18. Substructures

This chapter presents DOT&PF design information on substructure elements, which supplement the *AASHTO LRFD Specifications*. Section 11.6 of the *Alaska Bridges and Structures Manual* presents DOT&PF criteria for the selection of substructure components within the context of structure-type selection.

18.1. Abutments/Wingwalls

Various types of abutments are available to support the bridge superstructure. An abutment may include an end diaphragm, a stem wall, pile cap beam, backwall, and wingwalls.

In addition to vertical support, abutments provide lateral support for fill material on which the roadway rests immediately adjacent to the bridge.

Abutments are generally cast-in-place, reinforced concrete and founded on spread footings, drilled shafts, or driven pile footings, as appropriate for the site.

18.1.1. General Abutment Design and Detailing Criteria

The following applies to the design and detailing of abutments:

1. **Minimum Thickness**. The minimum allowable wall thickness is 12 inches.
2. **Abutment Slope**. The preferred abutment slope is 2H:1V measured normal to the centerline of bearing. This slope may sometimes be steepened to a minimum of 1½H:1V to avoid the need for a deeper prestressed concrete girder.
3. **Terminology**. An “end diaphragm” is always integral with the superstructure. The term “backwall” only applies where the wall is part of a seat abutment and, therefore, not integral with the superstructure.

18.1.2. Semi-Integral Abutments

The semi-integral abutment is DOT&PF’s preferred abutment configuration. Figures 18-1 and 18-2 present typical designs — one founded on piles and the other on a spread footing. For this type of abutment, the integral end diaphragm is cast around the girder ends and attached to the slab, but separated from the cap.

Thermal movements and live load rotations are accommodated through the bearings, girders, end diaphragms, and approach slabs.

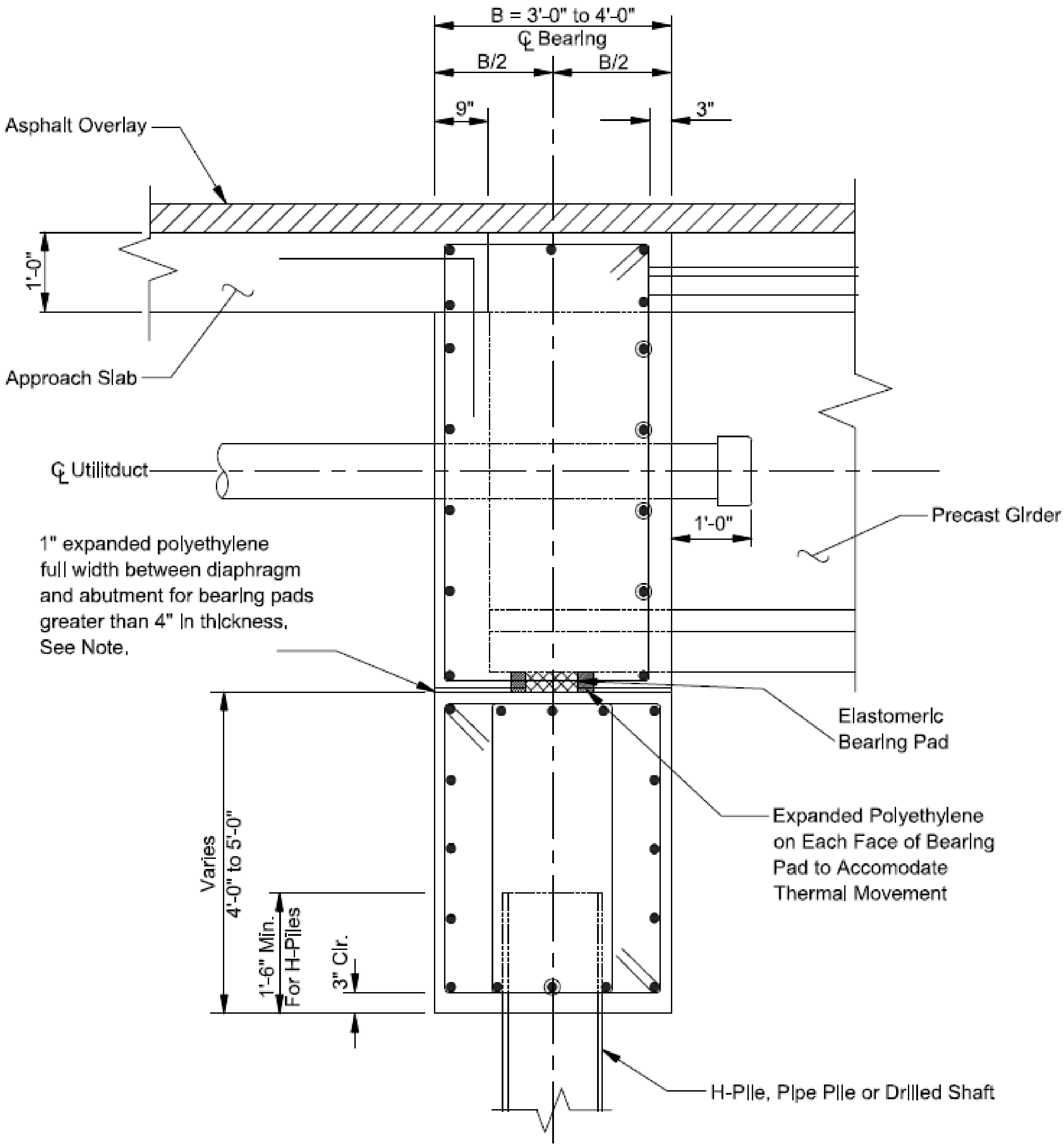
Semi-integral abutments allow diaphragm movement and rotation through the detailing of the bearing or connection of the girder, diaphragm, and the cap as either:

- **Fixed**. A pinned connection (free to rotate, fixed against translation).
- **Free**. A roller connection (free to rotate, free to translate).

