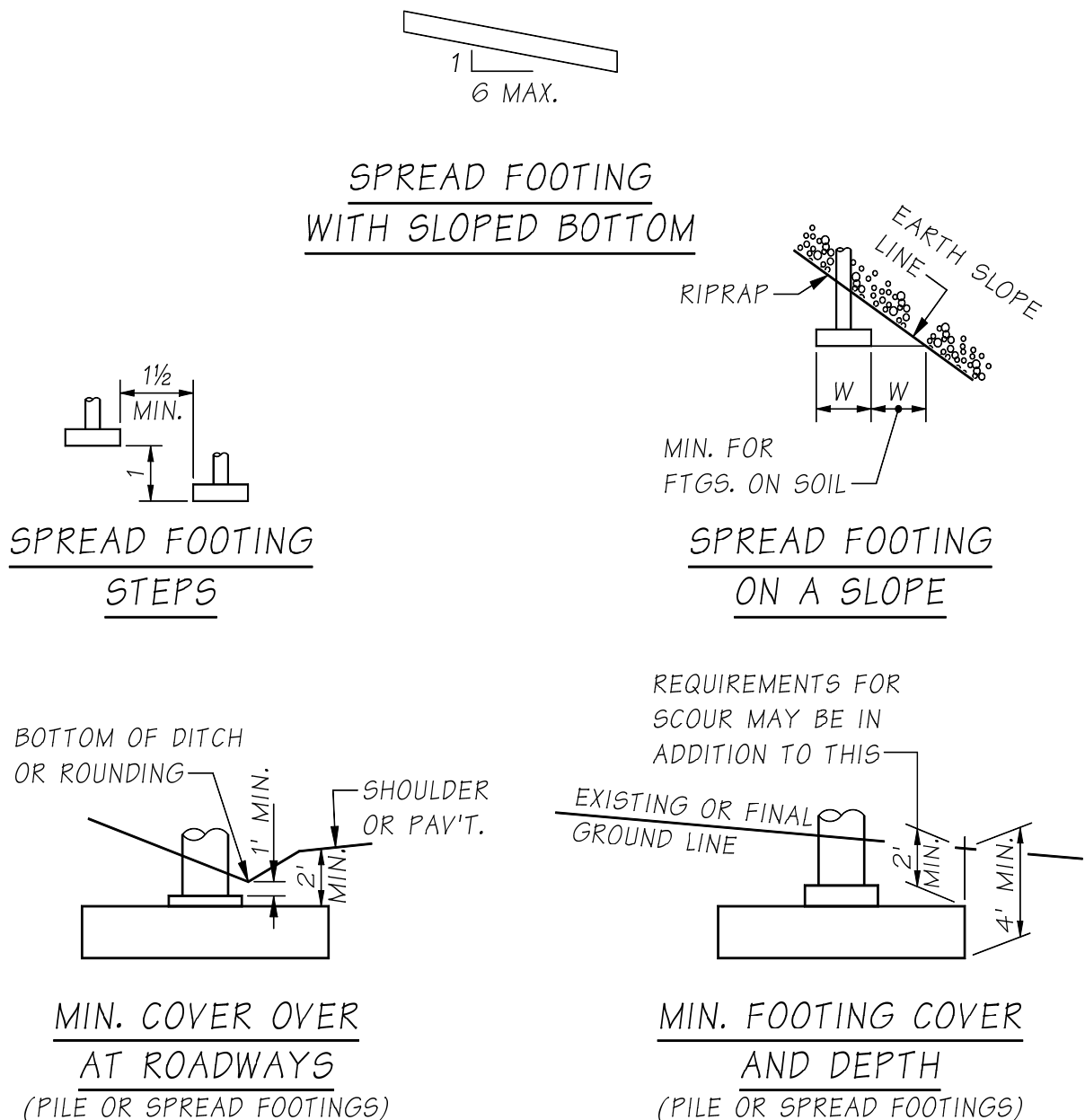
7.7 Footing Design

7.7.1 General Footing Criteria

The provisions given in this section pertain to both spread footings and pile supported footings.

A. Minimum Cover and Footing Depth

The geotechnical report may specify a minimum footing depth in order to ensure adequate bearing pressure. Stream crossings may require additional cover depth as protection against scour. The HQ Hydraulic Section shall be consulted on this matter. The end slope on the bridge approach fill is usually set at the preliminary plan stage but affects the depth of footings placed in the fill. Figure 7.7.1-1 illustrates footing criteria when setting footing elevations.

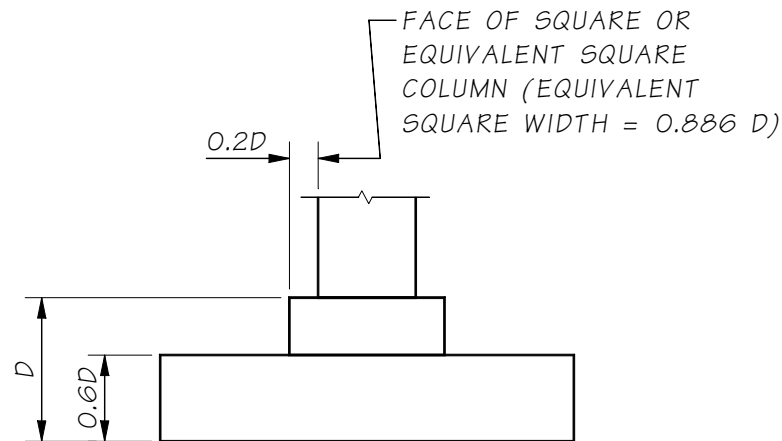


Guidelines for Footing Cover and Depth

Figure 7.7.1-1

B. Pedestals

A pedestal is sometimes used as an extension of the footing in order to provide additional depth for shear near the column. Its purpose is to provide adequate structural depth while saving concrete. For proportions of pedestals, see Figure 7.7.1-2. Since additional forming is required to construct pedestals, careful thought must be given to the tradeoff between the cost of the extra forming involved and the cost of additional footing concrete. Also, additional foundation depth may be needed for footing cover. Whenever a pedestal is used, the plans shall note that a construction joint will be permitted between the pedestal and the footing. This construction joint should be indicated as a construction joint with roughened surface.



Pedestal Dimensions
Figure 7.7.1-2

7.7.2 Loads and Load Factors

The following Table 7.7.2-1 is a general application of minimum and maximum load factors as they apply to a generic footing design. Footing design must select the maximum or minimum Load Factors for various modes of failure for the Strength and Extreme Event Limit States.

The dead load includes the load due to structural components and non-structural attachments (DC), and the dead load of wearing surfaces and utilities (DW). The live load (LL) does not include vehicular dynamic load allowance (IM).

Designers are to note, if column design uses magnified moments, then footing design must use magnified column moments.

Sliding and Overturning, e_o	Bearing Stress (e_c, s_v)
$LL_{min} = 0$	LL_{max}
DC_{min}, DW_{min} for resisting forces, DC_{max}, DW_{max} for causing forces,	DC_{max}, DW_{max} for causing forces DC_{min}, DW_{min} for resisting forces
EV_{min}	EV_{max}
EH_{max}	EH_{max}
LS	LS

Load Factors
Table 7.7.2-1

7.7.3 Geotechnical Report Summary

The Geotechnical Branch will evaluate overall bridge site stability. Slope stability normally applies to steep embankments at the abutment. If stability is in question, a maximum service limit state load will be specified in the report. Bridge design will determine the maximum total service load applied to the embankment. The total load must be less than the load specified in the geotechnical report.

Based on the foundations required in the Preliminary Plan and structural information available at this stage, the Report provides the following Geotechnical Engineering results. For all design limit states, the total factored footing load must be less than the factored resistance.

A. Bearing Resistance - Service, Strength, and Extreme Event Limit States

The nominal bearing resistance (q_n) may be increased or reduced based on previous experience for the given soils. The geotechnical report will contain the following information:

- Nominal bearing resistance (q_n) for anticipated effective footing widths, which is the same for the strength and extreme event limit states.
- Service bearing resistance (q_{ser}) and amount of assumed settlement.
- Resistance factors for strength and extreme event limit states (ϕ_b).
- Embedment depth requirements or footing elevations to obtain the recommended q_n .

Spread footings supported on SE walls or geosynthetic walls shall be designed with nominal bearing resistances not to exceed 6.0 ksf at service limit states and 9.0 ksf at strength and extreme event limit states. A vertical settlement monitoring program shall be conducted where nominal bearing resistance exceeds 4.0 ksf at service limit states or 7.0 ksf at strength or extreme event limit states. See GDM Section 15.5.3.5 for additional requirements.

B. Sliding Resistance - Strength and Extreme Event Limit States

The geotechnical report will contain the following information to determine earth loads and the factored sliding resistance ($R_R = \phi R_n$)

- Resistance factors for strength and extreme event limit states (ϕ_r, ϕ_{ep})
- If passive earth pressure (R_{ep}) is reliably mobilized on a footing: ϕ_f or S_u and σ'_v , and the depth of soil in front of footing that may be considered to provide passive resistance.

C. Foundation Springs - Extreme Event Limit States

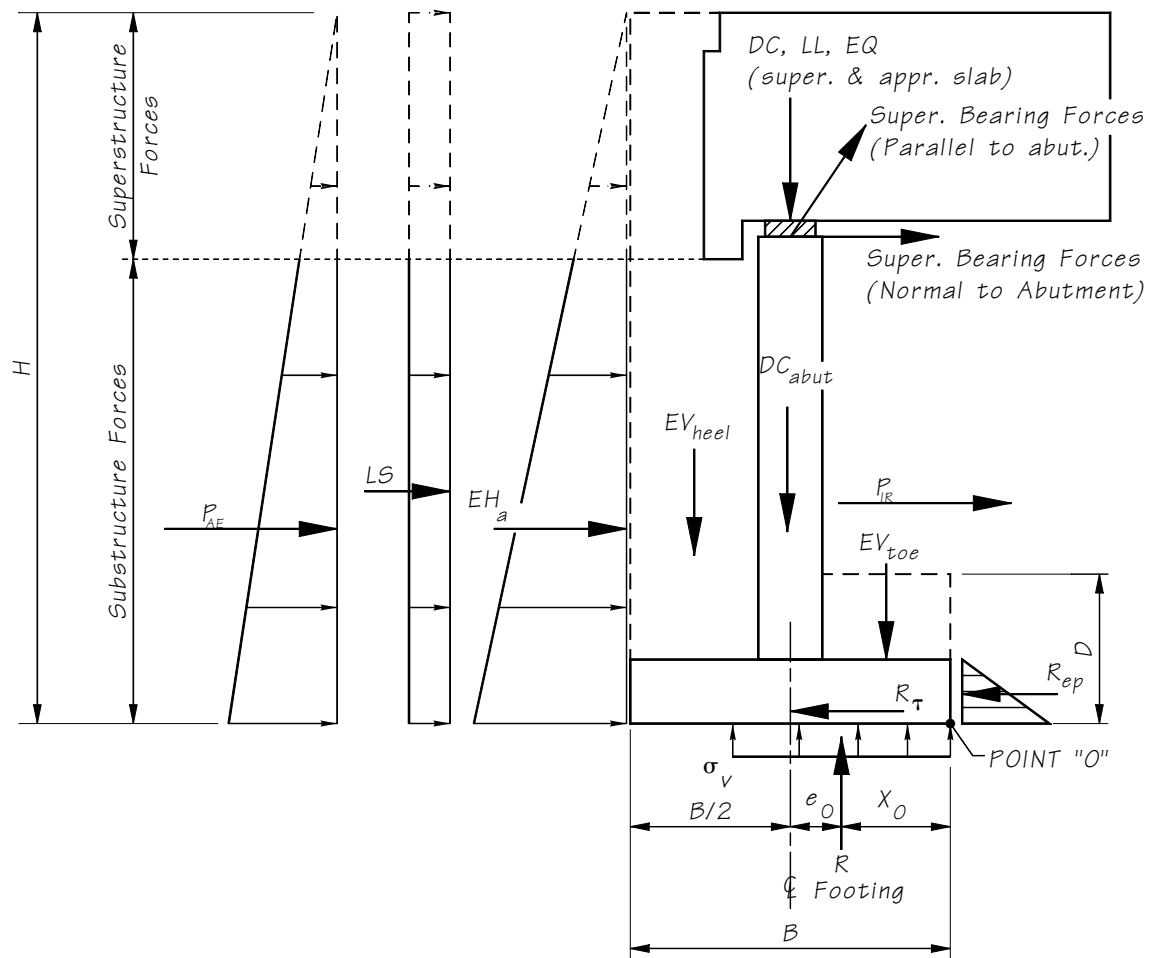
When a structural evaluation of soil response is required for a bridge analysis, the Geotechnical Branch will determine foundation soil/rock shear modulus and Poisson's ratio (G and μ). These values will typically be determined for shear strain levels of 2 to 0.2 percent, which are typical strain levels for large magnitude earthquakes.

7.7.4 Spread Footing Design

The following section is oriented toward abutment spread footing design. Spread footing designs for intermediate piers or other applications use the same concepts with the appropriate structural analysis. Structural designers should complete all design checks before consulting with the Geotechnical Engineer about any design problem. There may be several problem criteria that should be addressed in the solution.

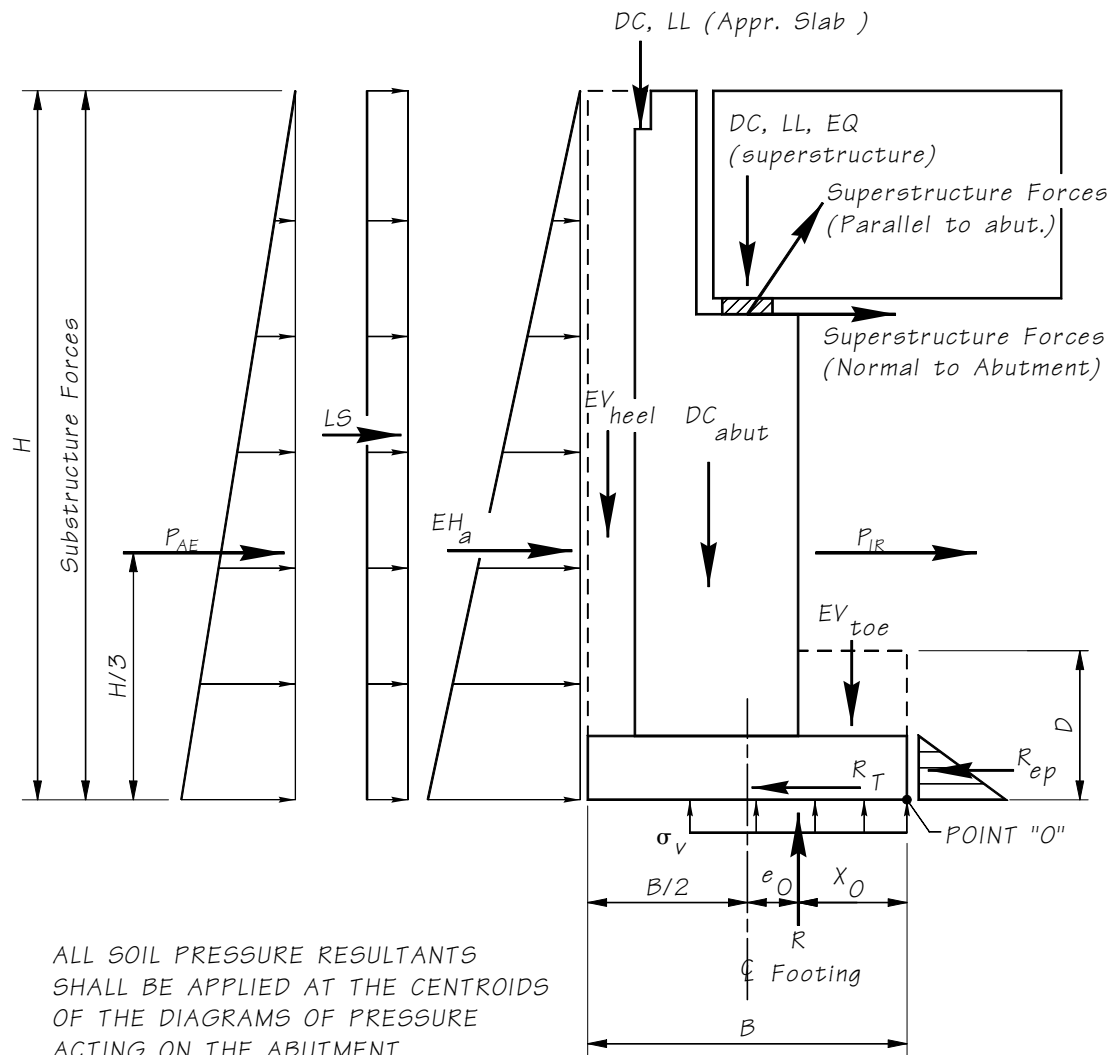
A. Abutment Spread Footing Force Diagram

Figures 7.7.4-1 and 7.7.4-2 diagram the forces that act on abutment footings. Each limit state design check will require calculation of a reaction (R) and the location (X_o) or eccentricity (e_o). The ultimate soil passive resistance (Q_{ep}) at the toe is determined by the Geotechnical Engineer and is project specific.



Cantilever (End Diaphragm) Abutment Force Diagram

Figure 7.7.4-1



L-Abutment Force Diagram
Figure 7.7.4-2

B. Bearing Stress

For geotechnical and structural footings design, the bearing stress calculation assumes a uniform bearing pressure distribution. For footing designs on rock, the bearing stress is based on a triangular or trapezoidal bearing pressure distribution. The procedure to calculate bearing stress is summarized in the following outline. See Abutment Spread Footing Force Diagrams for typical loads and eccentricity.

Step 1: Calculate the Resultant force (R_{str}), location ($X_{O_{str}}$) and eccentricity for Strength (e_{str}).

$$X_{O_{str}} = (\text{factored moments about the footing base}) / (\text{factored vertical loads})$$

Step 2A: For Footings on Soil:

Calculate the maximum soil stress (σ_{str}) based on a uniform pressure distribution. Note that this calculation method applies in both directions for biaxially loaded footings. See AASHTO LRFD Section 10.6.3.1.5 for guidance on biaxial loading. The maximum footing pressure on soil with a uniform distribution is:

$$\sigma_{str} = R/B' = R/2X_o = R/(B-2e), \text{ where } B' \text{ is the effective footing width.}$$

Step 2B: For Footings on Rock:

If the reaction is outside the middle $\frac{1}{3}$ of the base, use a triangular distribution.

$$\sigma_{str} \text{ max} = 2R/3 X_o, \text{ where "R" is the factored limit state reaction.}$$

If the reaction is within the middle $\frac{1}{3}$ of the base, use a trapezoidal distribution.

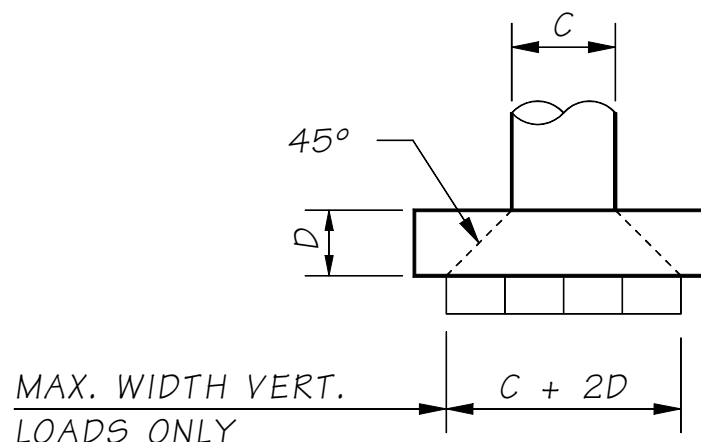
$$\sigma_{str} \text{ max} = R/B (1 + 6 e/B)$$

In addition, WSDOT limits the maximum stress (P/A) applied to rock due to vertical loads only. This is because the rock stiffness approaches infinity relative to the footing concrete. The maximum width of uniform stress is limited to $C+2D$ as shown in Figure 7.7.4-3.

Step 3: Compare the factored bearing stress (σ_{str}) to the factored bearing resistance ($\phi b_c q_n$) of the soil or rock. The factored bearing stress must be less than or equal to the factored bearing resistance.

$$\sigma_{str} \leq \phi b_c q_n$$

Step 4: Repeat steps 1 thru 3 for the Extreme Event limit state. Calculate $X_{o_{ext}}$, e_{ext} and σ_{ext} using Extreme Event factors and compare the factored stress to the factored bearing ($\phi_b q_n$).



Footings on Rock
Figure 7.7.4-3

C. Failure By Sliding

The factored sliding resistance (Q_R) is comprised of a frictional component ($\phi_T Q_T$) and the Geotechnical Branch may allow a passive earth pressure component ($\phi_{ep} Q_{ep}$). The designer shall calculate Q_R based on the soil properties specified in the geotechnical report. The frictional component acts along the base of the footing, and the passive component acts on the vertical face of a buried footing element. The factored sliding resistance shall be greater than or equal to the factored horizontal applied loads.

$$Q_R = \phi_T Q_T + \phi_{ep} Q_{ep}$$

The Strength Limit State ϕ_T and ϕ_{ep} are provided in the geotechnical report or AASHTO LRFD Section 10.5.5.2.2-1. The Extreme Event Limit State Q_T and Q_{ep} are generally equal to 1.0.