Usually in a single-span bridge, the design fixes one end (typically, the downhill end) while the other end is free to translate. In a multi-span bridge, both abutments will usually be free with fixity provided at the pier(s).

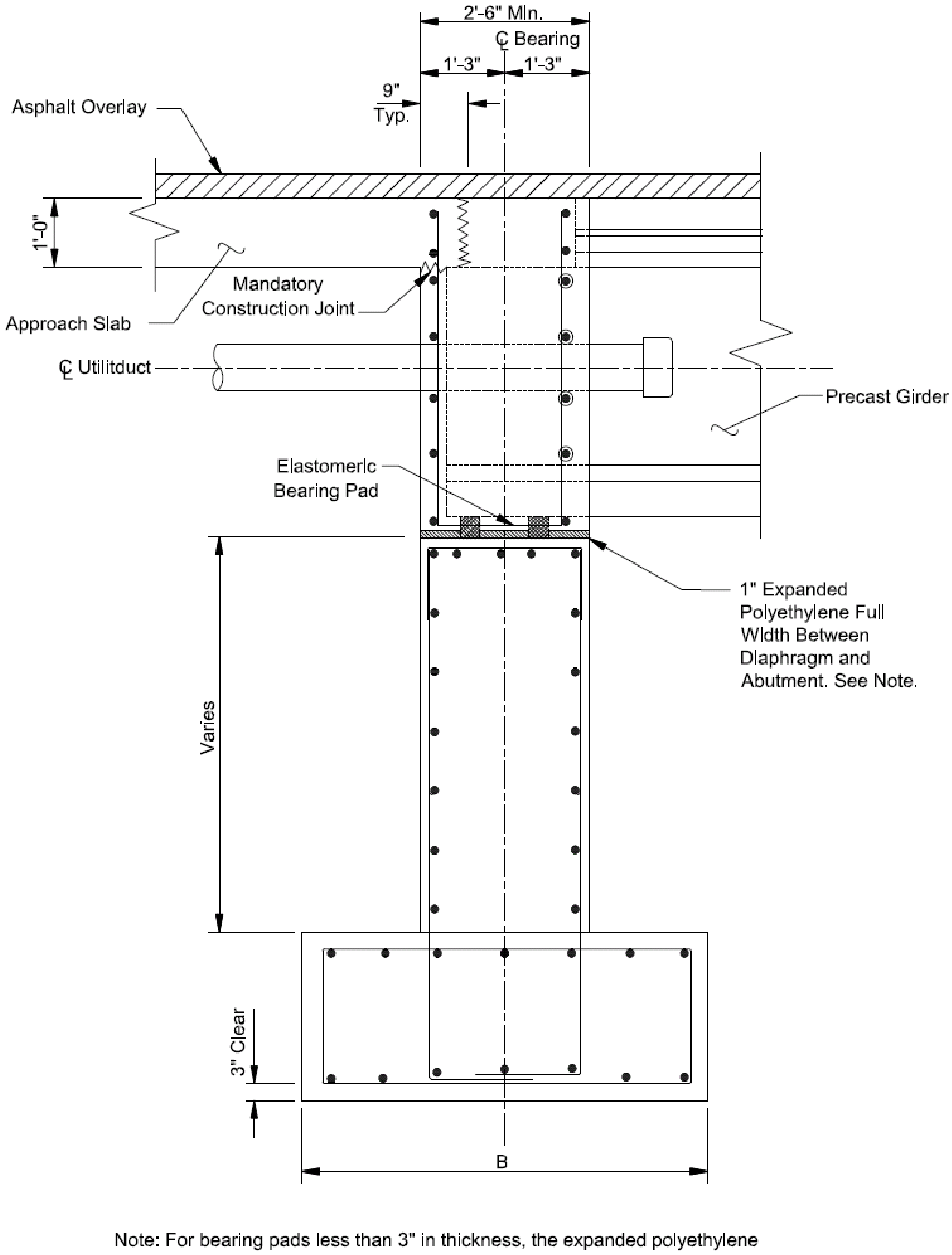
In addition, the following applies to the design of semi-integral abutments:

1. **Fixed End**. Assume a pinned (free to rotate, fixed against translation) end for the structural design of the superstructure.
2. **Diaphragm Width**. Typically, the end diaphragm width is the same as the pile cap beam but shall be a minimum of 30 inches. Commonly used pile sizes typically result in either 36-inch or 48-inch wide diaphragms for semi-integral abutments.



Note: For bearing pads less than 3" in thickness, the expanded polyethylene is the same thickness as the bearing pad.

Figure 18-1
Typical Semi-Integral Abutment
(On Piles)



Note: For bearing pads less than 3" in thickness, the expanded polyethylene is the same thickness as the bearing pad.

Figure 18-2
Typical Semi-Integral Abutment
(On Spread Footing)

18.1.3. Seat Abutments

Figure 18-3 presents a typical seat abutment. Seat width will generally be controlled by seismic design requirements, but in no case shall the seat width be less than 30 inches.

18.1.4. MSE-Wall Abutments

DOT&PF uses two basic types of abutments with MSE walls:

- **"True Abutment"**. An abutment supported by an MSE wall, in which the wall rests on a spread footing atop the reinforced earth. In the design of the MSE wall, consider the load from the spread footing as an earth surcharge load (ES). True abutments are generally limited to single-span bridges.
- **"False Abutment"**. A pile-supported abutment with the piles passing through the MSE embankment. Isolate the piles from the MSE backfill through sleeves to eliminate downdrag, and found the piles in the soils below the MSE wall. False abutments are typically used for multi-span bridges with a continuous superstructure.

Piles Within a False Abutment

Piles placed within the MSE backfill require special consideration. Ensure that the piles are placed prior to the construction of the wall.

As the wall is constructed, the subsoils beneath the wall and the MSE wall itself may compress. The piles, however, are rigid. The compression of the soils will induce a load into the piles due to friction. Depending on site materials, these downdrag forces can be substantial.

To reduce the friction on the piles and to mitigate the downdrag forces, place the piles in pile sleeves, or place a slightly larger corrugated pipe over the pile prior to backfilling. Fill the space between the pile and the corrugated pipe with pea gravel or similar free-draining material.

Modify the soil reinforcement when piles are located within the wall. Do not bend the soil reinforcement around the piles; the soil reinforcement must remain linear to develop its strength. Also, do not attach the soil reinforcement to the piles. Allow a reinforcement skew of up to 15 degrees from a line perpendicular to the wall face if the design accounts for this.

Reinforcing mats can be cut and skewed, but they must conform to the following:

- Do not allow single longitudinal wires.
- Reinforcing mats develop their strength from the cross wires. Provide at least two longitudinal wires to make the cross wire effective.
- Cut segments must meet minimum pull-out capacity factors of safety. Testing of cut segments is required to show that their full strength is developed.

The contractor must perform all cutting of reinforcement prior to the application of corrosion protection.

If cutting and skewing cannot resolve all conflicts, the bridge engineer may need to provide steel frames around the piles connecting straight soil reinforcing on either side of them.

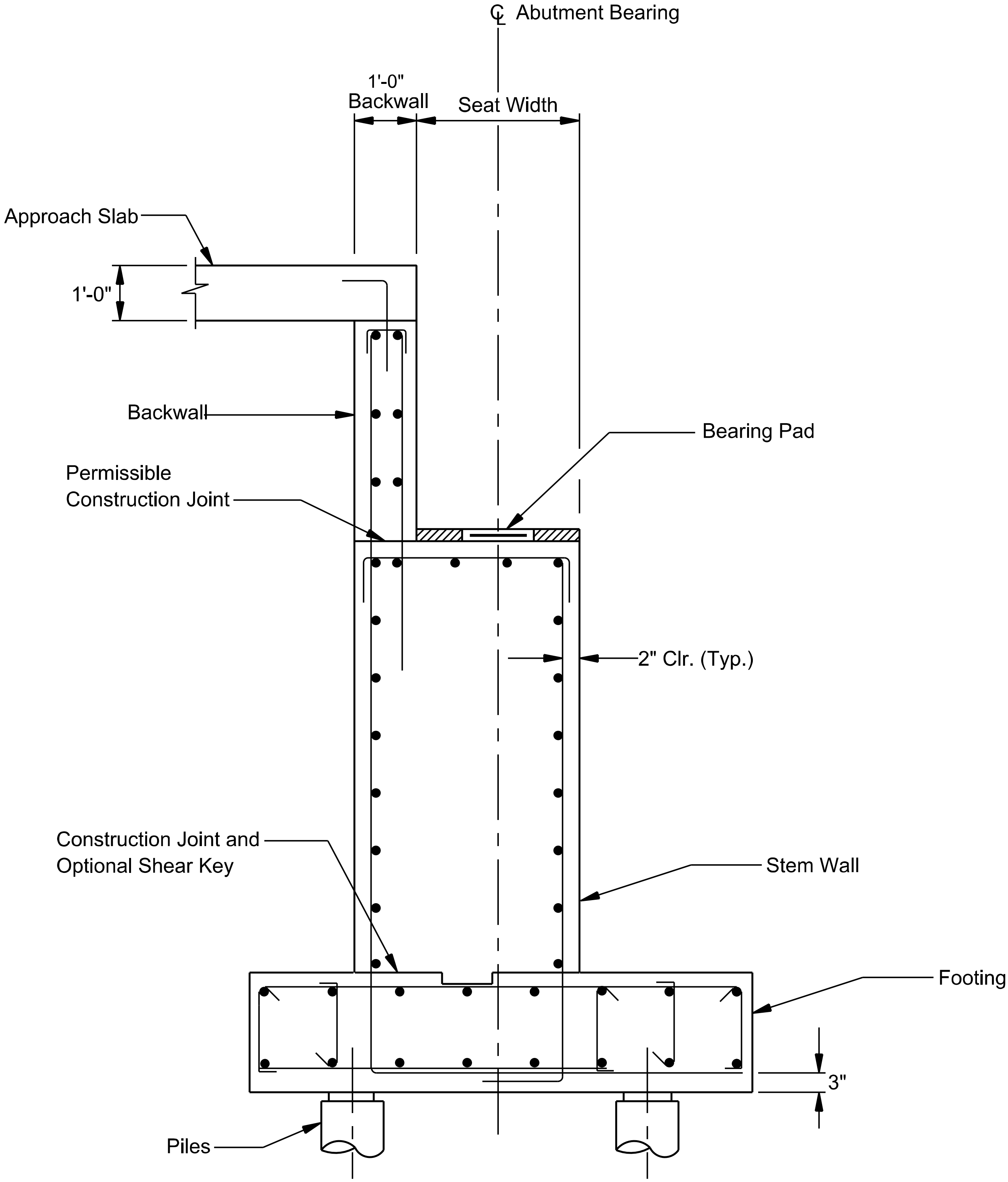
Design these frames to transfer all forces within the soil reinforcement, which must be corrosion protected. The wall supplier must detail all bridging frames in the shop drawings.