Where:

$$Q_T = (R) \tan \delta$$

$\tan \delta$ = Coefficient of friction between the footing base and the soil

$\tan \delta = \tan \phi$ for cast-in-place concrete against soil

$\tan \delta = (0.8)\tan \phi$ for precast concrete

R = Vertical force – Minimum Strength and Extreme Event factors are used to calculate R

ϕ = angle of internal friction for soil

D. Overturning Stability

Calculate the locations of the overturning reaction (R) for strength and extreme event limit states. Minimum load factors are applied to forces and moments resisting overturning. Maximum load factors are applied to forces and moments causing overturning. Note that for footings subjected to biaxial loading, the following eccentricity requirements apply in both directions.

See AASHTO LRFD Sections 11.6.3.3 (Strength Limit State) and 11.6.5 (Extreme Event Limit State) for the appropriate requirements for the location of the overturning reaction (R).

E. Footing Settlement

The service limit state bearing resistance (q_{ser}) will be a settlement-limited value, typically 1".

$$\text{Bearing Stress} = \sigma_{ser} < \phi q_{ser} = \text{Factored nominal bearing}$$

Where, q_{ser} is the unfactored service limit state bearing resistance and ϕ is the service resistance factor. In general, the resistance factor (ϕ) shall be equal to 1.0.

For immediate settlement (not time dependent), both permanent dead load and live load should be considered for sizing footings for the service limit state. For long-term settlement (on clays), only the permanent dead loads should be considered.

If the structural analysis yields a bearing stress (σ_{ser}) greater than the bearing resistance, then the footing must be re-evaluated. The first step would be to increase the footing size to meet bearing resistance. If this leads to a solution, recheck layout criteria and inform the Geotechnical Engineer the footing size

has increased. If the footing size cannot be increased, consult the Geotechnical Engineer for other solutions.

F. Concrete Design

Footing design shall be in accordance with AASHTO LRFD Section 5.13.3 for footings and the general concrete design of AASHTO LRFD Chapter 5. The following Figure 7.7.4-4 illustrates the modes of failure checked in the footing concrete design.

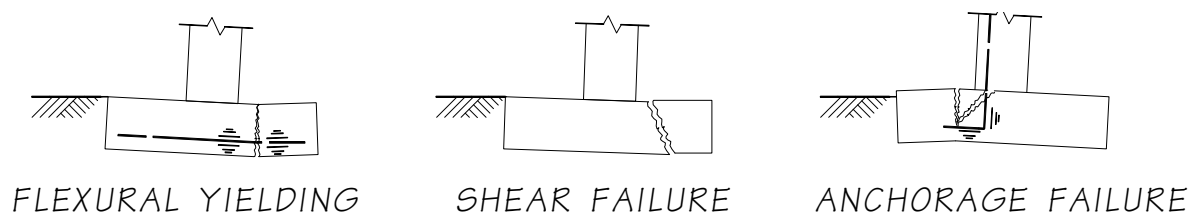


Figure 7.7.4-4

1. Footing Thickness and Shear

The minimum footing thickness shall be 1'-0". The minimum plan dimension shall be 4'-0". Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements (with or without reinforcement). If concrete shear governs the thickness, it is the engineer's judgment, based on economics, as to whether to use a thick footing unreinforced for shear or a thinner footing with shear reinforcement. Generally, shear reinforcement should be avoided but not at excessive cost in concrete, excavation, and shoring requirements. Where stirrups are required, place the first stirrup at $d/2$ from the face of the column or pedestal. For large footings, consider discontinuing the stirrups at the point where $v_u = v_c$.

2. Footing Force Distribution

The maximum internal forces in the footing shall be determined based on a triangular or trapezoidal bearing pressure distribution, see AASHTO LRFD Section 10.6.5.

3. Vertical Reinforcement (Column or Wall)

Vertical reinforcement shall be developed into the footing to adequately transfer loads to the footing. Vertical rebar shall be bent 90° and extend to the top of the bottom mat of footing reinforcement. This facilitates placement and minimizes footing thickness. Bars in tension shall be developed using $1.25 L_d$. Bars in compression shall develop a length of $1.25 L_d$, prior to the bend. Where bars are not fully stressed, lengths may be reduced in proportion, but shall not be less than $\frac{3}{4} L_d$.

The concrete strength used to compute development length of the bar in the footing shall be the strength of the concrete in the footing. The concrete strength to be used to compute the section strength at the interface between footing and a column concrete shall be that of the column concrete. This is allowed because of the confinement effect of the wider footing.

4. Bottom Reinforcement

Concrete design shall be in accordance with AASHTO LRFD. Reinforcement shall not be less than #6 bars at 12" centers to account for uneven soil conditions and shrinkage stresses.

5. Top Reinforcement

Top reinforcement shall be used in any case where tension forces in the top of the footing are developed. Where columns and bearing walls are connected to the superstructure, sufficient reinforcement shall be provided in the tops of footings to carry the weight of the footing and overburden assuming zero pressure under the footing. This is the uplift earthquake condition described under "Superstructure Loads." This assumes that the strength of the connection to the superstructure will carry such load. Where the connection to the superstructure will not support the weight of the substructure and overburden, the strength of the connection may be used as the limiting value for determining top reinforcement. For these conditions, the AASHTO LRFD requirement for minimum percentage of reinforcement will be waived. Regardless of whether or not the columns and bearing walls are connected to the superstructure, a mat of reinforcement shall normally be provided at the tops of footings. On short stub abutment walls (4' from girder seat to top of footing), these bars may be omitted. In this case, any tension at the top of the footing, due to the weight of the small overburden, must be taken by the concrete in tension.

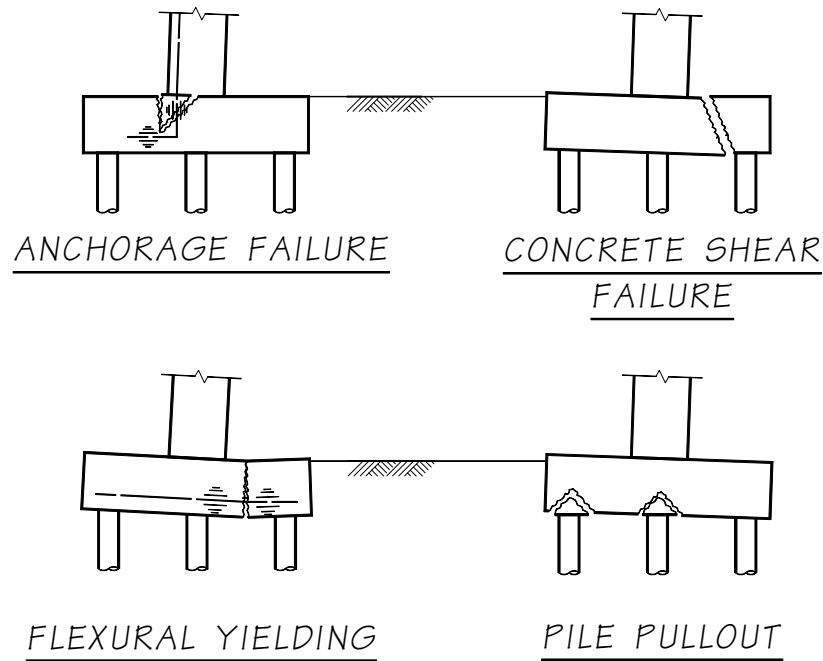
Top reinforcement for column or wall footings designed for two-way action shall not be less than #6 bars at 12" centers, in each direction while top reinforcement for bearing wall footings designed for one-way action shall not be less than #5 bars at 12" centers in each direction.

6. Shrinkage and Temperature Reinforcement

For footings greater than 3 feet thick, temperature and shrinkage reinforcing shall be provided on the side faces in accordance with AASHTO LRFD Section 5.10.8.

7.7.5 Pile-Supported Footing Design

The minimum footing thickness shall be 2'-0". The minimum plan dimension shall be 4'-0". Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements. The use of strut and tie modeling is recommended for the design of all pile caps and pile footings. [Figure 7.7.5-1](#) identifies the modes of failure that should be investigated for general pile cap/footing design.

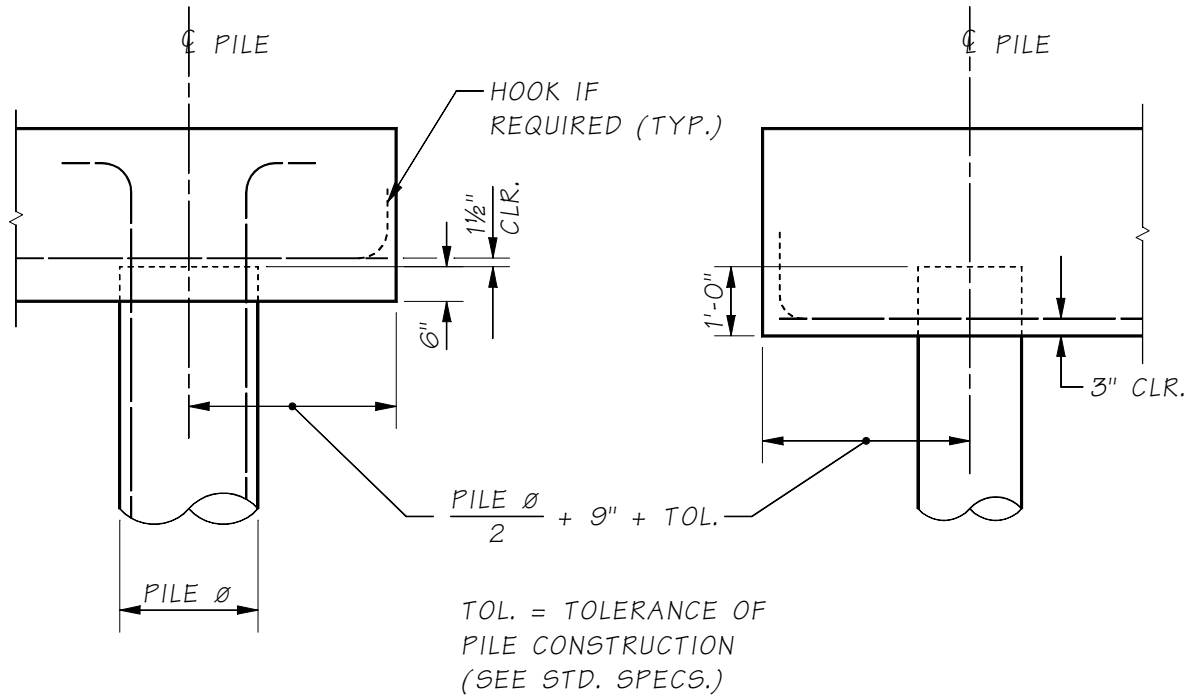


Pile Footing Modes of Failure

Figure 7.7.5-1

A. Pile Embedment, Clearance, and Rebar Mat Location

All piles shall have an embedment in the concrete sufficient to resist moment, shear, and axial loads. The steel casing for cast-in-place concrete piles with reinforcing extending into footings shall be embedded a minimum of 6". The clearance for the bottom mat of footing reinforcement shall be 1½" between the reinforcing and the top of the casing for CIP pile footings. See Figure 7.7.5-2 for the minimum pile clearance to the edge of footing.



C.I.P. PILE FOOTING

STEEL H-PILE FOOTING

Pile Embedment and Reinforcing Placement

Figure 7.7.5-2

B. Concrete Design

In determining the proportion of pile load to be used for calculation of shear stress on the footing, any pile with its center 6" or more outside the critical section shall be taken as fully acting on that section. Any pile with its center 6" or more inside the critical section shall be taken as not acting for that section. For locations in between, the pile load acting shall be proportioned between these two extremes. The critical section shall be taken as the effective shear depth (d_v) as defined in AASHTO LRFD Section 5.8.2.9. The distance from the column/wall face to the allowable construction centerline of pile (design location plus or minus the tolerance) shall be used to determine the design moment of the footing. The strut and tie design method should be used where appropriate.

7.8 Shafts

7.8.1 Axial Resistance

The factored axial resistance of the shaft (R) is generally composed of two parts: the nominal end bearing (R_p) and the nominal skin friction (R_s). The general formula is as follows, where ϕ is the limit state resistance factor.

$$R = \phi_p R_p + \phi_s R_s \quad (7.8.1-1)$$

The total factored shaft loading must be less than the factored axial resistance. R_p and R_s are treated as independent quantities although research has shown that the end bearing and skin friction resistance have some interdependence. R_p and R_s shown as a function of depth will be stated in the geotechnical report for the bridge. End bearing resistance, R_p , is typically provided by the Geotechnical Branch as a net value. Thus, the effective weight of the shaft can be reduced by the total weight of the excavated soil when examining compressive loads and resistances.

The designer shall consider all applicable factored load combination limit states and shaft resistances when determining shaft axial resistance and demand and shaft tip elevations. For some shaft designs, liquefiable soils, scour conditions and/or downdrag forces may need to be considered. Determining which limit states to include these conditions or forces can be complex. The Hydraulics Branch and the Geotechnical Engineer shall be consulted to ensure overly and/or under conservative load combinations and resistances are not being considered. Open and frequent communication is essential during design.

Although the AASHTO LRFD include water loads, WA , in Extreme Event I limit states, in most cases the loss of soil resistance due to scour conditions is not combined with Extreme Event I load combinations. The probability of a design earthquake occurring in the presence of the maximum scour event is low. However, in some instances it is appropriate to include some scour effects. When scour is included with Extreme Event I load combinations, the skin resistance of the soil, up to a maximum of 25 percent of the scour depth for the design flood (100 year event), shall be deducted from the resistance of the shaft. The loss of skin resistance for the full scour depth for the design flood shall be considered when checking axial resistance of the shaft for all strength and service limit states. The loss of skin resistance for the full scour depth of the check flood (500 year event) shall be considered when checking the axial resistance of the shaft for Extreme Event II limit states. It should be noted that scour does not produce a load effect on the structure but changes the geometry of the bridge pier and available soil resistance so that effects of other loads are amplified. The engineer may also need to consider scour effects on piers that are currently outside of the ordinary high water zones due to potential migration of rivers or streams during flood events. The Hydraulics Branch will provide guidance for these rare cases.

Downdrag forces may also need to be considered in some designs. Downdrag forces are most often caused by the placement of fill adjacent to shafts, which causes consolidation and settlement of underlying soils. This situation is applicable to service and strength limit states. Downdrag forces can also be caused by liquefaction-induced settlement caused by a seismic event. Pore water pressure builds up in liquefiable soils during ground shaking. And as pore water pressure dissipates, the soil layer(s) may settle, causing downdrag forces on the shaft to develop. These liquefaction induced

downdrag forces are only considered in the Extreme Event I limit state. However, downdrag induced by consolidation settlement is never combined with downdrag forces induced by liquefaction, but are only considered separately in their applicable limit states.

The downdrag is treated as a load applied to the shaft foundations. The settling soil, whether it is caused by consolidation under soil stresses (caused, for example, by the placement of fill), or caused by liquefaction, creates a downward acting shear force on the foundations. This shear force is essentially the skin friction acting on the shaft, but reversed in direction by the settlement. This means that the skin friction along the length of the shaft within the zone of soil that is contributing to downdrag is no longer available for resisting downward axial forces and must not be included with the soil resistance available to resist the total downward axial (i.e., compression) loads acting on the foundation.

In general, the Geotechnical Engineer will provide shaft soil resistance plots as a function of depth that includes skin friction along the full length of the shaft. Therefore, when using those plots to estimate the shaft foundation depth required to resist the axial compressive foundation loads, this “skin friction lost” due to downdrag must be subtracted from the resistance indicated in the geotechnical shaft resistance plots, and the downdrag load per shaft must be added to the other axial compression loads acting on the shaft.

Similarly, if scour is an issue that must be considered in the design of the foundation, with regard to axial resistance (both in compression and in uplift), the skin friction lost due to removal of the soil within the scour depth must be subtracted from the shaft axial resistance plots provided by the Geotechnical Engineer. If there is any doubt as to whether or not this skin friction lost must be subtracted from the shaft resistance plots, it is important to contact the Geotechnical Engineer for clarification on this issue. Note that if both scour and downdrag forces must be considered, it is likely that the downdrag forces will be reduced by the scour. This needs to be considered when considering combination of these two conditions, and assistance from the Geotechnical Engineer should be obtained.

The *Geotechnical Design Manual* Chapters 6, 8, and 23, should be consulted for additional explanation regarding these issues.

Following is a summary of potential load combination limit states that shall be checked if scour effects, liquefiable soils and/or downdrag forces are included in the design. The geotechnical report will provide the appropriate resistance factors to use with each limit state.

A. Embankment Consolidation Downdrag

Embankment downdrag from fill or the presence of compressible material below the foundations; no liquefaction.

Checks:

1. Include embankment induced downdrag loads with all Strength and Service Limit States. Do not include with Extreme Limit States. Use maximum load factor unless checking an uplift case, where the minimum shall be used. Subtract the skin friction lost within the downdrag zone from the shaft axial resistance plots provided by the Geotechnical Engineer.

B. Seismic Liquefaction Downdrag

Liquefiable soils with post-earthquake downdrag forces. No embankment downdrag.

If embankment downdrag is present, it shall not be included with liquefaction-induced downdrag therefore it would not be included in Check 3 below.

Checks:

1. Extreme Event I Limit State

Use static soil resistances (no loss of resistance due to liquefaction) and no downdrag forces. Use a live load factor of 0.5.

2. Extreme Event I Limit State

Use reduced soil resistance due to liquefaction and no downdrag forces. Use a live load factor of 0.5. The soils in the liquefied zone will not provide the static skin friction resistance but will in most cases have a reduced resistance that will be provided by the Geotechnical Engineer.

3. Extreme Event I Limit State

Post liquefaction. Include downdrag forces, a live load factor of 0.5 and a reduced post-liquefaction soil resistance provided by the Geotechnical Engineer. Do not include seismic inertia forces from the structure since it is a post-earthquake check. There will be no skin resistance in the post-earthquake liquefied zone. Therefore, subtract the skin friction lost within the downdrag zone from the shaft axial resistance plots provided by the Geotechnical Engineer.

C. Scour

Scour from design flood (100 year events) and check floods (500 year events.) The shaft shall be designed so that shaft penetration below the scour of the applicable flood event provides enough axial resistance to satisfy demands. Since in general the Geotechnical Engineer will provide shaft resistance plots that include the skin friction within the scour zone, the skin friction lost will need to be subtracted from the axial resistance plots provided to determine the shaft resistance acting below the scour depth. A special case would include scour with Extreme Event I limit states without liquefiable soils and downdrag. It is overly conservative to include liquefied soil induced downdrag and scour with the Extreme Event I limit states. The Hydraulics Branch and the Geotechnical Engineer will need to be consulted for this special case.

Checks:

1. Service and Strength Limit States

Subtract the skin friction lost within the scour depth (i.e., 100 percent of the scour depth for the 100 year design flood) from the shaft axial resistance plots provided by the Geotechnical Engineer, to estimate the shaft depth required to resist all service and strength limit demands.

2. Extreme Event II Limit State

Subtract the skin friction lost within the scour depth (i.e., 100 percent of the scour depth for the 500 year check flood event) from the shaft axial resistance plots provided by the Geotechnical Engineer, to estimate the shaft depth required to resist all Extreme Event II limit demands. Use a live load factor of 0.5. Do not include ice load, *IC*, vessel collision force, *CV*, and vehicular collision force, *CT*.

3. Extreme Event II Limit State

Subtract the skin friction lost within the scour depth (in this case only 50 percent of the scour depth for the 500 year check flood event) from the shaft axial resistance plots provided by the Geotechnical Engineer, to estimate the shaft depth required to resist all Extreme Event II limit demands. Use a live load factor of 0.5. In this case, include ice load, *IC*, vessel collision force, *CV*, and vehicular collision force, *CT*.

4. Extreme Event I Limit State (special case - no liquefaction)

Subtract the skin friction lost within the scour depth (i.e., in this case 25 percent of the scour depth for the 100 year design flood) from the shaft axial resistance plots provided by the Geotechnical Engineer, to estimate the shaft depth required to resist the Extreme Event I limit state demands.

D. Axial Resistance Group Reduction Factors

The group reduction factors for axial resistance of shafts for the strength and extreme event limit states shall be taken as shown in Table 7.8.1-1 unless otherwise specified by the Geotechnical Engineer. These reduction factors presume that good shaft installation practices are used to minimize or eliminate the relaxation of the soil between shafts and caving. If this cannot be adequately controlled due to difficult soils conditions or for other constructability reasons, lower group reduction factors shall be used as recommended by the Geotechnical Engineer of record. Alternatively, steps could be required during and/or after shaft construction to restore the soil to its original condition. The Geotechnical Engineer will provide these recommendations, which could include but is not limited to, pressure grouting of the tip, grouting along side of the shaft or full length casing.