Chapter 21 discusses the use and design of MSE walls in more detail.

18.1.5. Integral Abutments

Reference: LRFD Article 11.6.2.1.

As discussed in Section 11.6.2, the Chief Bridge Engineer must approve the use of integral abutments because of their inherent incompatibility with frozen ground and extreme thermal ranges.



Note: This typical seat abutment shows piles.
Drilled shafts, H-Piles and spread footings are also options.

Figure 18-3
Typical Seat Abutment

18.1.6. Piles at Abutments

Reference: LRFD Article 10.7.

This discussion specifically addresses the use of driven piles with abutments. See Section 17.4 for additional information on piles. The following criteria apply to piles for abutments:

Number

Use the most cost-effective pile type and size, but do not use fewer than three piles to support an abutment.

Pile Type

Typically, use HP14 × 117 in soils that are not susceptible to liquefaction. Use 18-inch or larger diameter, concrete-filled pipe piles or drilled shafts in soils susceptible to liquefaction.

Pile Spacing

Pile spacing should generally not result in more than one pile per girder; however, placing a pile beneath each girder is not critical. Space the piles across the length of the abutment to help distribute abutment loads uniformly to each pile.

Cap Overhang

The minimum cap overhang is 18 inches measured from the centerline of the pile, but in all cases, the cap overhang reinforcement must be adequately developed. Hooked or headed bars may be required to develop the cap reinforcement.

Pile Loads/Forces

For the semi-integral abutment, design the end diaphragm to resist the force from the bearings and lateral earth pressure, including seismic-induced earth pressures.

Pile-Cap Connections

To allow for constructability, the pile top lateral position must have a tolerance of ±6 inches. Extend steel H-piles a minimum of 12 inches (preferably 18 inches) into the cap.

Abutments supported on pipe piles should be designed and detailed similar to pier cap beams supported on pipe pile extensions *except* that the cap beam depth may be taken greater than 125 percent of the pile diameter.

Construction

Consider the placement tolerances for all abutment types and ensure pile fit within the cap dimensions and relative to the reinforcing steel.

18.1.7. Abutment Construction Joints

To accommodate normal construction practices, the bridge engineer should detail the following horizontal construction joints in the contract documents:

1. **Semi-Integral Abutments**. Place a mandatory construction joint between the approach slab and the top of the diaphragm.
2. **Seat Abutments**. Allow a horizontal construction joint between the top of the abutment seat and the bottom of the backwall. Some expansion joint types may require another construction joint at the approach slab seat.

Planned vertical construction joints are normally associated with staged construction. Make provisions for splicing or mechanical reinforcing couplers on horizontal reinforcing steel. Vertical reinforcing steel should be at least 3 inches from the construction joint.

Show keyways or roughened surfaces consistent with the structural design of the joint.

When the joint will be exposed to public view in the finished structure, provide a chamfered groove or similar technique to hide the joint. Allow a vertical construction joint between the wingwall-abutment interface.

18.1.8. Wingwalls

Reference: LRFD Article 11.6.1.4.

Provide wingwalls of sufficient length to retain the roadway embankment and to furnish protection against erosion. Figure 18-4 illustrates the typical dimensions and grading for wingwalls.

Orientation

Standard DOT&PF practice is to use wingwalls aligned parallel to the roadway centerline and attached to the abutment cap. The outside face of the wingwall should be co-linear with the bridge edge of deck. The bridge rails or barriers are supported by and extend to the end of the wingwall.

Occasionally, site constraints will require the use of wingwalls aligned parallel to the centerline of abutment bearing ("elephant ear" wingwalls). These wingwalls are susceptible to erosion which can result in undermining of approach rail posts and edge of roadway shoulder.

Thermal movement between the approach guardrail and bridge rails may also require special attention when elephant ear wingwalls are used. Only use "flared" wingwalls in combination with box culverts.

Length

Wingwall length is determined by extending the wingwall 5 feet to 8 feet beyond the hinge point between the embankment slope and the edge of shoulder.

Do not extend wingwalls more than 20 feet behind the rear face of the abutment without special design and detailing. Consider pile-supported, unattached, or other wingwall types for lengths greater than 20 feet.

Thickness

The thickness of any wingwall should match the barrier or curb width, but must not be less than 12 inches. Typical wingwall widths are 15 inches and 18 inches.

Unattached Wingwalls

Design unattached wingwalls as retaining walls. Unattached wingwalls are generally cast-in-place concrete retaining walls. Provide an expansion joint between the unattached wingwall and abutment. See Chapter 21 for DOT&PF practices on retaining walls.

18.1.9. Drainage

Provide positive drainage as needed in the embankment behind the abutment and wingwalls by using select backfill, porous backfill, weep holes, perforated drain pipe, a manufactured backwall drainage system, or a combination of these options. Provide details of the selected drainage system on the bridge plans.

Always consider ground water levels when evaluating an appropriate drainage system. Do not install drainage systems that allow pressurized backwater to saturate the abutment backfill during high-water events.

Generally, for relatively shallow girders supported on semi-integral abutments with parallel wingwalls, or elephant ear wingwalls less than 10 feet long, select backfill and porous backfill will be sufficient to promote good drainage.

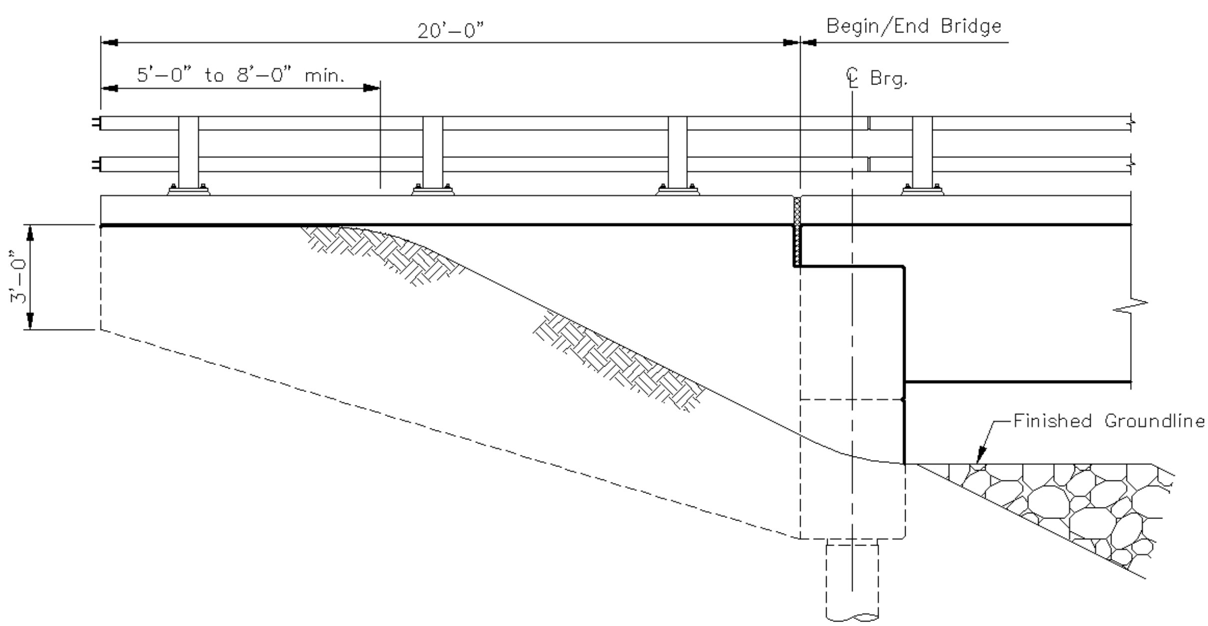


Figure 18-4
Typical Wingwall

18.1.10. Backfill

As discussed in the *AASHTO Guide Specifications for LRFD Seismic Bridge Design* Article C3.3, "Abutments as an Additional Energy-Dissipation Mechanism", the advantages of including abutments in the ERS could be offset by settlement of the fill during a seismic event. Typical DOT&PF practice does not include the abutments as part of the ERS but does require placement of Structural Fill behind the abutments to contribute to the resistance of seismic forces. Structural Fill is considered to be superior material with higher strength and stiffness than conventional roadway embankment material. For most bridges, the Structural Fill volume includes the width between the wingwalls, height from the top of superstructure or bottom of approach slab to the bottom of the abutment cap, and length of 50 feet from the beginning or end of bridges on paved roads and 30 feet for bridges on unpaved roads.

18.2. Piers

Reference: LRFD Article 11.7.

See Section 11.6.3 for DOT&PF practices on the selection of pier types.

18.2.1. Seismic Considerations

Design piers for non-seismic loads, then check them for seismic adequacy.

18.2.2. Pier Caps

Usage

In general, DOT&PF uses drop pier caps (non-integral with the superstructure) supported by pile extensions, a single column, multiple columns, or a solid pier wall. See Section 11.6.3 for more discussion.

Design

The cap depth-versus-length geometry affects the design of the pier caps. Where the distance between the centerline of the girder bearing and the column is less than approximately twice the depth of the cap, it may be appropriate to use the strut-and-tie model in LRFD Article 5.8.2 for the design of the cap; otherwise, use the sectional (beam) model for moment and shear.

Cap Width

Cap width is generally determined by adding 18 inches to the diameter of the supporting pipe piles. For caps supported on cast-in-place concrete columns or wall piers, the cap width should be 6 inches wider than the column diameter. Verify the resulting cap is sufficiently wide to accommodate the beam-seat widths dictated by seismic requirements.

Drop Caps

For crowned roadway sections, the bottom of the cap is level unless the bridge is very wide (greater than 100 feet). For superelevated cross sections, slope the bottom of the cap at the same rate as the cross slope of the top of the bridge deck. For decked bulb-tee girder bridges, slope the top of the cap parallel to the roadway crown or superelevation. Other girder types are set plumb. Thus, step down the tops of drop caps to account for elevation differences between girders with conventional cast-in-place decks.

18.2.3. Pier Cross Sections

The following summarizes DOT&PF practices for the cross section of piers.

1. **Round Columns**. The standard column has a minimum diameter of 2 feet with incremental increases in diameter of 6 inches. The preferred diameters for pile extension piers are 2 feet, 3 feet, and 4 feet.

2. **Solid Walls**. The minimum thickness is 2 feet (2′-6″ for railroad crash walls), which may be widened at the top to accommodate the bridge seat where required. Axial forces in the boundary edges of wall piers subjected to seismic loads may result in out-of-plane buckling, which may lead to excessive damage and loss of vertical load carrying capacity. Out-of-plane buckling of wall piers is affected by the wall pier geometry, reinforcement ratio, in-plane inelastic seismic demands and axial loading.