Soil Type	Shaft Group Configuration	Shaft Center-to-Center Spacing	Special Conditions	Group Reduction factor, η
Cohesionless (Sands, gravels, etc.)	Single row	2D		0.90
		2.5D		0.95
		3D or more		1.0
	Multiple row	2.5D*		0.67
		3D		0.80
		4D or more		1.0
	Single and multiple rows	2D or more	Shaft group cap in intimate contact with ground consisting of medium-dense or denser soil	1.0
Cohesive (Clays, clayey sands, and glacially overridden, well-graded soils such as glacial till)	Single and multiple rows	2D or more	Full depth casing is used and augering ahead of the casing is not allowed, or pressure grouting is used along the shaft sides to restore lateral stress losses caused by shaft installation, and the shaft tip is pressure grouted	1.0
	Single or multiple rows	2D or more		1.0

*Minimum spacing for multiple row configurations.

Group Reduction Factors for Axial Resistance of Shafts

Table 7.8.1-1

These group reduction factors apply to both strength and extreme event limit states. For the service limit state the influence of the group on settlement as shall be determined from the AASHTO LRFD and the *Geotechnical Design Manual* M 46-03.

7.8.2 Structural Design and Detailing

Standard Specifications Section 6-19 should be reviewed as part of the design of shafts. The structural design of shafts is similar to column design. The following guidelines shall be followed:

- A. For shaft foundation supporting columns in any SDC, Shafts shall be designed to resist the 1.25 times the plastic overstrength of the column above. Where elastic design methods are approved in SDC C or D, the shaft may be designed to resist 1.2 times the elastic seismic forces at the demand displacement.
- B. Concrete Class 5000P shall be specified for the entire length of the shaft for wet or dry conditions of placement.
- C. When shafts are constructed in water, the concrete specified for the casing shoring seal shall be Class 4000W.

- D. The assumed concrete compressive strength **may be taken as** f'_c for structural design of shafts. For seismic design, the expected compressive strength may be taken as 1.3 times this reduced value in accordance with AASHTO Seismic Section 8.4.4.
- E. The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, and etc.), but the structural resistance of permanent steel casing shall not be considered for structural design of shafts unless the design conforms to [Section 7.10](#).
- F. Cover requirements vary depending on the shaft diameter and shall be as specified below:
- Diameter less than or equal to 3'-0" = 3"
 - Diameter greater than 3'-0" and less than 5'-0" = 4"
 - Diameter greater than or equal to 5'-0" = 6"

Standard Specifications Section 6-19 lists exceptions to these cover requirements when permanent slip casings are used in column splice zones.

- G. In general, shaft reinforcing shall be detailed to minimize congestion, facilitate concrete placement by tremie, and maximize consolidation of concrete.
- H. The clear spacing between spirals and hoops shall not be less than 6" or more than 9", with the following exception. The clear spacing between spirals or hoops may be reduced in the splice zone in single column/single shaft connections because shaft concrete may be vibrated in this area, negating the need for larger openings to facilitate good flow of concrete through the reinforcing cage.
- I. The volumetric ratio and spacing requirements of the AASHTO Seismic Specifications for confinement need not be met. The top of shafts in typical WSDOT single column/single shaft connections remains elastic under seismic loads due to the larger shaft diameter (as compared to the column). Therefore this requirement does not need to be met.
- J. Shaft transverse reinforcement may be constructed as hoops or spirals. Spiral reinforcement is preferred for shaft transverse reinforcement. However, if #6 spirals at 6" (excluding the exception in 7.8.2.H) clear do not satisfy the shear design, circular hoops may be used. Circular hoops in shafts up to #9 bars may be lap spliced using a welded lap detail. Note: Welded lap splices for spirals are currently acceptable under the AWS D1.4 up to bar size #6. Recent testing has been performed by WSDOT for bar sizes #7 through #9. All tests achieved full tensile capacity (including 125 percent of yield strength.) Therefore, #7 through #9 welded lap spliced hoops are acceptable to use provided they are not located in possible plastic hinge regions. Circular hoops may also be fabricated using a manual direct butt weld, resistance butt weld, or mechanical coupler. Weld splicing of hoops for shafts shall be completed prior to assembly of the shaft steel reinforcing cage. Refer to Section 7.4.5 for additional discussion on circular hoops. Mechanical couplers may be considered provided cover and clearance requirements are accounted for in the shaft details. When welded hoops or mechanical couplers are used, the plans shall show a staggered pattern around the perimeter of the shaft so that no two adjacent welded splices or couplers are located at the same location.

- K. In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation, which comes from the TRAC Report titled, “*Noncontact Lap Splices in Bridge Column-Shaft Connections*”:

$$S_{max} = \frac{2\pi A_{sh} f_{ytr} l_s}{k A_l f_{ul}} \quad (7.8.2-1)$$

Where:

- S_{max} = Spacing of transverse shaft reinforcement
- A_{sh} = Area of shaft spiral or transverse reinforcement bar
- f_{ytr} = Yield strength of shaft transverse reinforcement
- l_s = Standard splice length of the column reinforcement, per AASHTO LRFD.
- A_l = Total area of longitudinal column reinforcement
- f_{ul} = Specified minimum tensile strength of column longitudinal reinforcement (ksi), 90 ksi for A615 and 80 ksi for A706
- k = Factor representing the ratio of column tensile reinforcement to total column reinforcement at the nominal resistance. In the upper half of the splice zone, $k = 1.0$. In the lower half of the splice zone, this ratio could be determined from the column moment-curvature analysis using computer programs XTRACT or CSiBridge. To simplify this process, $k = 0.5$ could safely be used in most applications

The additional lateral reinforcement in the upper half of the oversized pile shafts is required to control cracking in this region. The volumetric ratio of transverse reinforcement throughout the splice zone shall not be less than that provided by a #6 spiral with a 6" pitch.

- L. Longitudinal reinforcement shall be provided for the full length of shafts. The minimum longitudinal reinforcement in the splice zone of single column/single shaft connections shall be the larger of 0.75 percent A_g of the shaft or 1.0 percent A_g of the attached column. The minimum longitudinal reinforcement beyond the splice zone shall be 0.75 percent A_g of the shaft. The minimum longitudinal reinforcement in shafts without single column/single shaft connections shall be 0.75 percent A_g of the shaft.
- M. The clear spacing between longitudinal reinforcement shall not be less than 6" or more than 9". If a shaft design is unable to meet this minimum requirement, a larger diameter shaft shall be considered. Alternatively, Grade 80 reinforcing steel could be considered. Where 3-bar bundles are used, the plans shall allow the contractor to construct the cage with two of the three bars located towards the center of the shaft. This allows the contractor flexibility in constructing the cage, but it reduces the flexural resistance of the shaft.
- N. Longitudinal reinforcing in shafts should be straight with no hooks to facilitate concrete placement and removal of casing. If hooks are necessary to develop moment at the top of a shaft (in a shaft cap situation) the hooks should be turned toward the center of the shaft while leaving enough opening to allow concrete placement with a tremie.

- O. Locations of longitudinal splices shall be shown in the contract plans. Mechanical splices shall be placed in low stress regions and staggered 2'-0" minimum.
- P. Use of two concentric circular rebar cages shall be avoided.
- Q. Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD. Resistance factors for Extreme Event Limit States shall be per the latest AASHTO *Seismic Specifications*. The resistance factor for shear shall conform to the AASHTO LRFD.
- R. The axial load along the shaft varies due to the side friction. It is considered conservative, however, to design the shaft for the full axial load plus the maximum moment. The entire shaft normally is then reinforced for this axial load and moment.
- S. Access tubes for Crosshole Sonic Log (CSL) testing **or thermal wires for Thermal Integrity Profiling (TIP)** shall be provided in all shafts. One tube **or wire** shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, and shown in the plans. The number of access tubes **or wires** for shaft diameters specified as "X feet 6 inches" shall be rounded up to the next higher whole number. The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement and three inches clear of the vertical reinforcement, at a uniform spacing measured along the circle passing through the centers of the access tubes. If the vertical reinforcement is not bundled and each bar is not more than one inch in diameter, the access tubes shall be placed two inches clear of the vertical reinforcement. If these minimums cannot be met due to close spacing of the vertical reinforcement, then access tubes shall be bundled with the vertical reinforcement. **The thermal wires shall be placed around the shaft, inside the spiral or hoop reinforcement and shall be tied to the vertical reinforcement.**
- T. Shafts shall be specified in English dimensions and shall be specified in sizes that do not preclude any drilling method. Shafts shall be specified in whole foot increments except as allowed here. The tolerances in *Standard Specifications* Section 6-19 accommodate metric casing sizes and/or oversized English casing sizes. Oversized English casings are often used so that tooling for drilling the shafts, which are the nominal English diameter, will fit inside the casing. There are a few exceptions, which will be discussed below. See [Table 7.8.2-1](#) for casing sizes and tolerances.

Column A	Column B	Column C	Column D	Column E	Column F	Column G	Column H
Nominal (Outside) English Casing Diameter		*Maximum Increase in Casing Inside Diameter	*Maximum Decrease in Casing Inside Diameter	Maximum English Casing Diameter	Nominal (Outside) Metric Casing Diameter		
Feet	Inches	Inches	Inches	Inches	Meters	Feet	Inches
12.0	144	6	0	150	3.73	12.24	146.85
11.0	132	6	0	138	3.43	11.25	135.0
10.0	120	6	2	126	3.00	9.84 [#]	118.11
9.5	114	6	0	120	3.00	9.84	118.11
9.0	108	6	0	114	2.80	9.19	110.23
8.0	96	6	0	102	2.50	8.20	98.42
7.0	84	6	0	90	2.20	7.22	86.61
6.5	78	6	0	84	2.00	6.56	78.74
6.0	72	6.75 ^{##}	0	78	2.00	6.56	78.74
5.5	66	6	0	72			
5.0	60	12	1	72	1.5	4.92 [#]	59.05
4.5	54	12	0	66	1.50	4.92	59.05
4.0 **	48	12	0	60	1.5	4.92	59.05
4.0**	48	12	1	60	1.2	3.94 [#]	47.28
3.0	36	12	0	48	1.00	3.28	39.37
3.0	36	12	0	48	0.915	3.00	36.02
2.5	30	12	0	42			
2.0	24	12	0	36	0.70	2.30	27.56

*Check [Standard Specifications](#) Section 6-19.

**Construction tolerances would allow either 1.2 or 1.5 meter casing to be used.

Designer shall check that undersize shaft meets the design demands.

Exception to typical construction tolerance of 6".

Table 7.8.2-1

As seen in [Table 7.8.2-1](#), construction tolerances shown in Column "C" allow shaft diameters to be increased up to 12" for shafts 5'-0" diameter or less and increased up to 6" for shafts greater than 5'-0" in diameter. In most cases these construction tolerances allow either metric or English casings to be used for installation of the shafts.

There are a few exceptions to these typical tolerances. These exceptions are as follows:

1. 4.0' Diameter Shafts

The tolerances in Columns "C" and "D" of [Table 7.8.2-1](#) allow either an oversized 4.92' diameter shaft or an undersized 3.94' shaft to be constructed. The reinforcement cage shall be sized to provide a minimum of 3" of cover to the undersized diameter.

2. 5.0' Diameter Shafts

The tolerances in Columns "C" and "D" of [Table 7.8.2-1](#) allow either an oversized 6.0' diameter shaft or an undersized 4.92' diameter shaft to be constructed. The reinforcement cage shall be sized to provide a minimum of 4" of cover to the undersized diameter.

3. 6.0' Diameter Shafts

Maximum oversize tolerance of $6\frac{3}{4}$ " is allowed.

4. 10.0' Diameter Shafts

The tolerances in Columns "C" and "D" of [Table 7.8.2-1](#) allow either an oversized 10.5' diameter shaft or an undersized 9.84' diameter shaft to be constructed. The reinforcement cage shall be sized to provide a minimum of 4" of cover to the undersized diameter.

For all shaft diameters, the designer should bracket the design so that all possible shaft diameters, when considering the construction tolerances, will satisfy the design demands. The minimum shaft diameter (nominal or undersized) shall be used for design of the flexural and shear reinforcement.

The nominal English shaft diameter shall be specified on the plans. When requesting shaft resistance charts from the Geotechnical Engineer, the designer should request charts for the nominal English shaft diameter.

- U. Shafts supporting a single column shall be sized to allow for construction tolerances, as illustrated in [Figure 7.8.2-2](#).

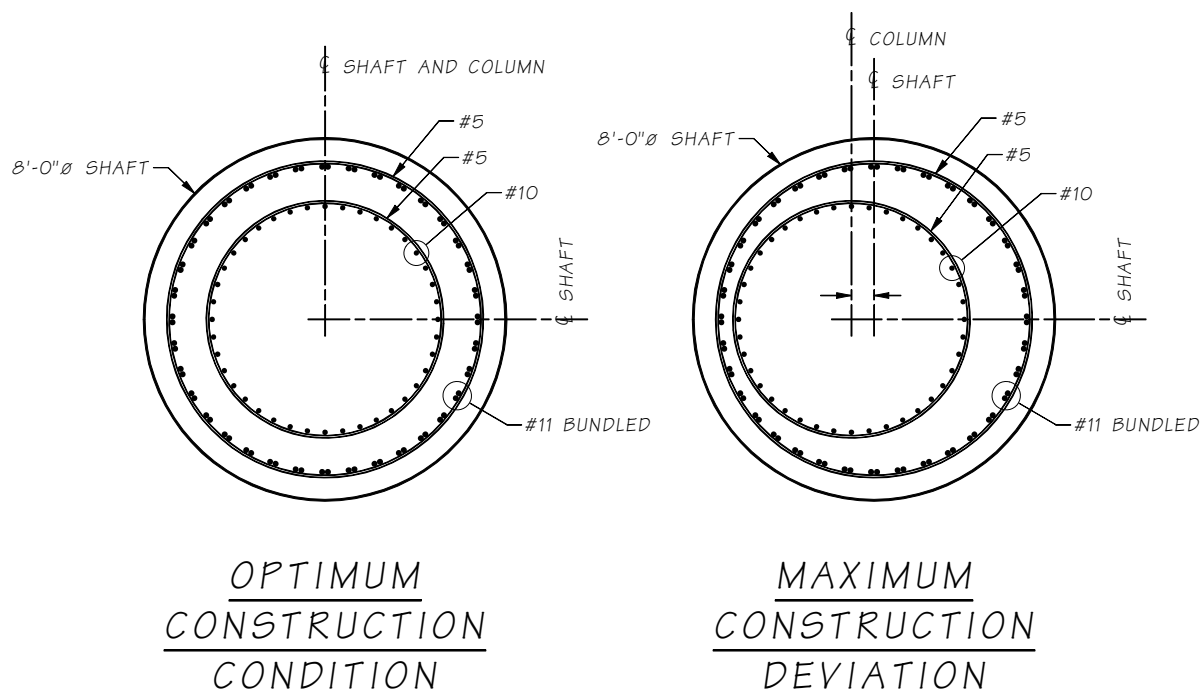


Figure 7.8.2-1

The shaft diameter shall be based on the maximum column diameter allowed by the following equation,

$$\text{Maximum Column Diameter} = \text{Shaft Diameter} - 2 * (\text{Shaft Concrete Cover}) - 2 * (\text{Shaft Horizontal Construction Tolerance}) - 2 * (\text{Shaft Cage Thickness})$$

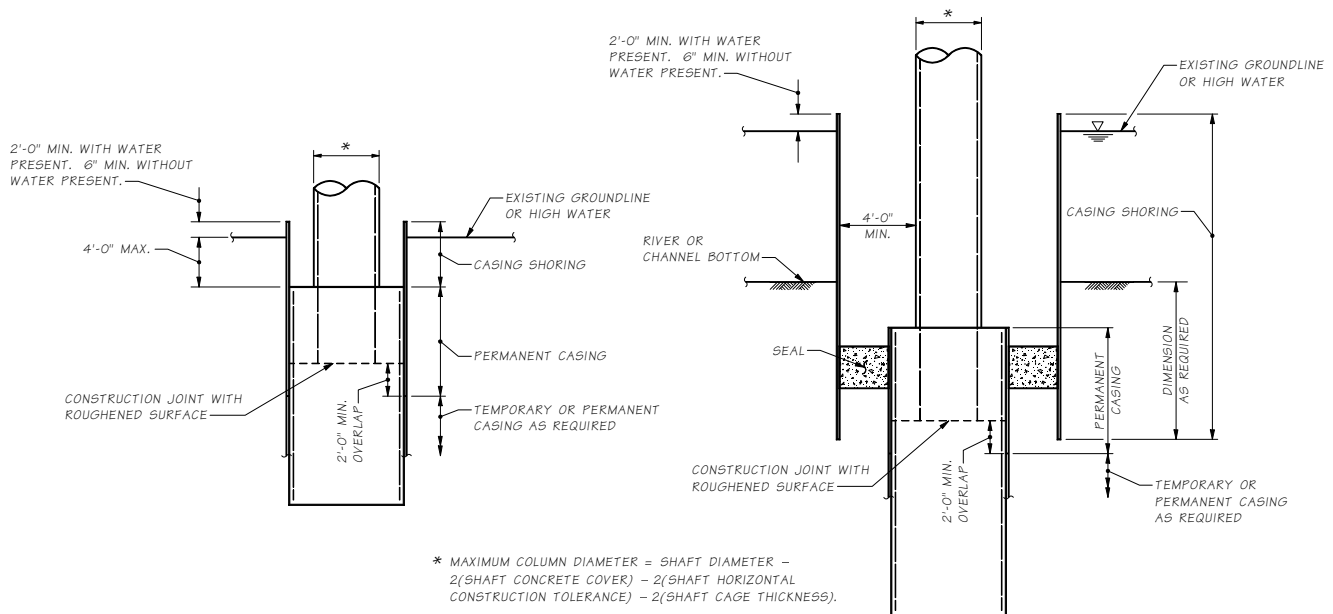
The shaft horizontal construction tolerance and shaft concrete cover shall conform to *Standard Specifications* Section 6-19.

If the column diameter used in design is larger than the maximum allowed for a given shaft size, as defined by the equation above, a larger shaft diameter shall be used.

The shaft diameter specified here should not be confused with the desirable casing shoring diameter discussed below.

- V. Casing shoring shall be provided for all shafts below grade or waterline. However, casing shoring requirements are different for shafts in shallow excavations and deep excavations. Shafts in deep excavations require a larger diameter casing shoring to allow access to the top of the shaft for column form placement and removal. The top of shafts in shallow excavations (approximately 4' or less) can be accessed from the ground line above, by reaching in or by "glory-holing", and therefore do not require larger diameter casing shoring, see [Figure 7.8.2-3](#).

The designer shall locate the top of in-water shafts above the water line when it simplifies construction. Where there is a strong design benefit to lowering the top of in-water shafts, they may be located below the groundline/mudline.

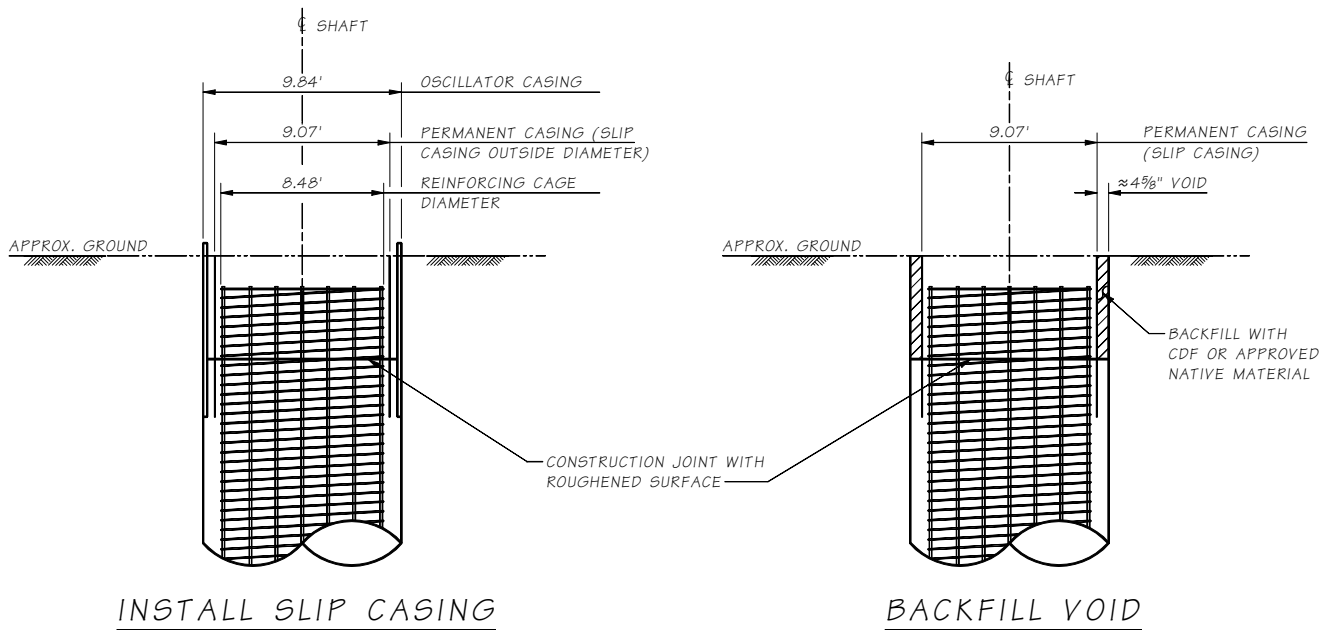


SHAFT CASING DETAILS
IN SHALLOW EXCAVATIONS
AND LOW WATER

SHAFT CASING DETAILS
IN DEEP EXCAVATIONS OR HIGH WATER

Figure 7.8.2-2

- W. Changes in shaft diameters due to construction tolerances shall not result in a reinforcing steel cage diameter different from the diameter shown in the plans (plan shaft diameter minus concrete cover). For example, metric casing diameters used in lieu of English casing diameters shall only result in an increase in concrete cover, except as noted below for single column/single shaft connections requiring slip casings. There are also exceptions for 4'-0", 5'-0", and 10'-0" diameter shafts, see [Table 7.8.2-1](#).
- X. Rotator and Oscillator drilling methods typically use a slip casing for permanent casing in single column/single shaft connections, as shown in [Figure 7.8.2-4](#).