   The minimum thickness of wall piers, b_w, shall be taken as the greatest of the following:

   - 2.5 feet for railroad crash walls or 2 feet for all other applications

   - If the height of the wall, h_w, is greater than 1.5 times the length of the wall, l_w, (see Figure 18-6) then the minimum wall thickness, b_w, shall be determined from Figure 18-5

   In lieu of Figure 18-5, the refined analysis for out-of-plane buckling of wall piers from Haro, A.G., Kowalsky, M.J., and Chai, Y.H. (2017) may be used to determine the minimum wall thickness, but in no case shall the wall thickness be less than 2 feet or 2.5 feet for railroad crash walls.

   Wall piers may be provided with a cap beam if wider support width is required.

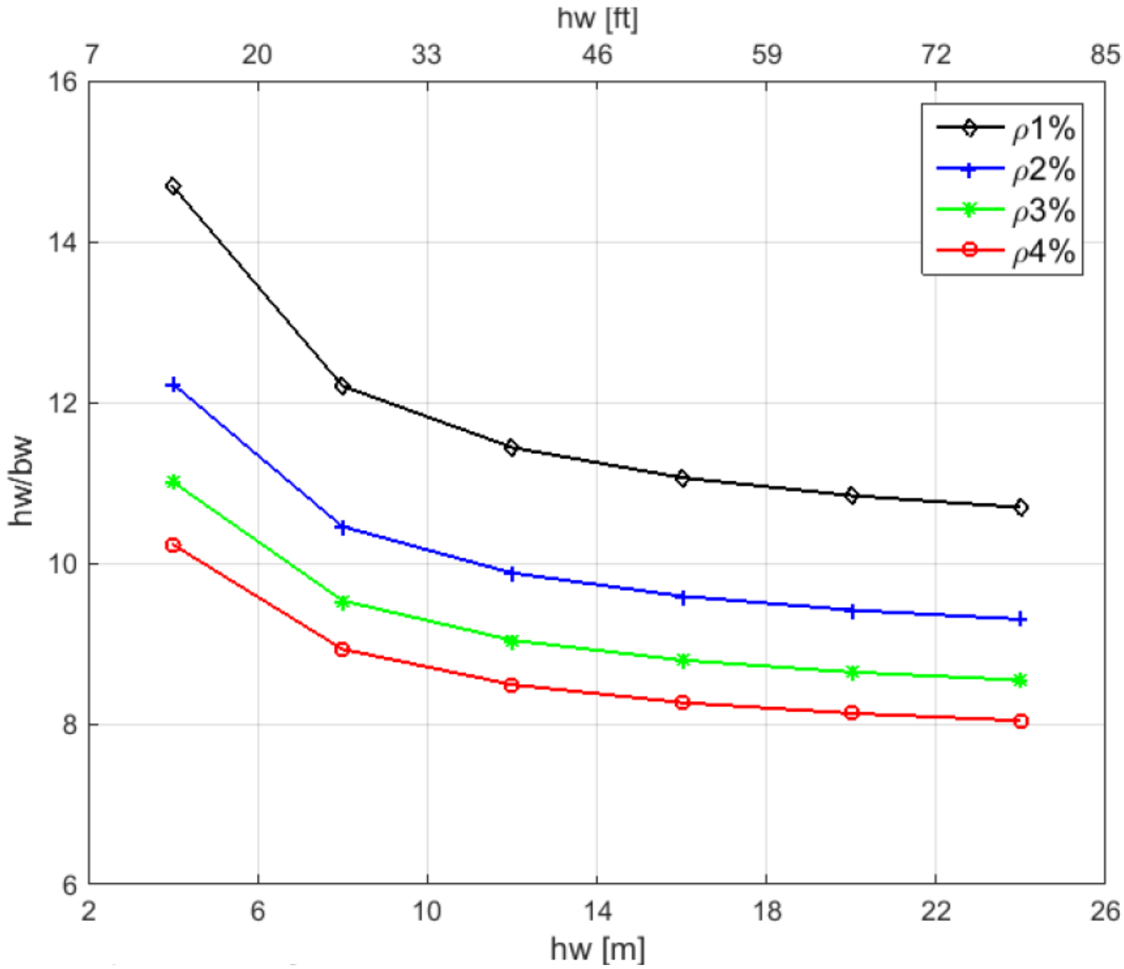


Figure 58-5
Minimum Wall Pier Thickness

18.2.4. Pier Foundations

Typical DOT&PF practice is to support single-column piers on oversized drilled shafts and to support pile bents on driven pipe piles. Multi-column or wall piers are infrequently used but may be considered where conditions warrant their use.

Enlarge the diameter of the drilled shaft relative to the column to force plastic hinging in the column and protect the drilled shaft from inelastic action. The drilled shaft diameter is typically 24 inches larger than the column diameter. Confirm that the diameters selected for the column and shaft will accommodate the overlapping reinforcing steel cages and cover requirements in both the column and drilled shaft. See Section 17.5 for a discussion on drilled shafts.

18.2.5. Column Reinforcement

Section 14.2 discusses DOT&PF practices for the reinforcement of structural concrete. This includes:

- concrete cover,
- bar spacing,
- lateral confinement reinforcement,
- corrosion protection,
- development of reinforcement, and
- splices.

The design of concrete pier columns must meet all applicable requirements in Section 14.2.

Transverse Reinforcement
Reference: LRFD Article 5.11.