10'-0" Ø Shaft Constructed With The Oscillator Method

Figure 7.8.2-3

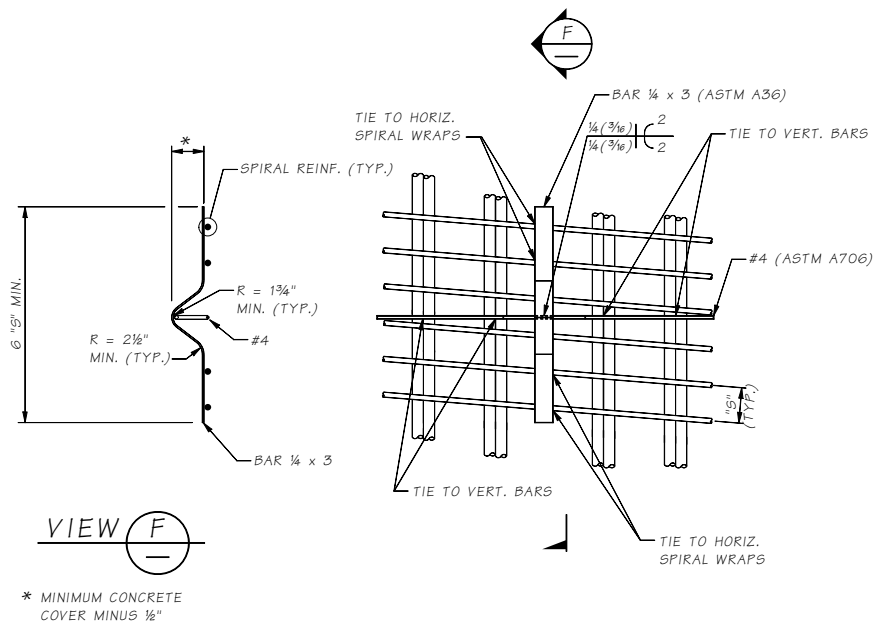
The use of the slip casing typically requires a modification to the reinforcing cage diameter. This should be considered during the structural design of the shaft. The slip casing also results in less concrete cover than the area of the shaft below the slip casing. See [Table 7.8.2-2](#) for expected reinforcing cage diameters and clear cover. Shafts shall be designed such that the reduced concrete cover is acceptable in this area because the casing is permanent. A minimum of 3" of concrete cover is achievable in this area for shafts 4'-0" diameter and larger and 1½" of cover for shafts less than 4'-0". These concrete cover requirements shall be kept as a minimum requirement. The reduction in strength (compared to the area below the slip casing) associated with the reduced shaft diameter that results from the slip casing is bounded within the shaft analysis and design methods prescribed here and elsewhere. Therefore the reduction in strength in this area can be ignored.

- Y. Reinforcing bar centralizers shall be detailed in the plans as shown in [Figure 7.8.2-5](#).

Nominal (Outside) Metric Casing Diameter		Maximum (Outside) Reinf. Cage Diameter to Accommodate Metric Casing ¹		Inside Diameter of Metric Casing ²	Nominal (Outside) Metric Slip Casing Diameter ³			Cage Clearance Below Slip Casing	Cage Clearance at Slip Casing ⁴
Meters	Feet	Inches	Feet	Inches	Inches	Feet	Meters	Inches	Inches
3.73	12.24	130.52	10.88	140.52	137.52	11.46	3.49	8.16	3.0
3.43	11.25	118.71	9.89	128.71	125.71	10.48	3.19	8.16	3.0
3.00	9.84	101.81	8.48	111.84	108.81	9.07	2.76	8.15	3.0
2.80	9.19	95.51	7.96	105.51	102.51	8.54	2.60	7.36	3.0
2.50	8.20	83.70	6.98	93.70	90.70	7.56	2.30	7.36	3.0
2.20	7.22	71.89	5.99	81.89	78.89	6.57	2.00	7.36	3.0
2.00	6.56	64.02	5.34	74.02	71.02	5.92	1.80	7.36	3.0
1.50	4.92	45.12	3.76	55.12	52.12	4.34	1.32	6.97	3.0
1.2	3.94	34.08	2.84	44.09	41.09	3.42	1.044	6.57	3.0
1.00	3.28	30.22	2.52	36.22	34.22	2.85	0.87	4.57	1.5
0.92	3.00	26.87	2.24	32.87	30.87	2.57	0.78	4.57	1.5

Notes:

1. Provided by Malcolm Drilling. Assumes minimum of 5" clearance to inside of oscillator casing on 4' and larger and uses 3" of clearance on smaller than 4' (1.2 meters).
2. Provided by Malcolm Drilling.
3. Provided by Malcolm Drilling. Slip casing is 3" smaller than inside diameter of temporary casing from 1.2 meters to 3 meters. 1 meter on down is 2" smaller in diameter.
4. Slip casing is typically $\frac{3}{8}$ " to $\frac{1}{2}$ " thick (provided by Malcolm Drilling). Cage clearance assumes $\frac{1}{2}$ " thick casing.

Table 7.8.2-2**CENTRALIZER NOTES:**

1. CENTRALIZERS SHALL BE EPOXY COATED OR PAINTED WITH INORGANIC ZINC AFTER FABRICATION.
2. EACH LEG SHALL BE TIED TO TWO VERTICAL BAR BUNDLES AND TWO SPIRAL WRAPS OR TWO HOOPS.
3. SEE STD. SPEC. 6-19.3(5)B FOR SPACING REQUIREMENTS.

Centralizer Detail
Figure 7.8.2-4

7.9 Piles and Piling

7.9.1 Pile Types

This section describes the piling used by the Bridge and Structures Office and their applications. In general, piles should not be used where spread footings can be used. However, where heavy scour conditions may occur, pile foundations should be considered in lieu of spread footings. Also, where large amounts of excavation may be necessary to place a spread footing, pile support may be more economical.

A. Cast-in-place Concrete Piles

Cast-in-place (CIP) concrete piles utilize driven steel pipe casings, which are then filled with reinforcing steel and concrete. The bottom of the casing is typically capped with a suitable flat plate for driving. However, the Geotechnical Branch may specify special tips when difficult driving is expected.

The Geotechnical Branch will determine the minimum wall thickness of the steel pipe casings based on driving conditions. However, the [Standard Specifications](#) require the contractor to provide a wall thickness that will prevent damage during driving.

B. Precast, Prestressed Concrete Piles

Precast, prestressed concrete piles are octagonal, or square in cross-section and are prestressed to allow longer handling lengths and resist driving stresses. Standard Plans are available for these types of piles.

C. Steel H Piles

Steel piles have been used where there are hard layers that must be penetrated in order to reach an adequate point bearing stratum. Steel stress is generally limited to 9.0 ksi (working stress) on the tip. H piling can act efficiently as friction piling due to its large surface area. Do not use steel H piling where the soil consists of only moderately dense material. In such conditions, it may be difficult to develop the friction capacity of the H piles and excessive pile length may result.

D. Timber Piles

Timber piles may be untreated or treated. Untreated piles are used only for temporary applications or where the entire pile will be permanently below the water line. Where composite piles are used, the splice must be located below the permanent water table. If doubt exists as to the location of the permanent water table, treated timber piles shall be used.

Where dense material exists, consideration should be given to allowing jetting (with loss of uplift capacity), use of shoes, or use of other pile types.

E. Steel Sheet Piles

Steel sheet piles are typically used for cofferdams and shoring and cribbing, but are usually not made a part of permanent construction.

CIP concrete piles consisting of steel casing filled with reinforcing steel and concrete are the preferred type of piling for WSDOT's permanent bridges. Other pile types such as precast, prestressed concrete piles, steel H piles, timber piles,

auger cast piles, and steel pipe piles shall not be used for WSDOT permanent bridge structures. These types of piles may be used for temporary bridges and other non-bridge applications subject to approval by the State Geotechnical Engineer and the State Bridge Design Engineer.

Micropiles shall not be used for new bridge foundations. This type of pile may be used for foundation strengthening of existing bridges, temporary bridges and other non-bridge applications subject to approval by the State Geotechnical Engineer and the State Bridge Design Engineer.

Battered piles shall not be used for bridge foundations to resist lateral loads.

The above limitations apply to all WSDOT bridges including mega projects and design-build contracts.

The above policy on pile types is the outcome of lengthy discussions and meetings between the bridge design, construction and Geotechnical Engineers. These limitations are to ensure improved durability, design and construction for WSDOT pile foundations.

In seismic applications there is a need for bi-directional demands. Steel H piles have proven to have little bending resistance for the purposes of resisting seismic load while circular CIP piles provide consistent capacities in all directions. Also, CIP pile casing is generally available in a full range of casing diameters. CIP piles are easily inspected after driving to ensure the quality of the finished pile prior to placing reinforcing steel and concrete.

Precast, prestressed concrete piles, and timber piles are difficult to splice and for establishing moment connections into the pile cap.

Micropiles have little bending resistance for the purposes of resisting lateral loads in seismic applications.

7.9.2 Single Pile Axial Resistance

The geotechnical report will provide the nominal axial resistance (R_n) and resistance factor (ϕ) for pile design. The factored pile load ($P_{U\text{ pile}}$) must be less than the factored resistance, ϕR_n , specified in the geotechnical report.

Pile axial loading ($P_{U\text{ pile}}$) due to loads applied to a pile cap are determined as follows:

$$(P_{U\text{ pile}}) = (P_{U\text{ pile group}})/N + M_{U\text{ group}} C/I_{\text{group}} + \gamma DD \quad (7.9.2-1)$$

Where:

- $M_{U\text{ group}}$ = Factored moment applied to the pile group. This includes eccentric *LL*, *DC*, centrifugal force (*CE*), etc. Generally, the dynamic load allowance (*IM*) does not apply.
- C = Distance from the centroid of the pile group to the center of the pile under consideration.
- I_{group} = Moment of inertia of the pile group
- N = Number of piles in the pile group
- $P_{U\text{ pile group}}$ = Factored axial load to the pile group
- DD = Downdrag force specified in the geotechnical report
- γ = Load factor specified in the geotechnical report

Pile selfweight is typically neglected. As shown above, downdrag forces are treated as load to the pile when designing for axial resistance. However, it should not be included in the structural analysis of the bridge.

See [Section 7.8.1](#) “Axial Resistance” of shafts for discussion on load combinations when considering liquefaction, scour and on downdrag effects. These guidelines are also applicable to piles.

7.9.3 Block Failure

For the strength and extreme event limit states, if the soil is characterized as cohesive, the pile group resistance shall also be checked for the potential for a “block” failure, as described in AASHTO LRFD Section 10.7.3.9. This check requires interaction between the designer and the Geotechnical Engineer. The check is performed by the Geotechnical Engineer based on loads provided by the designer. If a block failure appears likely, the pile group size shall be increased so that a block failure is prevented.

7.9.4 Pile Uplift

Piles may be designed for uplift if specified in the geotechnical report. In general, pile construction methods that require preboring, jetting, or spudding will reduce uplift capacity.

7.9.5 Pile Spacing

Pile spacing determination is typically determined collaboratively with the Geotechnical Engineer. The [Geotechnical Design Manual](#) M 46-03 specifies a minimum center-to-center spacing of 30" or 2.5 pile diameters. However, center-to-center spacings of less than 2.5 pile diameters may be considered on a case-by-case basis.

7.9.6 Structural Design and Detailing of CIP Concrete Piles

The structural design and detailing of CIP Concrete piles is similar to column design with the following guidelines:

- A. **Concrete** Class 5000P Concrete shall be specified for CIP concrete piles. The top 10' of concrete in the pile is to be vibrated. Use $1.0 f'_c$ for the structural design.
- B. For structural design, the reinforcement alone shall be designed to resist the total moment throughout the length of pile without considering strength of the steel casing. The minimum reinforcement shall be 0.75 percent A_g for SDC B, C, and D and shall be provided for the full length of the pile unless approved by the WSDOT Bridge Design Engineer. Minimum clearance between longitudinal bars shall meet the requirements in [Appendix 5.1-A2](#).
- C. If the pile to footing/cap connection is not a plastic hinge zone longitudinal reinforcement need only extend above the pile into the footing/cap a distance equal to $1.0 l_d$ (tension). If the pile to footing/cap connection is a plastic hinge zone longitudinal reinforcement shall extend above the pile into the footing/cap a distance equal to $1.25 l_d$.

- D. Since the diameter of the concrete portion of the pile is dependent on the steel casing thickness, the as-built diameter will not be known during design (since the casing thickness is determined by the contractor). As such, a casing thickness must be assumed for design. The structural engineer should work closely with the Geotechnical Engineer to determine a suitable casing thickness to assume based on expected driving conditions. A pile drivability analysis may be required for this. Otherwise, the following can typically be assumed:
- $\frac{1}{4}$ " for piles less than 14" in diameter
 - $\frac{3}{8}$ " for piles 14" to 18" in diameter
 - $\frac{1}{2}$ " for larger piles
- E. Steel casing for 24" diameter and smaller CIP piling should be designated by nominal diameter rather than inside diameter. *Standard Specifications* Section 9-10.5 requires steel casings to meet ASTM A252 Grade 2, which is purchased by nominal diameter (outside diameter) and wall thickness. A pile thickness should not be stated in the plans. As stated previously, the *Standard Specifications* require the contractor to determine the pile casing thickness required for driving.
- F. Transverse spiral reinforcement shall be designed to resist the maximum shear in the pile. Avoid a spiral pitch of less than 3". The minimum spiral shall be a #4 bar at 9" pitch. If the pile to footing/cap connection is not a plastic hinge zone the volumetric requirements of AASHTO LRFD Section 5.13.4.6 need not be met.
- G. Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD. Resistance factors for Extreme Event Limit States shall be per the latest *AASHTO Seismic Specifications*.
- H. Piles are typically assumed to be continuously supported. Normally, the soil surrounding a foundation element provides sufficient bracing against a buckling failure. Piles that are driven through very weak soils should be designed for reduced lateral support, using information from the Geotechnical Branch as appropriate. AASHTO LRFD 10.7.3.13.4 may be used to estimate the column length for buckling. Piles driven through firm material normally can be considered fully supported for column action (buckling not critical) below the ground.
- I. The axial load along the pile varies due to side friction. It is considered conservative, however, to design the pile for the full axial load plus the maximum moment. The entire pile is then typically reinforced for this axial load and moment.
- J. In all cases of uplift, the connection between the pile and the footing must be carefully designed and detailed. The bond between the pile and the seal may be considered as contributing to the uplift resistance. This bond value shall be limited to 10 psi. The pile must be adequate to carry tension throughout its length. For example, a timber pile with a splice sleeve could not be used.

7.9.7 Pile Splices

Pile splices shall be avoided where possible. If splices may be required in timber piling, a splice shall be detailed on the plans. Splices between treated and untreated timber shall always be located below the permanent water line. Concrete pile splices shall have the same strength as unspliced piles.

7.9.8 Pile Lateral Design

The strength limit state for lateral resistance is only structural, though the determination of pile fixity is the result of soil-structure interaction. A failure of the soil does not occur; the soil will continue to displace at constant or slightly increasing resistance. Failure occurs when the pile reaches the structural limit state and this limit state is reached, in the general case, when the nominal combined bending, shear, and axial resistance is reached.

Piles resist horizontal forces by a combination of internal strength and the passive pressure resistance of the surrounding soil. The capacity of the pile to carry horizontal loads should be investigated using a soil/structural analysis. For more information on modeling individual piles or pile groups, see [Section 7.2](#), Foundation Modeling and [Section 7.2.6](#) Lateral Analysis of Piles and Shafts.

7.9.9 Battered Piles

As stated previously, battered piles shall not be used to resist lateral loads for new bridge foundations. Where battered piles are used, the maximum batter shall be 4½:12. Piles with batters in excess of this become very difficult to drive and the bearing values become difficult to predict. Ensure that battered piling do not intersect piling from adjacent footings within the maximum length of the piles.

7.9.10 Pile Tip Elevations and Quantities

Pile length quantities provided to PS&E are based on the **minimum** tip elevation given in the geotechnical report or the **expected** depth required for **axial** design, whichever is greater. If the **minimum** tip elevation given in the geotechnical report is greater than the design tip elevation, overdriving the pile will be required. The Geotechnical Engineer shall be contacted to evaluate driving conditions. A General Provision will be required in the Special Provisions to alert the contractor of the additional effort needed to drive these piles.

If a minimum tip elevation is required, it shall be shown in the contract plans.

Minimum pile tip elevations provided in the geotechnical report may need to be adjusted to lower elevations depending on the results of the lateral, axial, and uplift analysis. If adjustment in the minimum tip elevations is necessary, or if the pile diameter needed is different than what was assumed for the geotechnical report, the Geotechnical Branch **must** be informed so that pile drivability can be re-evaluated.

Note that lateral loading and uplift requirements may influence (possibly increase) the number of piles required in the group if the capacity available at a reasonable minimum tip elevation is not adequate. This will depend on the soil conditions and the loading requirements. For example, if the upper soil is very soft or will liquefy, making the minimum tip elevation deeper is unlikely to improve the lateral response of the piles enough to be adequate. Adding more piles to the group or using a larger pile diameter to increase the pile stiffness may be the only solution.

7.9.11 Plan Pile Resistance

The Bridge Plan General Notes shall list the Ultimate Bearing Capacity (Nominal Driving Resistance, R_{ndr}) in tons. This information is used by the contractor to determine the pile casing thickness and size the hammer to drive the piles. The resistance for several piers may be presented in a table as shown in [Figure 7.9.11-1](#). If overdriving the piles is required to reach the minimum tip elevation, the estimated amount of overdriving (tons) shall be specified in the Special Provisions.

THE PILES SHALL BE DRIVEN TO AN ULTIMATE BEARING CAPACITY AS FOLLOWS:	
PIER NO.	ULTIMATE BEARING CAPACITY (TONS)
1	=====
4	=====

Figure 7.9.11-1

The total factored pile axial loading must be less than ϕR_n for the pile design. Designers should note that the driving resistance might be greater than the design loading for liquefied soil conditions. This is not an overdriving condition. This is due to the resistance liquefied soils being ignored for design, but included in the driving criteria to place the piles.

7.10 Concrete-Filled **Steel** Tubes

7.10.1 Scope

This section shall be taken to supersede AASHTO LRFD and AASHTO Seismic requirements for concrete-filled **steel** tubes (or pipes). The use of concrete-filled **steel** tubes (CFST) and reinforced concrete-filled **steel** tubes (RCFST) requires approval from the WSDOT Bridge Design Engineer **when used as a ductile element as part of an earthquake-resisting system**.

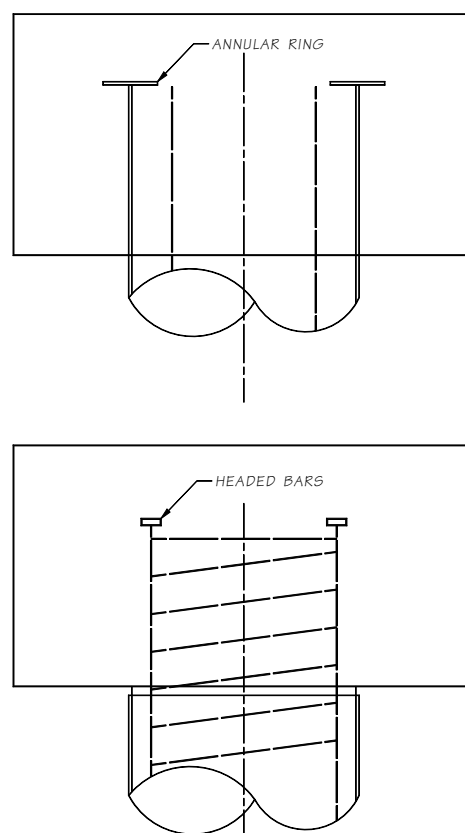
For a given diameter, CFST and RCFST have been shown to offer strength and stiffness beyond a conventional reinforced concrete (RC) member. And recent research has shown that CFST members can sustain large cyclic drifts with minimal damage. The design methods herein regarding concrete-filled **steel** tubes are largely based on study, testing and recommendations compiled by the University of Washington (UW).

The concrete for CFST members tested at the UW was a low-shrinkage, self-consolidating concrete. The nominal concrete strengths were 6 ksi and 10 ksi. This represents structural concrete with a minimum specified strength of 4 ksi, and an expected strength 25 percent to 50 percent larger.

Prior CALTRANS and ARMY research programs studied fully restrained connections for CFST pier to foundation connections.

Two of those connections are further discussed in this chapter and are shown in the Figure below. The first CFST-to-cap connection type involves an annular ring attached to the top of the CFST, and is partially embedded into the pile cap. This anchored connection resists flexural loading from the pile through strutting action to the bottom of the pile cap (resulting from the portion of tube of the CFST that is in tension) and the top of the pile cap (resulting from the portion of tube of the CFST column that in compression). The tests show this connection is both simple to construct and fully effective in transferring flexure. The current ACI procedure (ACI 318-2011) was recommended by the UW as a conservative approach to design against punching shear in this type of connection.

The second CFST-to-cap connection type requires a circular reinforcing cage with headed longitudinal bars that extend into the concrete cap. The CFST is discontinued just below the cap. This connection type is beneficial for plastic design in that it allows the designer some flexibility in the plastic demand that must be resisted by the cap. Refer to Section 7.10.4 for further discussion.



CFST Column-to-Cap Connections
Figure 7.10.1-1

Transition connections between RC shafts and CFST shafts have not been tested, but considerable analysis has been performed at the UW. Models have been developed to predict the strength of RCFST members, and this RCFST behavior may be used to provide increased strength over a significant length of the pile relative to conventional RC construction. Overstrength factors for capacity protection design of adjacent members and joint shear design at connections were not addressed in the research.

7.10.2 Design Requirements

A. Materials

1. The concrete for CFST and RCFST shall be class 5000P.
2. Steel tubes shall conform to one of the following:
 - i. API 5L Grade X42 or X52 for longitudinal seam welded or helical (spiral) seam submerged-arc welded tube
 - ii. ASTM A 252 Grade 2 or 3 for longitudinal seam welded or helical (spiral) seam submerged-arc welded tube
 - iii. ASTM A 572 or ASTM A 588 for longitudinal seam welded tube
3. When a CFST or RCFST will be used as an earthquake resisting element and is expected to develop a plastic hinge, the steel tubes shall be fabricated from steel meeting the mechanical and chemistry requirements of AASHTO M 270 GR 50 (ASTM A 709 GR 50) regardless of fabrication method.
4. For capacity protected members at the extreme event limit state, expected material properties may be used to determine the expected nominal moment capacity. The expected yield strength, F_{ye} , for steel tubes shall be taken as $1.1F_y$.

B. Limit States

For strength limit states, the resistance factors for axial load effects on CFST shall be taken per AASHTO LRFD for tension- and compression- controlled reinforced concrete sections. The resistance factor for flexure shall be taken as 0.9. The resistance factor for shear shall be taken as 0.85. For extreme event limit states, resistance factors shall be taken as 1.0.

C. General Dimensions

The minimum tube wall thickness shall not be taken less than 3/8 inch at the time of installation. To develop the full plastic capacity of CFST or RCFST members, it is necessary to ensure that local buckling does not occur prior to development of the strength of the tube. Therefore the following D/t limits are recommended:

1. For members subjected to elastic forces:

$$\frac{D}{t} \leq 0.22 \frac{E}{F_y} \quad (7.10.2-1)$$

2. For members subjected to plastic forces::

$$\frac{D}{t} \leq 0.15 \frac{E}{F_y} \quad (7.10.2-2)$$

Where D is the outside diameter of the tube (in.), and t is the wall thickness of the tube (in.).

D. Stiffness

The effective stiffness, EI_{eff} , of circular CFST, as defined in Equation 7.10.2-3, shall be used to evaluate deflections, deformations, buckling resistance, and moment magnification. The effective stiffness factor, C' , is defined in Equation 7.10.2-4.

$$EI_{eff} = E_s I_s + C' E_c I_c \quad (7.10.2-3)$$

$$C' = 0.15 + \frac{P}{P_0} + \frac{A_s}{A_s + A_c} \leq 0.9 \quad (7.10.2-4)$$

P_0 is the nominal compressive resistance (**crushing load**) without moment, P is the factored axial load effect, and A_s is the combined area of the steel tube and steel reinforcing.

E. Flexure and Axial Resistance

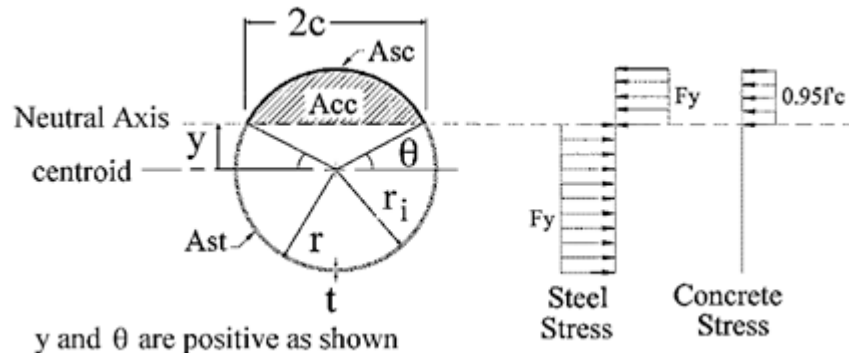
The flexural strength of CFST and RCFST members may be determined using the plastic stress distribution method (PSDM). The appropriate limit state stresses and geometry is shown in Figure 7.10.2-1.

Solutions for the interaction diagrams can be developed using parametric equations for $P(y)$ and $M(y)$ where y is the distance from the centroid to the neutral axis.

A positive value of P is a net compressive force. M and y are positive with the sign convention shown in Figure 7.10.2-1. The parameter y varies between plus and minus r_i , where r_i is the radius of the concrete core.

Stress is assumed to be plastically developed over the following regions of the section:

- A_{cc} = area of concrete effective in compression
- A_{sc} = area of the steel tube in compression
- A_{st} = area of the steel tube in tension
- A_{bc} = area of the internal steel reinforcing in compression
- A_{bt} = area of the internal steel reinforcing in tension

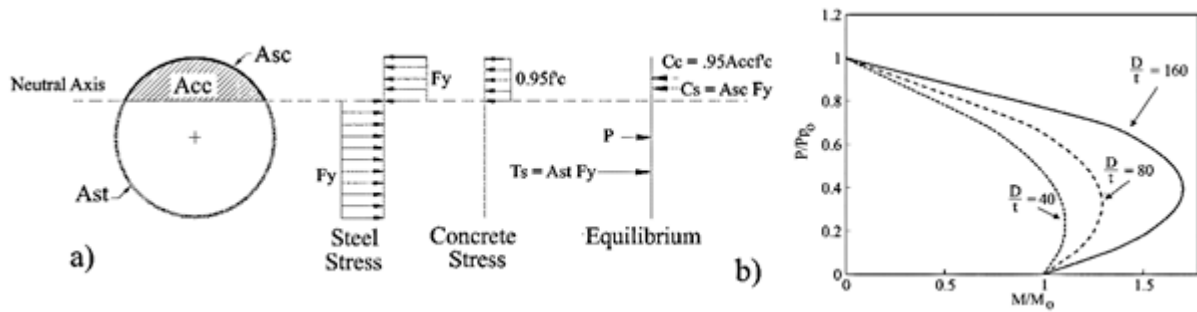


Plastic Stress Distribution Method

Figure 7.10.2-1

Alternatively, a strain-compatibility analysis can be performed with appropriate plastic stress-strain relationships.

1. **CFST Interaction** – A parametric solution for the nominal interaction diagram can be developed using Figure 7.10.2-2 and Equations 7.10.2-5 through 7.10.2-9. Figure 7.10.2-2b also shows normalized interaction curves for various D/t ratios.



Plastic Stress Distribution for CFST

Figure 7.10.2-2

$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta \right) r_i^2 - yc \right) * 0.95f'_c - 4\theta t r_m F_y \quad (7.10.2-5)$$

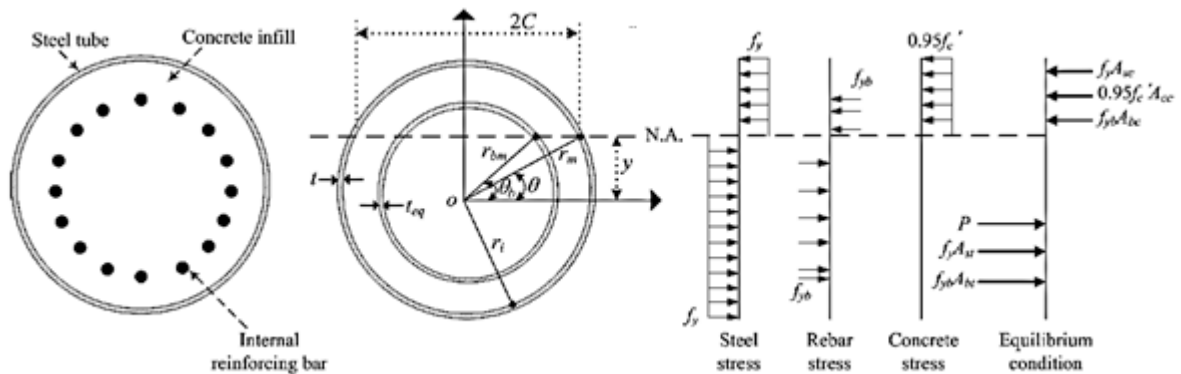
$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3} \right) * 0.95f'_c + 4ct \frac{r_m^2}{r_i} F_y \quad (7.10.2-6)$$

$$c = r_i \cos \theta \quad (7.10.2-7)$$

$$\theta = \sin^{-1} \left(\frac{y}{r_m} \right) \quad (7.10.2-8)$$

$$r_m = r - \frac{t}{2} \quad (7.10.2-9)$$

2. **RCFST Interaction** – A parametric solution for the nominal interaction diagram can be developed using Figure 7.10.2-3 and Equations 7.10.2-7 through 7.10.2-14. The internal steel reinforcing is idealized as a thin ring.



Plastic Stress Distribution for RCFST

Figure 7.10.2-3

$$P_n(y) = \left(\left(\frac{\pi}{2} - \theta \right) r_i^2 - yc \right) * 0.95f'_c - 4\theta t r_m F_y - t_b r_{bm} (4\theta_b F_{yb} + (\pi - 2\theta_b) 0.95f'_c) \quad (7.10.2-10)$$

$$M_n(y) = \left(c(r_i^2 - y^2) - \frac{c^3}{3} \right) * 0.95f'_c + 4ct \frac{r_m^2}{r_i} F_y + 4t_b r_{bm} c_b (F_{yb} - 0.95f'_c) \quad (7.10.2-11)$$

$$c_b = r_b \cos \theta_b \quad (7.10.2-12)$$

$$\theta_b = \sin^{-1} \left(\frac{y}{r_{bm}} \right) \quad (7.10.2-13)$$

$$t_b = \frac{nA_b}{2\pi r_{bm}} \quad (7.10.2-14)$$

The associated variables are defined as:

- r = radius to the outside of the steel tube (in)
- r_i = radius to the inside of the steel tube (in)
- r_m = radius to the center of the steel tube (in)
- r_{bm} = radius to the center of the internal reinforcing bars (in)
- t = wall thickness of the tube (in)
- t_b = wall thickness of a notional steel ring equivalent to the internal reinforcement (in)
- c = one half the chord length of the tube in compression (in)
- c_b = one half the chord length of a notional steel ring equivalent to the internal reinforcement in compression (in)
- θ = angle used to define c (rad.)
- θ_b = angle used to define c_b (rad.) θ_b shall be taken as $\pi/2$ if y/r_b is greater than 1 and θ_b shall be taken as $-\pi/2$ if y/r_b is less than -1.
- A_b = area of a typical steel bar comprising the internal reinforcement (in²)
- n = number of internal steel reinforcing bars

The requirements of AASHTO Seismic 8.16.2 for piles with permanent steel casing shall be applied to RCFST. Accordingly, the extent of longitudinal reinforcement may be reduced to only the upper portion of the member as needed to provide the required resistance of the member.

For CFST and RCFST, the area of the steel casing shall be included in the determination of the longitudinal reinforcement ratio. For RCFST, the minimum required longitudinal reinforcement ratio may be reduced to 0.005.

A. Stability Considerations for Unbraced of Partially-braced Members

Piles and shafts are typically assumed to be continually braced by the surrounding soil. Therefore they are not normally subject to P-Δ effects or other secondary effects. However, it is recognized that special circumstances such as scour, soil liquefaction, **piles used in marine structures**, or other conditions may leave piles and shafts subject to less than full bracing. In these circumstances, it may be necessary to consider stability effects.

B. Shear Resistance

The shear resistance of CFST and RCFST shall be taken as:

$$V_u = \phi V_n = \phi g_4 [g_1 (0.6 f_y g_2 A_s) + 0.0316 g_3 A_c \sqrt{f'_c}] \quad (7.10.2-15)$$

Where:

A_s = cross-sectional area of the steel tube

A_c = area of concrete within the steel tube

g_1 = coefficient for the shear capacity of the steel tube = 2.0

g_2 = coefficient for the effective shear area of steel tube = 0.5

g_3 = coefficient for the effect on concrete strength in shear due to confinement from the steel tube = 3.0

g_4 = coefficient for bond development between the concrete and steel tube = 1.0

The value of 1.0 for term g_4 is based on the assumption that the tube is fully developed as a composite section with the concrete and should be at least one diameter, D , from the point of maximum moment. If this condition is not satisfied, a reduction in g_4 may be required.

Equation 7.10.2-15 does not account for the influence of axial load on shear capacity. An alternate equation is presented in the UW research that accounts for axial load and can be utilized at the designer's discretion.

C. Corrosion

The design wall thickness for tubes shall be reduced for corrosion over a 75-year minimum design life. Minimum corrosion rates are specified below, except that the design thickness loss due to corrosion shall not be taken to be less than $\frac{1}{16}$ inch.

Soil embedded zone (undisturbed soil): 0.001 inch per year

Soil embedded zone (fill or disturbed soils): 0.003 inch per year

Immersed Zone (fresh water): 0.002 inch per year

Immersed and Tidal Zone (salt water): 0.004 inch per year

Splash Zone (salt water): 0.006 inch per year

Atmospheric Zone: 0.004 inch per year

The corrosion rates are taken from July 2008 CALTRANS memo to Designers 3-1, FHWA NHI-05-042 Design and Construction of Driven Pile Foundations, and the *Ferries Terminal Design Manual*.

The rates for corrosion in soil above assume that the soil is not highly corrosive. A site-specific assessment should be considered where a corrosive soil environment is suspected or known to exist. The potential for scour shall be considered when choosing a design corrosion rate.

7.10.3 CFST-to-Cap Annular Ring Connections

CFST-to-cap connections shall be designed as fully-restrained connections capable of resisting all load effects. The preferred connection to a concrete cap includes an annular ring at the top of the embedded tube. The connection design involves:

- Design of the annular ring
- Determination of the embedment depth
- A punching shear evaluation in the cap
- General design of the cap for flexure and shear

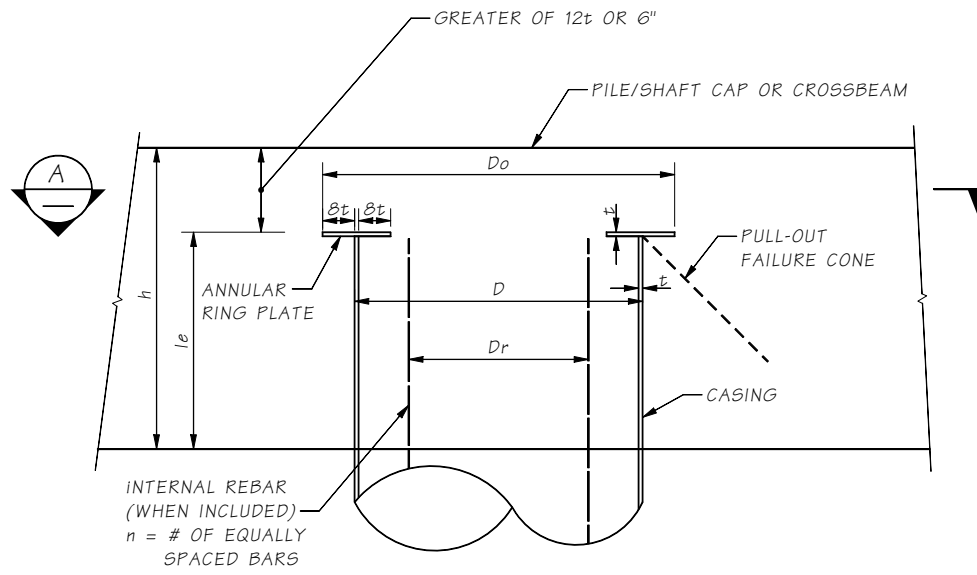
An alternative to the annular ring connection involves using a conventional reinforcing cage to splice the CFST to the cap.

- Reinforced concrete connection design

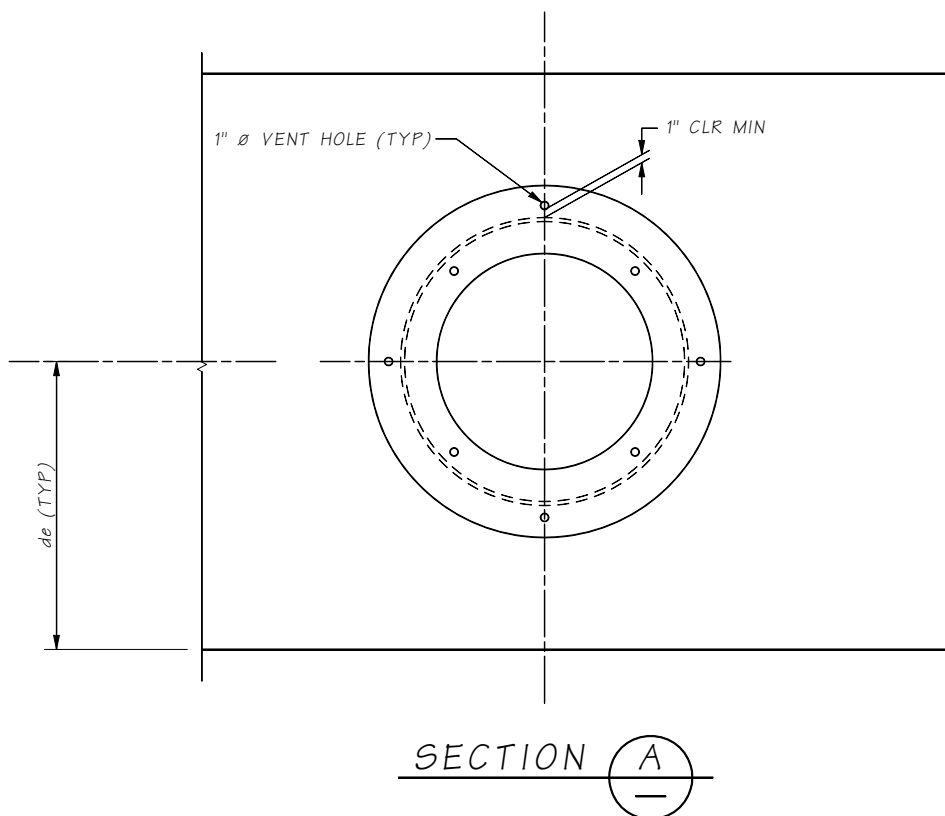
A. Annular Ring

An annular ring shall be welded to the end of the tube to provide anchorage and stress distribution, as shown in Figure 7.10.3-1. The ring shall be made of a steel of the same thickness and grade as the steel tube. The ring shall extend outside and inside the tube a distance of $8t$, where t is the thickness of the tube.

The ring shall have 1" diameter vent holes near the connection to the CFST to ensure concrete consolidation under the annular ring. There shall be a minimum of 4 holes equally spaced on the ring both outside the tube and inside the tube as shown in Figure 7.10.3-1.