General. Use spirals as transverse reinforcing steel in round columns. Allow butt-welded (electric “flash” resistance) spliced hoops in high seismic areas with radiographic testing and destructive testing.

Spiral Splices. Almost all spiral reinforcement will require a splice. LRFD Article 5.11 provides requirements for splices in spiral reinforcement. The contract documents must identify plastic hinge regions where a spiral splice is not allowed. Refer to Section 14.2.1.

Longitudinal Reinforcement

Reference: LRFD Article 5.11.

Use #8 or larger longitudinal column reinforcing bars, with #10 bars being the preferred minimum. Detail the longitudinal reinforcing steel continuous with a maximum spacing of 8 inches center-to-center.

Fully develop the longitudinal column reinforcing bars where these bars enter into the pier cap and the spread footing, pile cap, or drilled shaft. Longitudinal column reinforcing bars extend into the pier cap to be as close as possible to the top of the cap.

The preferred detail for longitudinal reinforcement is continuous, unspliced reinforcement. Provide a note on the bridge plans delineating the "no splice zones."

If longitudinal column reinforcing bars require splices, use the provisions in LRFD Article 5.11. Do not locate splices within the plastic-hinge regions of the column. (Refer to Section 14.2.1.) Use a minimum stagger of 2 feet between adjacent splices. Also stagger splices in bundled bars at a minimum of 2 feet. If epoxy-coated bars are used, specify mechanical couplers tested with reinforcing bars coated as required for the design, and the couplers must use a compatible coating.

The contractor is not permitted to change the location or type of splice from those in the contract documents unless approved by the bridge engineer.

18.2.6. Column Construction Joints

Use construction joints at the top and bottom of the column. Where columns exceed 25 feet in height, permit intermediate construction joints. Where applicable, locate all construction joints at least 12 inches above the water elevation expected during construction.

18.2.7. Solid Walls

It is acceptable to reduce the dimensions of the wall in the transverse direction by providing cantilevers to form a hammerhead pier. Figure 18-6 illustrates the typical detailing for pier wall tie bars.

18.2.8. Pipe Pile Extension Bents

Pipe pile extension bents have proven to be constructable and cost-effective, and to provide reliable performance. They are the most commonly used pier type in Alaska. Use a single row of vertical piles. Battered piles are not allowed without approval of the Chief Bridge Engineer.

Filling steel pipe piles with concrete increases the member's strength and stiffness. The concrete core also provides a means of connecting the pipe to the reinforced concrete cap beam. The moment capacity of a concrete-filled pipe pile is about three to four times that of a comparably sized reinforced concrete column.

The concrete-filled core in the pipe pile extends to a point where the moment demand is less than half of the maximum moment demand (i.e., below ground plastic hinge moment). The length of the concrete-filled core must include the effects of scour. Extend the concrete below the bottom of liquefiable soil layers into competent soil.

Extending the steel pipe pile into the concrete cap beam results in very large joint stresses and flexural demands that lead to unacceptable flexural hinging in the cap beam.

To limit the demands acting on the cap beam, the steel pipe portion of the pile is terminated several inches below the bottom of the cap beam. The resulting forces acting on the cap beam are those of the reinforced concrete core alone and the design follows the same procedure as that of a conventional reinforced concrete column-to-cap beam design.

The moment-curvature relationship of concrete-filled steel pipe piles can be calculated in the same manner as that used for conventional reinforced concrete (i.e. equilibrium and strain compatibility).

Use the expected material properties of the steel shell in the seismic analysis and design of concrete-filled steel pipe piles.

The expected material properties for commonly used steel pipe piles are provided in Table 18-1, in which D is the outside diameter of the pipe and t is the pipe wall thickness. For spiral welded pipe piles fabricated in accordance with the DOT&PF Special Provisions, use the properties of ASTM A709 Grade 50T3.