PILE CAP OR CAP BEAM

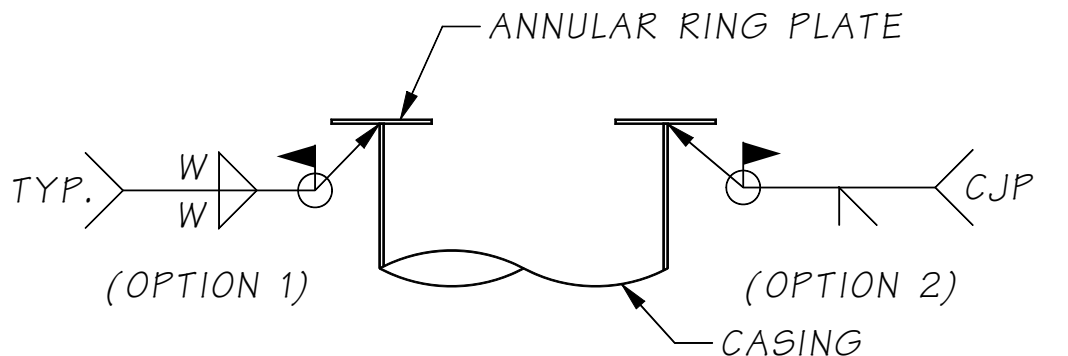


Annular Ring Connections
Figure 7.10.3-1

The ring shall be welded to the tube with complete joint penetration (CJP) welds or fillet welds on both the inside and outside of the tube. The fillet welds must be capable of developing the full tensile capacity of the tube. For this purpose, the minimum size, w , of the fillet welds shall be taken as:

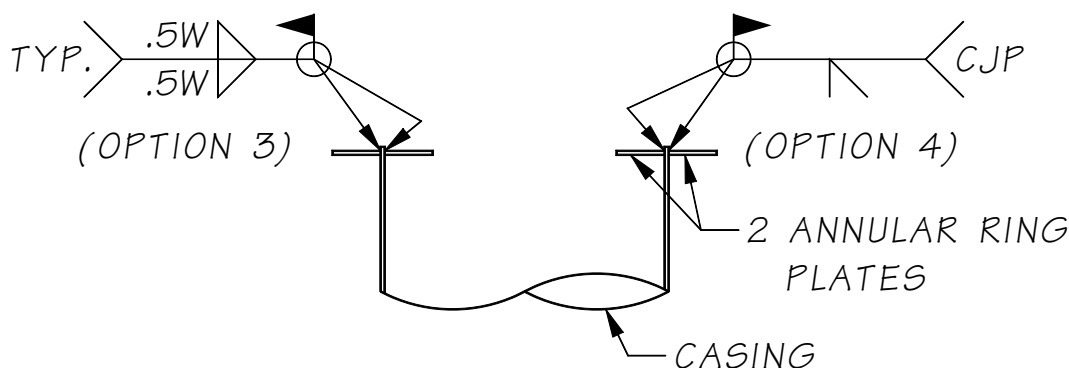
$$w \geq \frac{1.47F_u t}{F_{exx}} \quad (7.10.3-1)$$

Where F_u is the specified minimum tensile strength of the tube steel (ksi), and F_{exx} is the classification strength of the weld metal (ksi). **The fillet weld size equation is based on AASHTO and does not include the effects of loading direction. To further refine the weld size requirements to include loading direction, refer to the AISC Steel Construction Manual.** Typical CFST weld details are shown in Figure 7.10.3-2.



OPTION 1 AND OPTION 2

W - SIZE WELDS TO DEVELOP
FULL CAPACITY OF CASING WALL



OPTION 3 AND OPTION 4

NOTE: FOR OPTION 3, THE MINIMUM FILLET WELD SIZE SHALL BE LIMITED TO MINIMUM FILLET WELD SIZE PER AWS D1.1

Annular Ring Weld Detail
Figure 7.10.3-2

B. Embedment

The tube and the annular ring shall be embedded into the pile cap a distance, l_e , as defined in the following equations. To develop the yield strength of the CFST and plastic behavior is not expected, then the embedment length shall satisfy:

$$l_e \geq \sqrt{\frac{D_o^2}{4} + \frac{3.95DtF_y}{\sqrt{f'_{cf}}}} - \frac{D_o}{2} \quad (7.10.3-2)$$

To ensure full plastic behavior of the CFST, then the embedment length shall satisfy:

$$l_e \geq \sqrt{\frac{D_o^2}{4} + \frac{5.27DtF_u}{\sqrt{f'_{cf}}}} - \frac{D_o}{2} \quad (7.10.3-3)$$

Where f'_{cf} (ksi) is the specified 28-day compressive strength of the cap, D_o is the outside diameter of the annular ring as shown in Figure 7.10.3-1. This embedment length will develop the full plastic capacity of the CFST. As part of the displacement based design of the structure, hinge lengths and strain limits will need to be defined for the CFST. Values for the hinge length and strain limits have not been well documented in the research, however for steel casing fabricated from ASTM A 709 GR 50 steel, an assumed hinge length of one diameter, D , and reduced ultimate tensile strain of 0.13 may be used.

C. Punching Shear

The pile cap shall have adequate concrete depth, h , to preclude punching through the pile cap. The value of h shall satisfy:

$$h \geq \sqrt{\frac{D^2}{4} + \frac{1.68C_{max}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \quad (7.10.3-4)$$

In addition to the total cap depth requirement, h , a minimum of $12t$ or $6''$, whichever is greater, shall be provided above the annular ring. A layer of longitudinal reinforcement above the annular ring shall be provided to engage the punching shear strut forces.

Where the total compressive force of the couple, C_{max} , shall be taken as:

$$C_{max} = C_c + C_s \quad (7.10.3-5)$$

C_c and C_s are the compression forces in the concrete and the steel due to the combined bending and axial load as computed by the plastic stress distribution method for the most extreme load effect at the appropriate limit state.

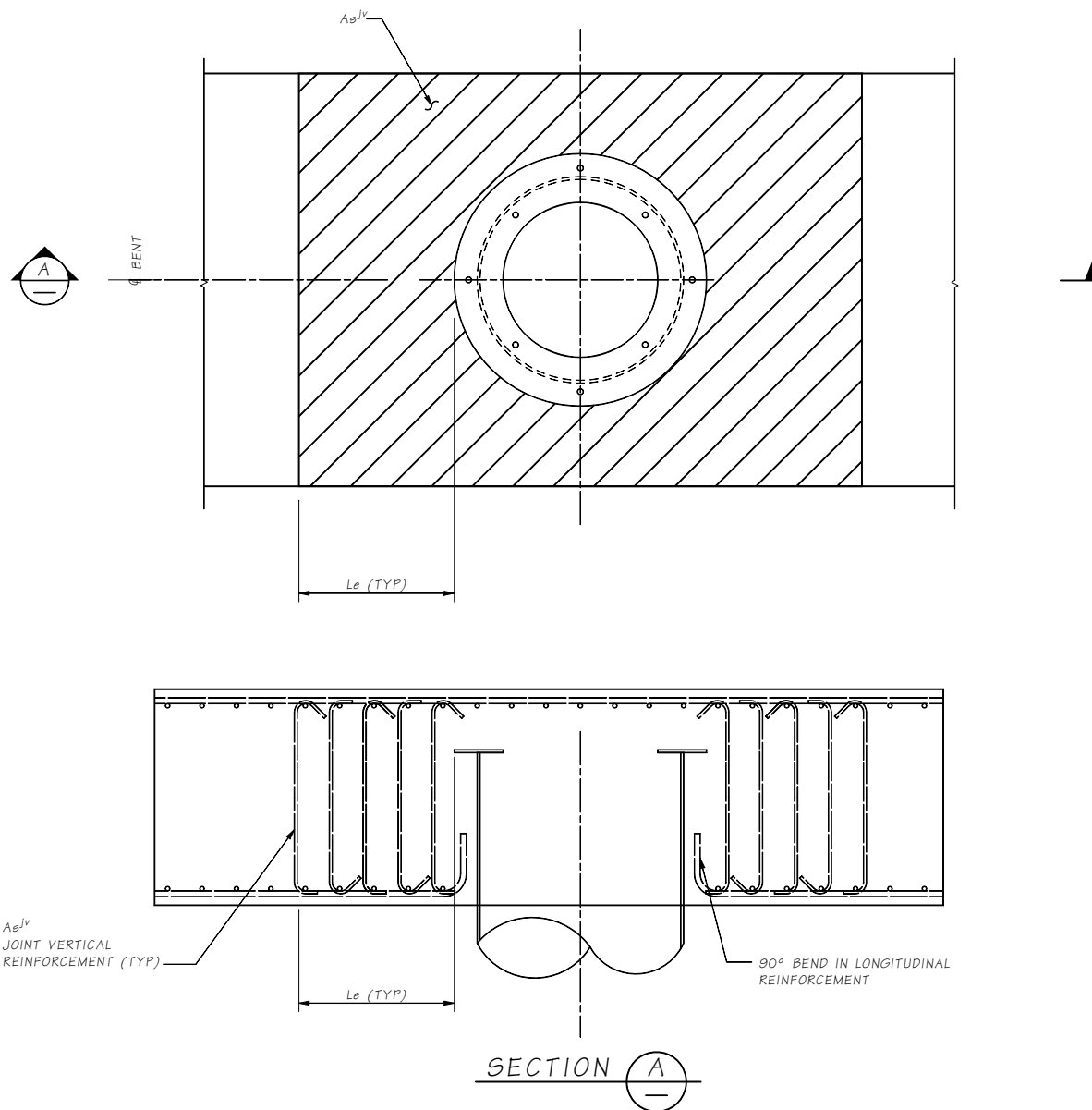
D. Pile Cap and/or Cap Beam Reinforcement

The pile cap should follow conventional design practice and must be adequate to sustain the foundation design loads. However, the concrete cap thickness shall be large enough to preclude punching shear and cone pullout of the CFST piles.

The edge distance shall be large enough to accommodate concrete struts **originating** at the base of the ring. The minimum edge distance, d_e , measured from center-of-tube to the edge of the cap shall be taken as:

$$d_e \geq D \quad (7.10.3-6)$$

CFSTs shall be adequately spaced to avoid intersecting concrete struts. The cap shall be designed to resist all flexural load effects. The flexural reinforcement in both directions shall be spaced uniformly across the length and width of the cap, but the bottom mat of flexural reinforcement will be interrupted by the concrete tube. The interrupted bars shall be provided, but they shall not be relied on to contribute to the flexural resistance of the cap. Figure 7.10.3-3 shows the configuration of the longitudinal reinforcing where it conflicts with the steel tube. Standard 90° hooks shall be used.



Reinforcement Detail at Cap Connection

Figure 7.10.3-3

The cap shall be designed to resist all shear load effects. Note that the minimum required embedment results in an average shear stress in the critical area surrounding the tube of $6\sqrt{f'_c}$ (psi). Assuming the concrete is capable of resisting a shear stress of approximately $2\sqrt{f'_c}$, vertical reinforcement will be required to resist an average shear stress of approximately $4\sqrt{f'_c}$. Additional requirements for shear demand resulting from other load combinations must also be considered.

Vertical ties shall be provided within the anchorage regions such that vertical ties intersect the pull-out cone on each side of the CFST subject to shear. **The vertical reinforcing, A_s^{jv} , shall be included according to Equation 7.10.3-7 where A_{st} is the total area of the steel tube embedded into the cap. This value provides a conservative amount of vertical reinforcing steel and satisfies the $4\sqrt{f'_c}$ requirement.**

$$A_s^{jv} = 0.65A_{st} \quad (7.10.3-7)$$

7.10.4 CFST-to-Cap Reinforced Concrete Connections

A circular reinforcing with headed longitudinal bars may be used to connect a CFST member to a concrete cap, where the steel casing is discontinued just below the cap. **Terminating the steel casing below the cap, as opposed to terminating at the bottom of the cap or just inside the cap, reduces the chances of concrete spalling at relatively low demands.** The reinforcing cage shall satisfy all of the requirements for a reinforced concrete connection, as well as the additional requirements in this section.

A. CFST Requirements

A steel ring shall be welded to the inside of the steel casing 3" below the top of casing to aid in transfer of forces from the reinforcement cage to the steel casing. A square bar the size of the steel casing wall thickness is sufficient. The bar may be continuous or consist of discontinuous segments, however the total length shall be at least 75% of the steel casing circumference and shall be equally distributed. Transverse reinforcing shall be used and shall extend into the concrete cap.

B. Embedment and Concrete Cover

The minimum embedment length, l_e , of the reinforcing cage into the cap shall satisfy:

$$l_e \geq \frac{\psi_e F_{yb}}{2\sqrt{f'_{cf}}} d_b \quad (7.10.4-1)$$

$$l_e \geq \sqrt{\frac{D^2}{4} + \frac{2F_{yb}A_{st}}{\sqrt{f'_{cf}}}} - \frac{D}{2} \quad (7.10.4-2)$$

Where ψ_e is a coating factor, which shall be taken as 1.0 for uncoated bars, and 1.2 for epoxy-coated bars.

The reinforcing cage shall extend into the CFST at least a distance of $2l_d$ below the top of the steel casing, where l_d is the development length of the longitudinal reinforcing **as defined in AASHTO.**

The concrete cover above headed longitudinal reinforcing shall exceed $3d_h$, where d_h is the diameter of the head. The concrete side cover adjacent to a head shall exceed d_h . When headed bars are not used, the development of longitudinal reinforcement into the cap shall be as specified in AASHTO LRFD or AASHTO Seismic, as appropriate.

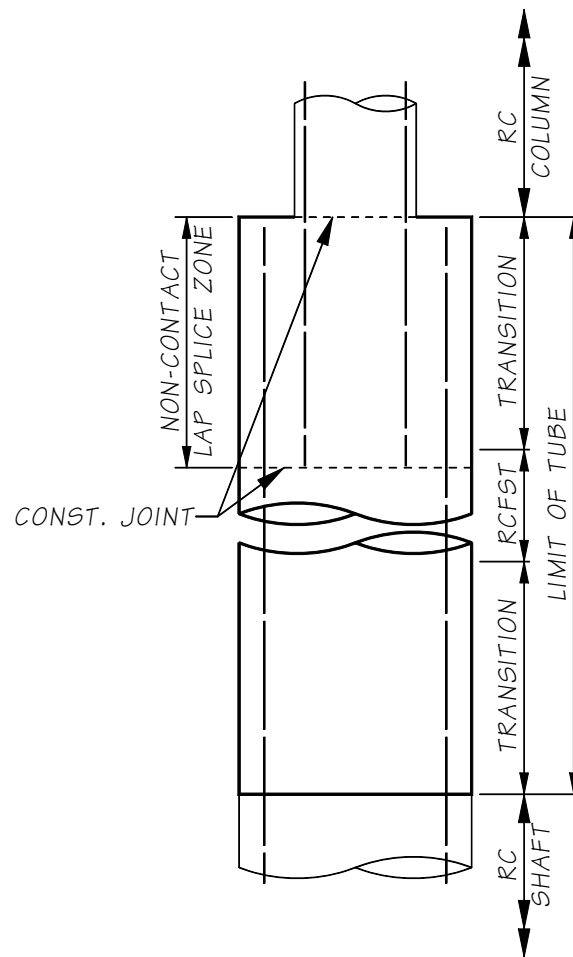
C. Pile Cap and/or Cap Beam Reinforcement

The pile cap should follow conventional design practice and must be adequate to sustain the foundation design loads. Joint reinforcement shall follow the requirements of the AASHTO Guide Specifications.

7.10.5 RCFST-to-Column and CFST-to Column Connections

Direct RCFST-to-column connections shall be designed as fully-restrained connections capable of resisting all load effects. The recommended RCFST shaft to reinforced concrete column connection is shown in Figure 7.10.4-1.

All column reinforcement shall be extended into the RCFST shaft for a length greater than or equal to the length required for noncontact lap splices between columns and shafts. The contribution of steel casing to the structural resistance of RCFST's varies from zero at the end of the tube to fully composite at the end of the transition zone. The transition zone length may be taken as $1.0D$. The use of slip casing in determining the resistance for RCFST shafts is not permitted.

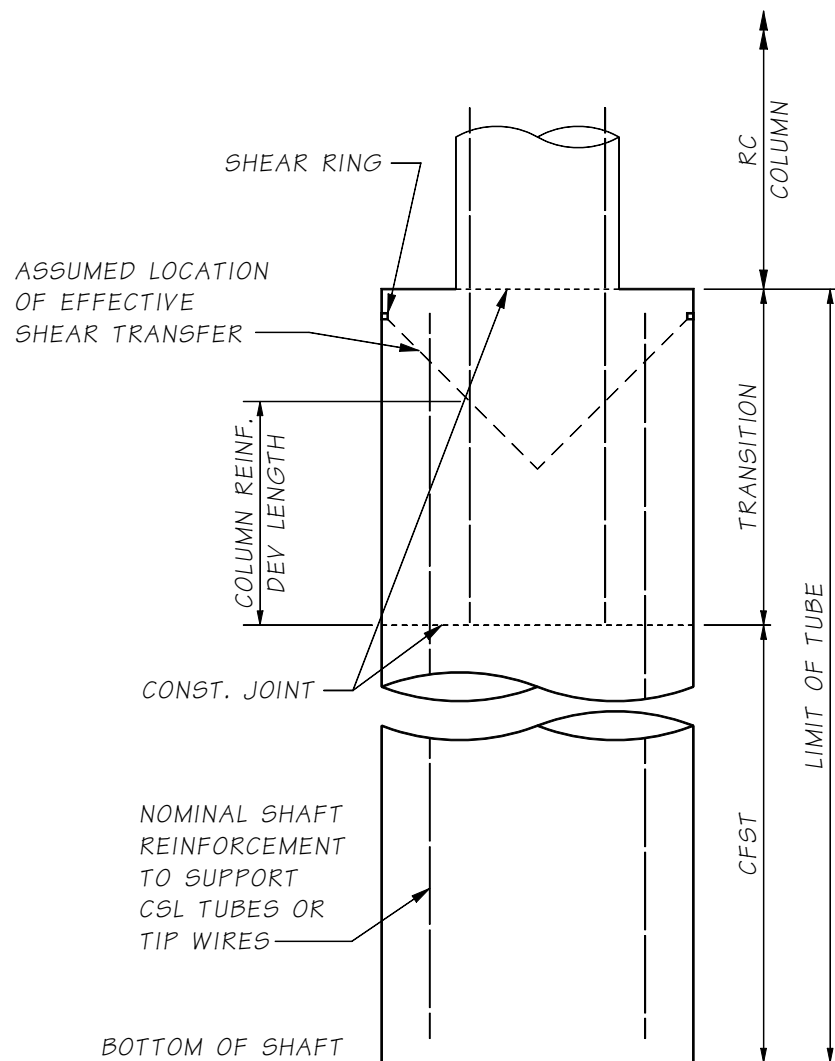


RCFST-to-Column Connection

Figure 7.10.5-1

CFST-to column connections do not require additional reinforcement within the shaft. However, the concrete within the shaft does require testing. A cage may be installed to support the CSL tubes or thermal integrity wire that is required. A steel ring shall be welded to the inside of the steel casing 3" below the top of casing to aid in transfer of forces from the column reinforcement cage to the steel casing. See Section 7.10.4A for steel ring requirements.

All column reinforcement shall be extended into the CFST shaft the maximum of $0.5D$ (shaft diameter) plus the column reinforcement development length and $1.0D$.



CFST-to Column Connection
Figure 7.10.5-2

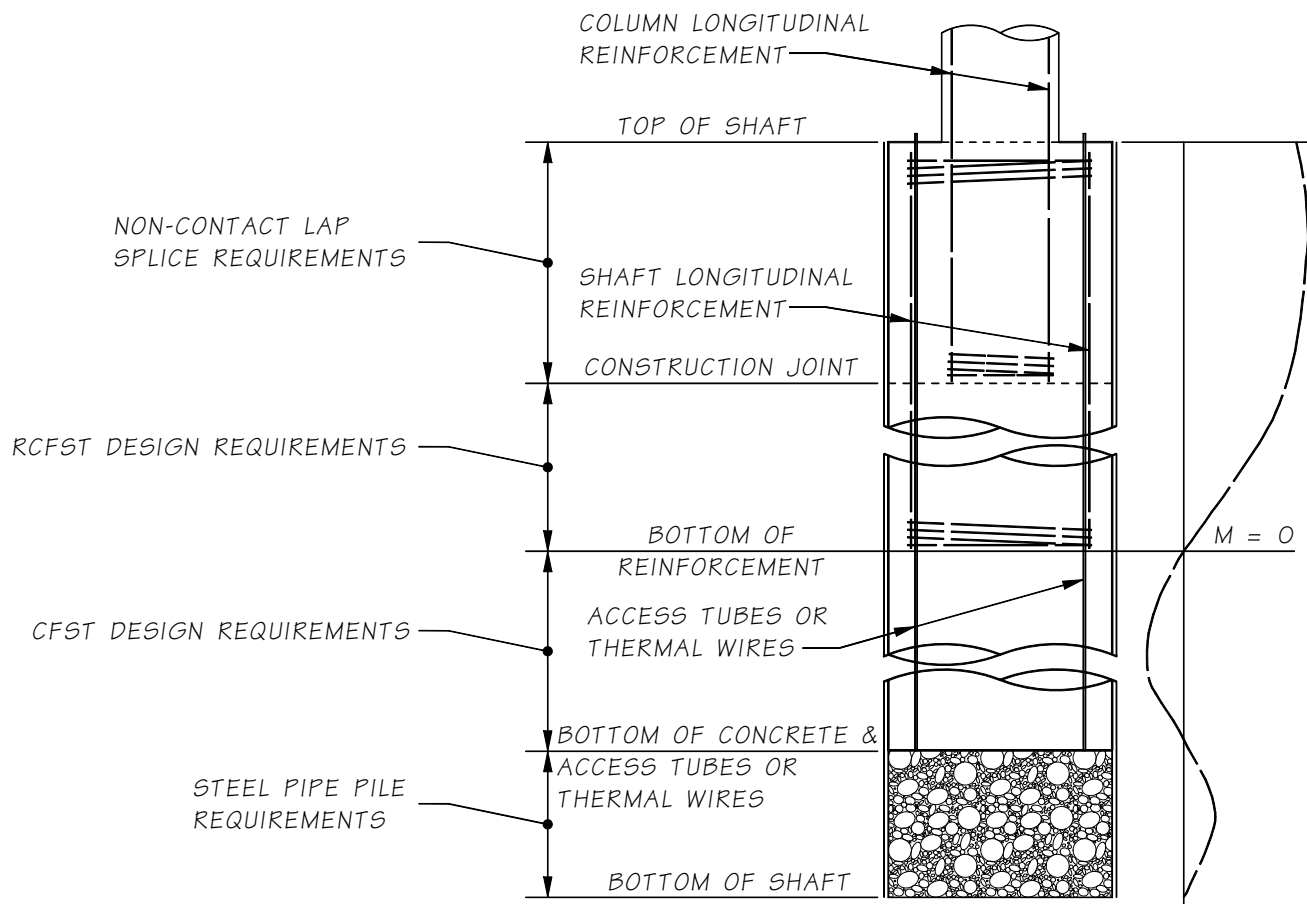
7.10.6 Partially-filled CFST

The use of partially-filled steel tubes for bridge foundations requires the approval of the WSDOT Bridge Design Engineer, and will only be used where conventional CFST members are grossly uneconomical or unconstructible.

Design zones of partially filled steel piles and shafts are shown in Figure 7.10.5-1. Longitudinal and transverse reinforcement shall extend to at least the first point of zero moment along the member under the peak loading condition.

Crosshole sonic log (CSL) testing or thermal integrity profiling (TIP) shall be performed in accordance with *Standard Specifications* Section 6-19.3(9). CSL tubes or thermal wires shall extend to the bottom of concrete.

Corrosion losses shall be considered on each exposed surface of the steel tube.



Partially-filled CFST
Figure 7.10.6-1

7.10.7 Construction Requirements

For CFSTs with tubes installed open-ended, the insides of the tube shall be excavated and then cleaned with an appropriate tool to remove all adhering soil and other material. When excavating, a nominal plug of soil shall be left at the bottom of the pile as determined by the Geotechnical Engineer. The Geotechnical Engineer shall be consulted to determine the method for computing axial compressive and uplift (if applicable) capacities of the CFSTs during design. In addition, the method for accepting CFSTs during construction, if installed as a driven pile, will need to be recommended by the Geotechnical Engineer. When driving an open ended pile, the total resistance obtained consists of a contribution from end bearing, external side friction, and internal side friction. Also, depending on the pile diameter and soil conditions, the pile could drive with or without a soil plug. When excavating the soil from inside of an open-ended driven pile, the resistance can be altered from the as-driven condition due to loss of internal side friction. All of these factors need to be considered during the design and shall be based on recommendations from the Geotechnical Engineer. Open-ended CFSTs installed using techniques similar to a drilled shaft shall be designed and constructed as a drilled shaft. Closed-ended CFSTs driven as a pile may be designed and accepted in the field similar to a closed-ended steel pile.

Welding for ASTM A 252 pipe shall conform to AWS D1.1/D1.1M, latest edition, Structural Welding Code, except that all weld filler metal shall be low hydrogen material selected from Table 4.1 in AASHTO/AWS D1.5M/D1.5:2010 *Bridge Welding Code*. All seams and splices shall be complete penetration welds.

Welding and joint geometry for the seam shall be qualified in accordance with AWS D1.1/D1.1M, latest edition, Structural Welding Code. The Contractor may submit documentation of prior qualification to the Engineer to satisfy this requirement.

For the fabrication of helical (spiral) seam submerged-arc welded pipe piles, the maximum radial offset of strip/plate edges shall be $\frac{1}{8}$ inch. The offset shall be transitioned with a taper weld and the slope shall not be less than a 1-to-2.5 taper. The weld reinforcement shall not be greater than $\frac{3}{16}$ inches and misalignment of weld beads shall not exceed $\frac{1}{8}$ inch.

If spirally welded pipe piles are allowed, skelp splices shall be located at least 1'-0" away from the annular ring.

Nondestructive evaluation (NDE) requirements for field welded splices shall be identified on the plans. The location of splices and NDE requirements shall be divided into 3 possible zones as determined by the Engineer:

1. No splices permitted – highly stressed areas
2. Splices permitted with 100 percent UT and visual inspection – moderately stressed areas
3. Splices permitted with 100 percent visual inspection – low stressed areas

7.10.8 Notation

A_b	=	area of a single bar for the internal reinforcement (in ²)
A_c	=	net cross-sectional area of the concrete (in ²)
A_g	=	cross-sectional area of the steel tube (in ²)
A_s	=	cross-sectional area of the steel tube and the longitudinal internal steel reinforcement (in ²)
c	=	one half the chord length of the tube in compression (in)
c_b	=	one half the chord length of a notional steel ring equivalent to the internal reinforcement in compression (in)
D	=	outside diameter of the tube (in.)
D_o	=	outside diameter of the annular ring (in.)
d_b	=	nominal diameter of a reinforcing bar (in)
d_e	=	minimum edge distance from center of CFST to edge of cap (in)
d_f	=	depth of cap (in)
E_c	=	elastic modulus of concrete (ksi)
El_{eff}	=	effective composite flexural cross-sectional stiffness of CFST or RCFST (k-in ²)
E_s	=	elastic modulus of steel (ksi)
F_{exx}	=	classification strength of weld metal (ksi)
F_u	=	specified minimum tensile strength of steel (ksi)
F_y	=	specified minimum yield strength of steel (ksi)
F_{yb}	=	specified minimum yield strength of reinforcing bars used for internal reinforcement (ksi)

f'_c	=	minimum specified 28-day compressive strength of concrete (ksi)
f'_{cf}	=	minimum specified 28-day compressive strength of concrete in a cap or footing (ksi)
g_1	=	coefficient for the shear capacity of the steel tube
g_2	=	coefficient for the effective shear area of steel tube
g_3	=	coefficient for the effect on concrete strength in shear due to confinement from the steel tube
g_4	=	coefficient for bond development between the concrete and steel tube
h	=	cap depth above the CFST required to resist punching shear in a cap (in)
I_c	=	uncracked moment of inertia of the concrete about the centroidal axis (in ⁴)
I_s	=	moment of inertia of the steel tube and the longitudinal internal steel reinforcement about the centroidal axis (in ⁴)
l_e	=	Required embedment length for CFST embedded in a concrete cap (in)
$M(y)$	=	nominal moment resistance as a function of the parameter y (kip-in)
M_o	=	plastic moment resistance of members without axial load (kip-in)
n	=	number of equally spaced longitudinal internal steel reinforcement
$P(y)$	=	nominal compressive resistance as function of the parameter y (kips)
P_u	=	factored axial load acting on member (kip)
P_o	=	compressive resistance of a member without consideration of flexure (kips)
r	=	radius to the outside of the steel tube (in)

7.11 Bridge Standard Drawings

7.3-A1-1 Column Silo Cover

7.8-A1-1 Typical Shaft Details

7.12 Appendices

Appendix 7.3-A2	Noncontact Lap Splice Length Column to Shaft Connections
Appendix 7-B1	Linear Spring Calculation Method II (Technique I)
Appendix 7-B2	Pile Footing Matrix Example Method II (Technique I)
Appendix 7-B3	Non-Linear Springs Method III

Appendix 7.3-A2

Noncontact Lap Splice Length Column to Shaft Connections

Column Diameter (ft)	Shaft Diameter (ft)	Column Longitudinal Bar Size							
		#6	#7	#8	#9	#10	#11	#14	#18
		Basic Lap Splice Length, L _s (in)							
		36.7	42.8	49.0	55.2	62.2	69.0	82.9	110.5
		Minimum Noncontact Lap Splice Length, L _{ns} = L _s + s							
3'-0"	5.5 *	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	6.0	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
	6.5	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
	7.0	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	9'-6"	11'-6"
	8.0	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	10'-0"	12'-0"
3'-6"	6.0 *	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	6.5	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
	7.0	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
	8.0	5'-6"	6'-0"	6'-6"	7'-6"	8'-0"	8'-6"	9'-6"	12'-0"
	9.0	6'-0"	6'-6"	7'-0"	8'-0"	8'-6"	9'-0"	10'-0"	12'-6"
4'-0"	6.5 *	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	7.0	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
	8.0	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	9'-6"	11'-6"
	9.0	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	10'-0"	12'-0"
	9.5	6'-0"	6'-6"	7'-0"	8'-0"	8'-6"	9'-0"	10'-0"	12'-6"
4'-6"	7.0 *	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	8.0	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
	9.0	5'-6"	6'-0"	6'-6"	7'-6"	8'-0"	8'-6"	9'-6"	12'-0"
	9.5	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	10'-0"	12'-0"
	10.0	6'-0"	6'-6"	7'-0"	8'-0"	8'-6"	9'-0"	10'-0"	12'-6"
5'-0"	8.0 *	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
	9.0	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	9'-6"	11'-6"
	9.5	5'-6"	6'-0"	6'-6"	7'-6"	8'-0"	8'-6"	9'-6"	12'-0"
	10.0	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	10'-0"	12'-0"
	11.0	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	10'-6"	12'-6"
5'-6"	8.0 *	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	9.0	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
	9.5	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	9'-6"	11'-6"
	10.0	5'-6"	6'-0"	6'-6"	7'-6"	8'-0"	8'-6"	9'-6"	12'-0"
	11.0	6'-0"	6'-6"	7'-0"	8'-0"	8'-6"	9'-0"	10'-0"	12'-6"
6'-0"	9.0 *	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
	9.5	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
	10.0	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	9'-6"	11'-6"
	11.0	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	10'-0"	12'-0"
	12.0	6'-6"	7'-0"	7'-6"	8'-0"	8'-6"	9'-0"	10'-6"	12'-6"

(Continued on following page)

Column Diameter (ft)	Shaft Diameter (ft)	Column Longitudinal Bar Size							
		#6	#7	#8	#9	#10	#11	#14	#18
		Basic Lap Splice Length, L _s (in)							
		36.7	42.8	49.0	55.2	62.2	69.0	82.9	110.5
		Minimum Noncontact Lap Splice Length, L _{ns} = L _s + s							
6'-6"	9.0 *	4'-0"	4'-6"	5'-0"	6'-0"	6'-6"	7'-0"	8'-0"	10'-6"
	9.5	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	8'-6"	10'-6"
	10.0	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	11.0	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
	12.0	5'-6"	6'-0"	6'-6"	7'-6"	8'-0"	8'-6"	9'-6"	12'-0"
7'-0"	9.5 *	4'-0"	4'-6"	5'-0"	6'-0"	6'-6"	7'-0"	8'-0"	10'-6"
	10.0	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	8'-6"	10'-6"
	11.0	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
	12.0	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	8'-0"	9'-6"	11'-6"
7'-6"	10.0 *	4'-0"	4'-6"	5'-0"	6'-0"	6'-6"	7'-0"	8'-0"	10'-6"
	11.0	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"
	12.0	5'-0"	5'-6"	6'-0"	7'-0"	7'-6"	8'-0"	9'-0"	11'-6"
8'-0"	11.0 *	4'-6"	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	8'-6"	10'-6"
	12.0	5'-0"	5'-6"	6'-0"	6'-6"	7'-0"	7'-6"	9'-0"	11'-0"
8'-6"	11.0 *	4'-0"	4'-6"	5'-0"	6'-0"	6'-6"	7'-0"	8'-0"	10'-6"
	12.0	4'-6"	5'-0"	5'-6"	6'-6"	7'-0"	7'-6"	8'-6"	11'-0"

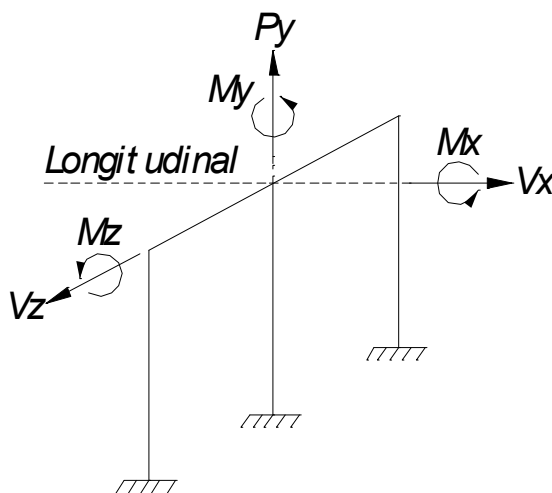
Notes:

1. All values based on normal weight concrete with $f'_c = 4.0$ ksi.
2. The basic splice length, L_s , is based on 1.7 times the tension development length L_d . L_d is calculated per Section 5.11.2.1.1 of the AASHTO LRFD Bridge Design Specifications 7th Edition, 2015 Interim Revisions. The 1.7 (Class C lap splice) factor was maintained for the basic splice length calculations to be consistent with the original TRAC research for noncontact lap splices. This table does not apply when column longitudinal bars are bundled.
3. Development lengths are based on uncoated deformed bars. For epoxy coated bars add $0.2 \cdot L_s$ or $0.5 \cdot L_s$ to L_{ns} depending on column bar spacing and clear cover. See AASHTO 5.11.2.1.2 for spacing and cover requirements.
4. The clearance between the column and shaft reinforcement, s , is based on an out-to-out dimension of the cages. Concrete cover to the column reinforcing is 2", and concrete cover to the shaft reinforcing is per Std. Spec. 6-19.3(5)C. Also, s has been increased by the allowable shaft construction tolerances in Std. Spec. 6-19.3(1)A. If an oversized cage is used in conjunction with an oversize casing, s may need to be increased further.
5. The reinforcement confinement factor, λ_{rc} , is assumed to be 0.4 for all cases and is based on the assumption that the column reinforcement is well confined with column transverse reinforcement, shaft transverse reinforcement, and in most cases a permanent shaft casing.
6. All noncontact splice lengths have been rounded up to the nearest 6".

* Minimum common shaft diameter for specified column diameter to meet minimum clearance and construction tolerances.

Method II (Technique I) - Matrix Coefficient Definitions

The stiffness matrix containing the spring values and using the standard coordinate system is shown in [Figure 7-B1-1](#). The sign of all the terms must be determined based on the sign convention.



Global Coordinate System
Figure 7-B1-1

$$\begin{Bmatrix} Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{Bmatrix} \begin{Bmatrix} K11 & 0 & 0 & 0 & 0 & K16 \\ 0 & K22 & 0 & 0 & 0 & 0 \\ 0 & 0 & K33 & K34 & 0 & 0 \\ 0 & 0 & K43 & K44 & 0 & 0 \\ 0 & 0 & 0 & 0 & K55 & 0 \\ K61 & 0 & 0 & 0 & 0 & K66 \end{Bmatrix} \times \begin{Bmatrix} \Delta x \\ \Delta y \\ \Delta z \\ \theta x \\ \theta y \\ \theta z \end{Bmatrix} = \begin{Bmatrix} Vx \\ Py \\ Vz \\ Mx \\ My \\ Mz \end{Bmatrix}$$

Standard Global Matrix
Figure 7-B1-2

Where the linear spring constants or K values are defined as follows using the Global Coordinates:

- $K11 = +V_{x(app)}/\Delta_x$ = Longitudinal Lateral Stiffness (kip/in)
- $K22 = AE/L$ = Vertical or Axial Stiffness (k/in)
- $K33 = -V_{z(app)}/\Delta_z$ = Transverse Lateral Stiffness (k/in)
- $K44 = +M_{x(app)}/\theta_x$ = Transverse Bending or Moment Stiffness (kip-in/rad)
- $K55 = JG/L$ = Torsional Stiffness (kip-in/rad)
- $K66 = +M_{z(app)}/\theta_z$ = Longitudinal Bending or Moment Stiffness (kip-in/rad)
- $K34 = -V_{z(ind)}/\theta_x$ = Transverse Lateral Cross-couple term (kip/rad)
- $K16 = +V_{x(ind)}/\theta_z$ = Longitudinal Lateral Cross-couple term (kip/rad)
- $K43 = +M_{x(ind)}/\Delta_z$ = Transverse Moment Cross-couple term (kip-in/in)
- $K61 = +M_{z(ind)}/\Delta_x$ = Longitudinal Moment Cross-couple term (kip-in/in)

Fixed Head vs. Free Head Spring Calculations

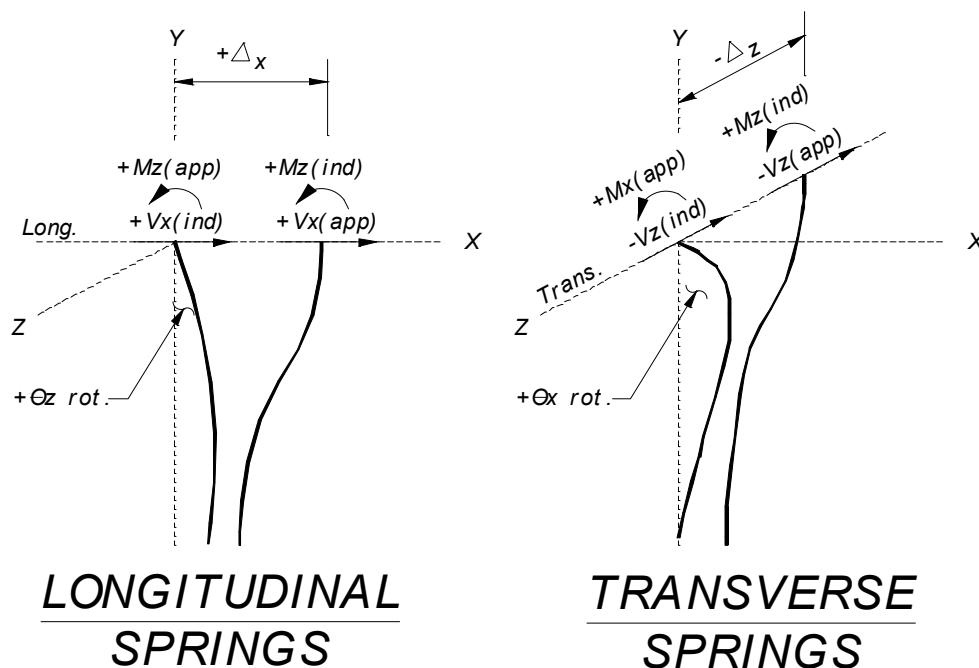
Fixed Head

If the shear and moment are creating deflection in OPPOSING directions where the spring is located, a fixed head boundary condition is required to model the loaded foundation in a finite element model. See [Figure 7-B1-3](#) for the fixed head coordinate system assumed in the following spring calculations.

Since applying load to a fixed end results in no reaction, a soil/structure interaction analysis will generally analyze the shear and moment simultaneously as a free head. Using the soil response results, a cross-couple correction term will be required in a FEM to produce the induced moment in the element modeling the fixed head condition. If accurate stresses in fixed head element are not required, the cross-couple term may be omitted.

There are two ways to model fixed head pile group. The most common method for a column footing is to use a group spring to model a group of piles or shafts as one set of springs. This method uses six linear springs to represent the foundation behavior. Lateral loads resisted by Cross-couples terms do not apply and individual pile loads must be calculated from the FEM results.

The second method would be to model the individual piles. This is more helpful for analyzing local stresses in the foundation cap element and for each pile. Cross-couple terms may be included and individual pile loads are generated in the FEM.



Fixed Head Coordinate System

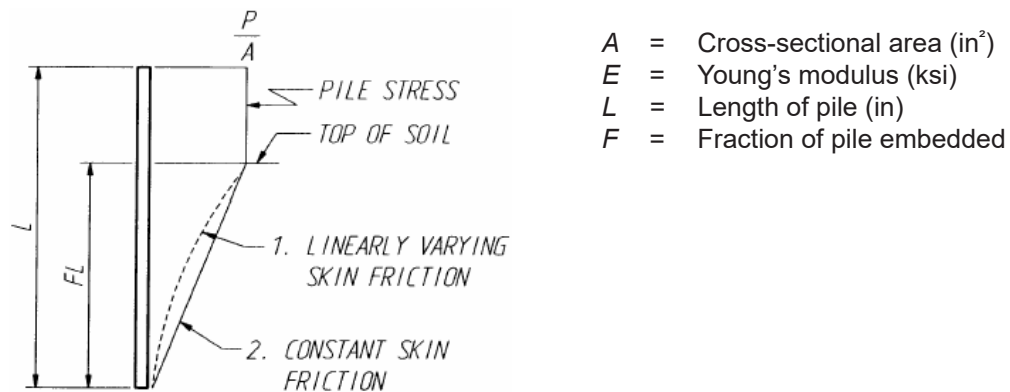
Figure 7-B1-3

Free Head

If the shear and moment are creating deflection in the SAME direction where the spring is located, a free head boundary condition is required to model the loaded foundation in a finite element model. If a free head boundary condition is assumed Method II (Technique II) described in [Section 7.2.5](#).