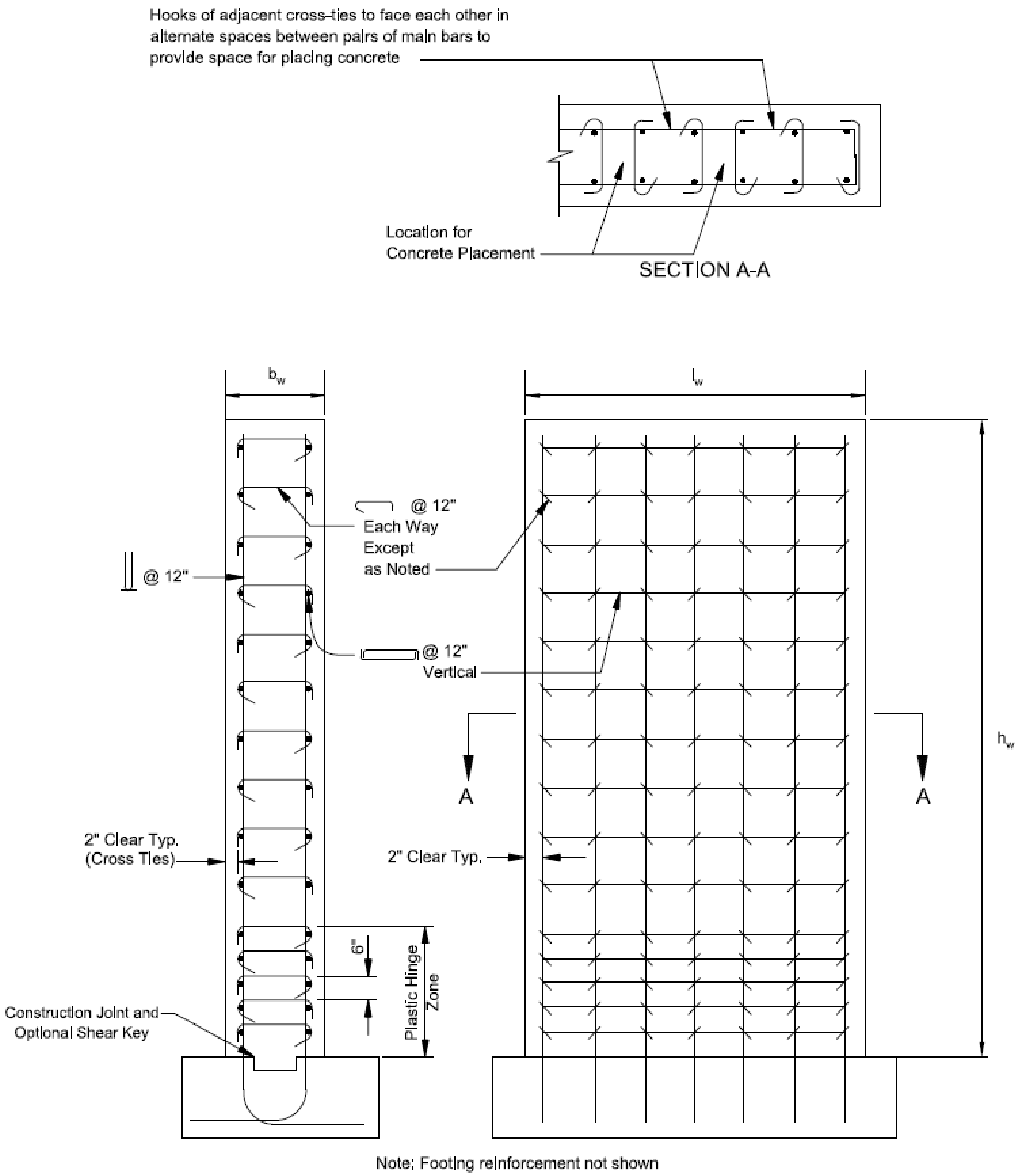


Figure 18-6
Pier Wall Tie Bars

The ultimate curvature of a concrete-filled pipe shall be based upon the reduced ultimate tensile strain of the steel pipe.

The onset of pipe wall buckling strain may be used to evaluate expected pipe performance under the design seismic event.

The Mander model can be used as the basis of generating the stress-strain relationship of the confined concrete core but the maximum confined concrete compressive strain shall not be taken greater than 0.02 (Brown at al 2013).

Table 18-1
Seismic Steel Pipe Pile Material Properties

Property	Notation	API 5L X52 PSL 2	ASTM A709 Grade 50T3	ASTM A53 Grade B
Specified minimum yield stress (ksi)	f_y	52.2	50	35
Expected yield stress (ksi)	f_{ye}	60	55	55
Expected tensile strength (ksi)	f_{ue}	78	78	78
Expected yield strain	ε_{ye}	0.0021	0.0019	0.0019
Onset of strain hardening	ε_{sh}	0.015	0.015	0.015
Onset of pipe wall buckling strain	ε_{cr}	0.022-(D/t)/9000	0.022-(D/t)/9000	0.022-(D/t)/9000
Reduced ultimate tensile strain	ε^R_{su}	0.026	0.026	0.026
Ultimate tensile strain	ε_{su}	0.12	0.12	0.09
Overstrength factor	λ_{mo}	1.2	1.2	1.4

18.3. Cold Climate Effects on Earthquake Resisting Elements

Much of Alaska experiences prolonged periods of temperatures below -40ºF. Concrete and steel demonstrate increasing strength at decreasing temperature. While not normally problematic for most bridge members, members that are sized based upon capacity design principles (i.e., capacity-protected elements) may experience increased demands at low temperatures.

Review the historic climate data at the bridge site. Include the effects of cold climate in the design if the record low temperature at the site is less than -20ºF.

If the adjoining capacity-protected member is insulated from severe temperature effects (e.g. buried footing or drilled shaft) then include the cold climate effects when determining the overstrength plastic hinging moment and associated forces of the hinging element. Analyze the moment-curvature response of the hinging elements using the following material properties (Montejo et al 2008):

$$f'_{ce\text{-}cold} = 1.4 \times f'_{ce}$$
$$f_{ye\text{-}cold} = 1.1 \times f_{ye}$$
$$f_{ue\text{-}cold} = 1.1 \times f_{ue}$$

where:

f'_{ce} = expected concrete compressive strength
f_{ye} = expected yield stress of steel
f_{ue} = expected tensile strength of steel

If both the hinging element and the capacity-protected element are exposed to the same temperature (e.g. column-cap connection) then the temperature related adjustments noted above would be expected to occur in both members.

Despite the increase in material strength, neither concrete nor reinforcing steel demonstrates a significant change in strain response at -40ºF. However a decrease in the analytical plastic hinge length occurs and is required to be taken as:

$$L_{p\text{-}cold} = 0.6 \times L_p$$

where:

L_p = analytical plastic hinge length

The effects of cold climate may also impact the stiffness of the supporting soils as outlined in Section 17.6.4.

18.4. References.

Haro, A.G., Kowalsky, M.J., and Chai, Y.H. (2017). *Seismic Load Path Effects in Reinforced Concrete Bridge Columns and Wall Piers – Volume 2: Out-of-Plane Buckling Instability of Pier Walls*. North Carolina State University, University of California Davis.

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