Vertical Springs (K22)

Vertical spring constants can be calculated from the following three assumptions. See [Figure 7-B1-4](#) and the following definitions. REF: *Seismic Design of Highway Bridges Workshop Manual*, Pub. No. FHWA-IP-81-2, Jan 1981.



Pile Stress
Figure 7-B1-4

Point Bearing Piles:

$$K22 = \frac{AE}{L}$$

Friction Piles with linearly varying skin friction:

$$K22 = \frac{AE}{\left(1 - \frac{2F}{3}\right)L}, \text{ with } F = 1.0 \text{ (fully embedded), } K22 = 3 \frac{AE}{L}$$

Friction Piles with constant skin friction:

$$K22 = \frac{AE}{\left(1 - \frac{F}{2}\right)L}, \text{ with } F = 1.0 \text{ (fully embedded), } K22 = 2 \frac{AE}{L}$$

Torsional Springs (K55)

In general, torsional spring constants for individual piles are based on the mechanics of the pile. The equation for torsional stiffness is given below.

$$K55 = \frac{M}{\phi} = \frac{T}{\phi} = \frac{JG}{L}$$

Where:

$$\begin{aligned} G &= 0.4E \\ J &= \text{Torsional moment of inertia} \\ L &= \text{Length of pile} \end{aligned}$$

Lateral Springs (K11 & K33)

A fixed head lateral spring can be found by applying the shear and axial load in a soil response program with the rotation at the top equal to zero and finding the lateral deflection that results. The spring value is the applied shear divided by the resulting deflection.

$$K11 = \frac{V_{x(app)}}{\Delta_x} \text{ (longitudinal)} \qquad K33 = \frac{V_{z(app)}}{-\Delta_z} \text{ (transverse)}$$

Rotational Springs (K44 & K66)

Ideally a fixed head boundary condition would result in no rotation. Therefore K44 and K66 would be infinitely stiff.

In the past, the fixed head rotational springs were found by applying the moment and axial load in a soil response program with the translation at the top equal to zero and finding the rotation that results. The spring value is the applied moment divided by the resulting rotation.

$$K66 = \frac{M_{z(app)}}{\theta_z} \text{ (longitudinal)} \qquad K44 = \frac{M_{x(app)}}{\theta_x} \text{ (transverse)}$$

Cross-Couple Springs (K16, K34, K43 & K61)

Fixed Head

Cross-couple springs will not be symmetric for non-linear modeling foundation modeling. Since finite element programs will use matrix multiplication to generate reactions, doing the math is the easy way to show the effect of cross-couple terms. Note that K16 and K34 terms will have opposite signs.

$$\begin{Bmatrix} V_x \\ V_x \\ P_y \\ V_z \\ M_x \\ M_y \\ M_z \end{Bmatrix} \begin{bmatrix} & V_x & P_y & V_z & M_x & M_y & M_z \\ V_x & K11 & 0 & 0 & 0 & 0 & K16 \\ P_y & 0 & K22 & 0 & 0 & 0 & 0 \\ V_z & 0 & 0 & K33 & K34 & 0 & 0 \\ M_x & 0 & 0 & K43 & K44 & 0 & 0 \\ M_y & 0 & 0 & 0 & 0 & K55 & 0 \\ M_z & K61 & 0 & 0 & 0 & 0 & K66 \end{bmatrix} \times \begin{Bmatrix} \text{Disp.} \\ \Delta_x \\ \Delta_y \\ \Delta_z \\ \theta_x \\ \theta_y \\ \theta_z \end{Bmatrix} = \begin{Bmatrix} \text{Force} \\ V_x \\ P_y \\ V_z \\ M_x \\ M_y \\ M_z \end{Bmatrix}$$

The longitudinal reactions are:

$$V_x = K11 \cdot \Delta_x + K16 \cdot \theta_z \text{ and } M_z = K61 \cdot \Delta_x + K66 \cdot \theta_z$$

The transverse reactions are:

$$V_z = K33 \cdot \Delta_z + K34 \cdot \theta_x \text{ and } M_x = K43 \cdot \Delta_z + K44 \cdot \theta_x$$

For a true fixed head boundary condition (translation only) in the X and Z directions, there will be no rotation about the X and Z axis. θ_x and θ_z will be zero (or approach zero). This means the K34 and K16 cross-couple terms will not affect the shear reactions. Likewise, the K66 and K44 rotational terms zero out and do not affect the moment reaction. This leaves the K61 and K43 cross-couple terms to generate induced moments based on the deflections in the X and Z directions. Designers should note, the cross-couple moments are applied to a fixed footing element and are resisted axially by the piles. This affects the local stress in the footing and axial loading of the pile much more than the column moment and shear, which is usually the primary focus for design.

K11 and K66 (or K33 and K44) alone do not predict the shape or reaction of the foundation element. The cross-couple term K16 (or K34) will add a shear force to correct the applied moment deflection.

Modeling real life features may be somewhat different than the theoretically true fixed condition. The top of a column at the superstructure or some pile and shaft applications may have opposing shear and moment, however the moment may be much less than the theoretical induced free head moment value. In other words, there may be significant rotations that need to be accounted for in the spring modeling. Designers need to be aware of this situation and use engineering judgment. The FEM would have rotations about the X and Z axis. θ_x and θ_z will NOT be zero and both the cross-couples terms and rotational springs may significantly affect the analysis.

The spring value for the lateral cross-couple term is the induced shear divided by the associated rotation.

$$K16 = \frac{V_{x(ind)}}{\theta_z} \text{ (longitudinal)} \quad K34 = \frac{V_{z(ind)}}{\theta_x} \text{ (transverse)}$$

The spring value for the moment cross-couple term is the induced shear divided by the associated rotation.

$$K61 = \frac{M_{x(ind)}}{-\Delta_z} \text{ (longitudinal)} \quad K43 = \frac{M_{z(ind)}}{-\Delta_x} \text{ (transverse)}$$

Method II (Technique I) – Pile Footing Matrix Example

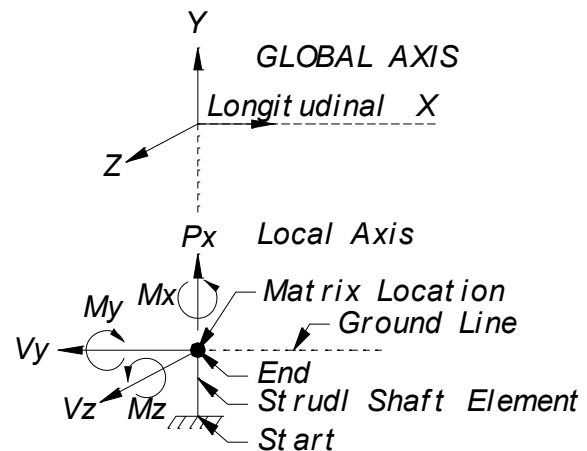
A matrix with cross-couple terms is a valid method to model pile supported footings. The analysis assumes the piles will behave similar to a column fixed at the bottom (in the soil) with lateral translation only at the top (no rotation). This requires fixed head boundary conditions to calculate spring values.

The LPILE program will solve for non-linear soil results for individual piles. See group effects in [Section 7.2.5](#) to reduce the soil properties of a pile in a group in both the transverse and longitudinal directions. This sample matrix calculates a foundation spring for an individual pile.

The pile spring requires eight pile stiffness terms for a matrix as discussed in [Appendix 7-B1](#). The following sample calculations discuss the lateral, longitudinal, and cross-couple spring coefficients for a GTStrudl local coordinate system. See [Appendix 7-B1](#) for axial and torsion springs.

The maximum FEM transverse and longitudinal seismic loads (V_y , M_z , V_z , M_y and axial P_x) provide two loads cases for analysis in LPILE. The LPILE results of these two load cases will be used to calculate lateral, longitudinal, and cross-couple spring coefficients.

This sample calculation assumes there are no group effects. Only the longitudinal direction will be calculated, the transverse direction will be similar. A standard global coordinate system is assumed for the bridge. This sample will assume a different local axis coordinate system to input matrix terms, as shown in Figure 7-B2-1. When the coordinate system changes, the sign convention of shear and moment also will change. This will be expressed in a 6×6 matrix by changing the location of the spring values and in sign of any cross-couple terms.



Local Axis Coordinate System
Figure 7-B2-1

The locations of matrix terms are shown in [Figure 7-B1-2](#). The displacements are local and this requires the spring coefficients to be moved to produce the correct local reactions. The X axis is the new vertical direction. The Y axis is the new longitudinal direction. The spring coefficient definitions and notation remains the same as defined in [Appendix 7-B1](#). Note the shift in diagonal terms and locations of the cross-couple terms.

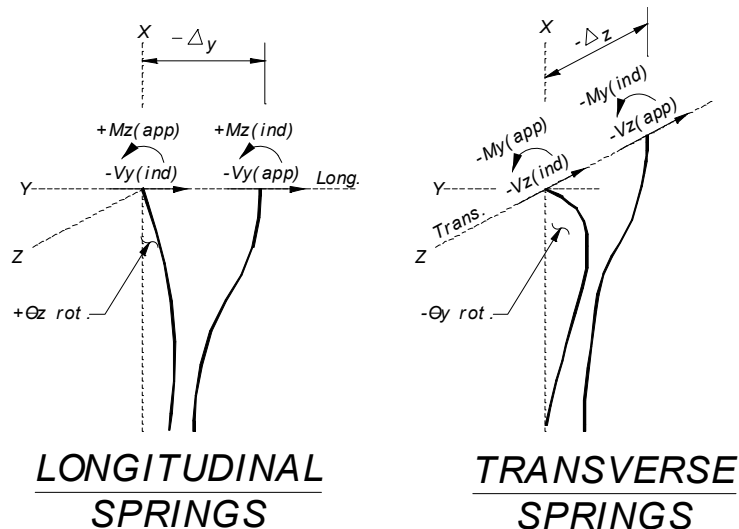
$$\begin{Bmatrix} P_x \\ P_y \\ P_z \\ M_x \\ M_y \\ M_z \end{Bmatrix} = \begin{Bmatrix} K_{22} & 0 & 0 & 0 & 0 & 0 \\ 0 & K_{11} & 0 & 0 & 0 & K_{16} \\ 0 & 0 & K_{33} & 0 & K_{34} & 0 \\ 0 & 0 & 0 & K_{55} & 0 & 0 \\ 0 & 0 & K_{43} & 0 & K_{44} & 0 \\ 0 & K_{61} & 0 & 0 & 0 & K_{66} \end{Bmatrix} \begin{Bmatrix} \Delta x \\ \Delta y \\ \Delta z \\ \theta_x \\ \theta_y \\ \theta_z \end{Bmatrix} = \begin{Bmatrix} P_x \\ P_y \\ P_z \\ M_x \\ M_y \\ M_z \end{Bmatrix}$$

Matrix in Local Coordinate System

Figure 7-B2-2

Where the linear spring constants or K values are defined as follows (see Figure 7-B2-3 for direction and sign convention):

- $K_{11} = -V_{y(app)}/-\Delta_y$ = Longitudinal Lateral Stiffness (kip/in)
 $K_{22} = AE/L$ = Vertical or Axial Stiffness (k/in)
 $K_{33} = -V_{z(app)}/-\Delta_z$ = Transverse Lateral Stiffness (k/in)
 $K_{44} = -M_{y(app)}/-\theta_y$ = Transverse Bending or Moment Stiffness (kip-in/rad)
 $K_{55} = JG/L$ = Torsional Stiffness (kip-in/rad)
 $K_{66} = M_{z(app)}/\theta_z$ = Longitudinal Bending or Moment Stiffness (kip-in/rad)
 $K_{34} = -V_{z(ind)}/-\theta_y$ = Transverse Lateral Cross-couple term (kip/rad)
 $K_{16} = -V_{y(ind)}/+\theta_z$ = Longitudinal Lateral Cross-couple term (kip/rad)
 $K_{43} = -M_{y(ind)}/-\Delta_z$ = Longitudinal Moment Cross-couple term (kip-in/in)
 $K_{61} = +M_{z(ind)}/-\Delta_y$ = Transverse Moment Cross-couple term (kip-in/in)



Local Coordinate System Sign Conventions

Figure 7-B2-3

Results from the local coordinate system

- $P_x = 50,000$ lbs (axial load)
 $V_y = -60,000$ lbs (shear along longitudinal axis)
 $V_z = -40,000$ lbs (shear along transverse axis)
 $M_y = -2,230,000$ lb-in (moment about longitudinal axis)
 $M_z = 3,350,000$ lb-in (moment about transverse axis)

Load Case 1 - Longitudinal Direction

Load Case 1 applies the lateral load (V_y) and axial load (P_x), and restrains the top against rotation (slope = 0 rad).

Input to LPILE:

Boundary condition code	=	2
Lateral load at the pile head	=	-60000.000 lbs
Slope at the pile head	=	0.000 in/in
Axial load at the pile head	=	50000.000 lbs

Output from LPILE:

X (in)	Deflection Δ_y (in)	Moment $M_{z(ind)}$ (lbs-in)	Shear $V_{y(app)}$ (lbs)	Slope (Rad.)
0.000	-0.13576	3.761E+06	-60000.000	0.000000

Load Case 2 - Longitudinal Direction

Load case 2 applies the moment load (M_z) and axial load (P_x), and restrains the top against deflection (deflection = 0 rad).

Input to LPILE:

Boundary condition code	=	4
Deflection at the pile head	=	0.000 in
Moment at the pile head	=	3.350E+06 in-lbs
Axial load at the pile head	=	50000.000 lbs

Output from LPILE:

X (in)	Deflection (in)	Moment $M_{z(ind)}$ (lbs-in)	Shear $V_{y(ind)}$ (lbs)	Slope θ_z (Rad.)
0.000	0.00000	3.350E+06	-33027.667	0.001192

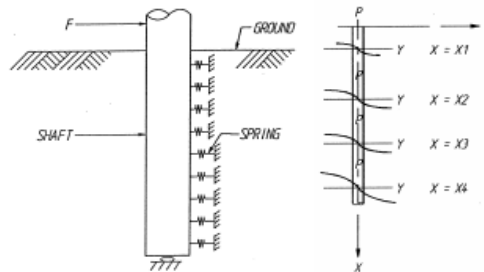
Springs Constants – Longitudinal Direction

$$\begin{aligned}
 K11 &= -V_{y(app)}/-\Delta_y = -60 \text{ kip}/-0.13576 \text{ in} = 442 \text{ kip/in} \\
 K66 &= M_{z(app)}/\theta_z = 3,350 \text{ kip-in}/0.001192 \text{ rad} = 2,810,403 \text{ kip-in/rad} \\
 K16 &= -V_{y(ind)}/+\theta_z = -33 \text{ kip}/0.001192 \text{ rad} = -27,685 \text{ kip/rad} \\
 K61 &= +M_{z(ind)}/-\Delta_y = 3,761 \text{ kip-in}/-0.13576 \text{ in} = -27,703 \text{ kip-in/in}
 \end{aligned}$$

$$\begin{Bmatrix} P_x \\ P_x \\ V_y \\ V_z \\ M_x \\ M_y \\ M_z \end{Bmatrix} \begin{Bmatrix} K22 & 0 & 0 & 0 & 0 & 0 \\ 0 & 442 \frac{\text{kip}}{\text{in}} & 0 & 0 & 0 & -27,685 \frac{\text{kip}}{\text{rad}} \\ 0 & 0 & K33 & 0 & K34 & 0 \\ 0 & 0 & 0 & K55 & 0 & 0 \\ 0 & 0 & K43 & 0 & K44 & 0 \\ 0 & -27,703 \frac{\text{kip-in}}{\text{in}} & 0 & 0 & 0 & 2,810,403 \frac{\text{kip-in}}{\text{rad}} \end{Bmatrix} \times \begin{Bmatrix} \text{Disp.} \\ \Delta x \\ \Delta y \\ \Delta z \\ \theta_x \\ \theta_y \\ \theta_z \end{Bmatrix} = \begin{Bmatrix} \text{Force} \\ P_x \\ V_y \\ V_z \\ M_x \\ M_y \\ M_z \end{Bmatrix}$$

Method III – Non-Linear Springs

A finite element model may use non-linear springs based on PY curves to represent foundation response as shown in PY curves graph the relationship between the lateral soil resistance and the associated deflection of the soil. Generally, P stands for a force per unit length (of pile) such as kips per inch. Y is the corresponding horizontal deflection (of pile) in units such as inches.



Pile Model Using a Set of Non-linear PY Curves

Figure 7-B3-1

Node placement for springs should attempt to imitate the soil layers. Generally, the upper $\frac{1}{3}$ of the pile in stiff soils has the most significant contribution to the lateral soil reaction. Springs in this region should be spaced at most 3 feet apart. Spacing of 2.5 feet has demonstrated results within 10% of LPILE output moment and shear.

Springs for the lower $\frac{2}{3}$ of the pile can transition to a much larger spacing. Stiff foundations in weak soils will transfer loads much deeper in the soil and more springs would be sensible.

Transverse and longitudinal springs must include group reduction factors to analyze the structure/soil response. Soil properties are modified in LPILE to account for lateral group effects. LPILE then generates PY curves based on the modified soil properties and desired depths. See [Section 7.2.5](#) for group effects.

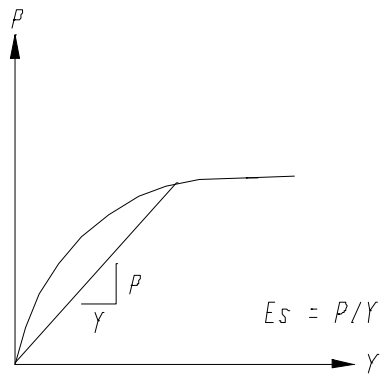
FEM programs will accept non-linear springs in a Force (F) vs. Deflection (L) format. P values in a PY curve must be multiplied by the pile length associated with the spring in the FEM. This converts a P value in Force/Length units to Force. This cannot be done during a response spectrum analysis with some FEM programs.

Soil Modulus - ES

Soil Modulus is defined as the force per length (of a pile) associated with a soil deflection. As shown in [Figure 7-B3-2](#), ES is a slope on the PY curve or P/Y . ES is a secant modulus since the PY relationship is nonlinear and the modulus is a constant. The units are F/L per L or F/L^2 , such as kips per square inch.

Subgrade Modulus - kS

A closely related term is the Subgrade Modulus (or Modulus of Subgrade Reaction) provided in a geotechnical report. This is defined as the soil pressure associated with a soil deflection. The units are F/L^2 per L or F/L^3 , such as kips per cubic inch.



Secant Modulus Illustration
Figure 7-B3-2

7.99 References

1. AASHTO (2011) "AASHTO *Guide Specification for LRFD Seismic Bridge Design*," American Association of State Highway and Transportation Officials, Washington, D.C.
2. ACI (2011) "*Building Code Requirements for Structural Concrete and Commentary*," American Concrete Institute, Farmington Hills, MI.
3. AISC (2010) "*Specifications for Structural Steel Buildings*" ANSI/AISC Standard 360-10, American Institute of Steel Construction, Chicago, IL.
4. AISI. American Iron and Steel Institute.
5. Caltrans. (2008). Memo to Designers 3-1 Deep Foundations, California Department of Transportation, Sacramento, CA.
6. Hannigan, P. J., Goble, G.G., Likins, G.E., and Rausche, F. (2006). "Design and Construction of Driven Pile Foundation," FHWA NHI-05-042, Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., Vol. I.
7. Roeder, C.W, Lehman, D.E.(2012) Initial Investigation of Reinforced Concrete-filled Tubes for use in Bridge Foundations, Report No. WA-RD 776.1, Washington State Transportation Center (TRAC), University of Washington, Seattle, WA.
8. Roeder, C.W., Lehman, D.E., and Bishop, E. (2010) "*Strength and Stiffness of Circular Concrete-filled Tubes*," ASCE, Journal of Structural Engineering, Vol 136, No 12, pgs, 1545-53, Reston, VA.
9. Roeder, C.W, Lehman, D.E., and Thody, R. (2009) "*Composite Action in CFT Components and Connections*," AISC, Engineering Journal, Chicago, IL.
10. Stephens, M.T, Lehman, D.E, and Roeder, C.W. (2016) "*Concrete-Filled Tube Bridge Pier Connections for Accelerated Bridge Construction*." California Department of Transportation, Report Number CA15-2417.

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