SECTION 11
ABUTMENT, PIERS, AND RETAINING WALLS

11.1 GENERAL REQUIREMENTS

This section provides design guidance and construction requirements for abutments, piers, and retaining walls. Abutments and piers support bridge superstructures, whereas retaining walls function primarily as earth retaining structures but can serve a dual purpose as an abutment.

11.2 CODE REQUIREMENTS

The design of abutments, piers, and retaining walls shall be in accordance with AASHTO, this BDM, the Geotechnical Design Manual, and current Staff Bridge Worksheets.

AASHTO Section 11

11.3 ABUTMENTS

CDOT permits the following abutment types:

- Integral
- Semi-integral
- Tall Wall
- Seat Type
- Geosynthetic Reinforced Soil (GRS)
- Other, (i.e., semi-deep, exposed multi-column in front of a retaining wall, integral on sheet piling) with approval from Unit Leader in coordination with Foundations SMEs.

Abutments shall be designed for all applicable AASHTO load combinations. Loads from the girders shall be applied at the centerline of bearing and can be assumed continuous over the centerline of foundation elements. Dynamic load allowance shall be included in the design of the bearing cap and diaphragm but not the foundation elements. The Designer need only apply one-half of the approach slab dead load to the bearing cap. Live loading on the approach slab may be ignored for abutment loading purposes since bridge live loads will generally control. If no approach slab is provided, equivalent soil heights for live load surcharge of varying abutment heights shall be as provided in AASHTO. Joints shall not be provided in the abutments as required per AASHTO 11.6.1.6 because CDOT has not had an issue with abutments without them. If the height of the bearing cap varies more than 18 in. from each end, the Designer should slope the bottom of the cap.

AASHTO Table 3.11.6.4-1, 3.11.6.5

When Strut & Tie Models are used for the design, they must be shared with the design checker to obtain concurrence on the models. Refer to Section 37.5 of this BDM for more details.

Pile and drilled shaft spacing and minimum clearances shall be per AASHTO. The minimum foundation element length shall be 10 ft. below bottom of bearing cap.

AASHTO 10.7.1.2, 10.7.1.3, & 10.8.1.2

The Structure Selection Report shall document the recommended type of abutment selected for the project.

11.3.1 Integral Abutments

Integral abutments are preferred for most bridges due to the elimination of expansion joints and bearings at supports, simplified construction, and reduced maintenance costs. Integral abutments rigidly attach both superstructure and supporting foundation elements so that the thermal translation and girder end rotations are transferred from the superstructure through the abutment to the foundation elements. The superstructure and substructure act as a single structural unit by distributing system flexibilities throughout the soil.

Use integral abutments where continuous structure units are shorter than the lengths shown in Table 11-1 (from FHWA Evaluation of Integral Abutments Final Report, 2006). A bridge unit includes one or more spans and can be separated at a pier from an adjacent unit by an expansion device or a fixed gap.

Table 11-1: Limiting Structure Lengths for Integral Abutments

Girder Material	Maximum Unit Length
Steel	460 ft.
Cast-in-Place Concrete	460 ft.
Precast and Post Tensioned Concrete	As calculated (460 ft. max.)

Assumptions:

- Point of zero movement is located at the midpoint of the bridge unit.
- Maximum unit lengths shown are per current research recommendations.

In addition to meeting the maximum unit length restrictions in Table 11-1, the total factored movement in one direction, expanding or contracting, at the integral abutment from the point of zero movement shall be 2 in. or less. The total factored movement shall include temperature, creep, shrinkage, and elastic shortening. The temperature range used to determine the movement shall be per Section 14 of this BDM and AASHTO. Assume a base uniform temperature of 60° in calculating the directional movement toward each abutment. ***AASHTO 3.12.2***

With Unit Leader approval, greater unit lengths may be used if analysis shows that abutment, foundation, and superstructure design limits are not exceeded, and that the expansion joint can accommodate movement at the end of the approach slab. Include an analysis backing up the decision with the design calculations for the structure. The Structure Selection Report shall include a discussion of this approach. CDOT has successfully used longer unit lengths on integral bridges of 1,000 ft. (for the Vasquez over Colorado Blvd bridge) by using a finger plate expansion device. Unit lengths when using a 0-4 in. strip seal shall be limited to 800 ft.

Do not use integral abutments when a straight-line grade between ends of a unit exceeds 5 percent. Research shows that the presence of high grades tends to lock up one end, thereby causing higher movements on the other.

During design, a pinned connection is assumed to develop between the pile cap and foundation element to allow the transfer of vertical and shear loads into the foundation element. If a pin does not develop, a fixed or partially fixed condition will be present, which can cause cracking in the deck and girders due to the developed moment from lack of girder rotation.

The preferred pile orientation is to align the weak axis of the pile with the centerline of abutment. The Designer should use the detail shown on Figure 11-1. Weak axis bending generates less resisting force in the piles from unintended frame-action with the superstructure and better accommodates bridge displacements, when compared with strong axis bending. A single row of piles shall be used with integral abutments.

To increase pile flexibility, the Designer may use the details shown on Figure 11-1 and shall determine the pile depth to establish stability. If oversized holes are used, the length shall be determined by the design and the hole shall have a minimum diameter of pile d + 1 ft., where "d" is pile depth. This detail increases the depth to point of fixity, thereby decreasing pile stiffness. Assume the point of fixity for laterally loaded piling as either the location of zero movement or location of maximum moment. The pile should extend a minimum length of 10 ft. beyond the prebore/pipe and through the overburden until stability is achieved. Design the single row of piles as an axial loaded beam-column interaction. Check steel H-piles for lateral stability and buckling capacities. Ignore soil confinement to the full depth of estimated scour or limits of pea gravel fill when not in a scour situation. However, the soil confinement of pea gravel may be considered when the designer needs the extra lateral stability that it provides, either to reduce the pile length or to avoid upsizing to a larger pile size. If the soil confinement of pea gravel is considered and if project specific geotechnical information is not available, the designer may use the following parameters: k = 300 pci, Φ = 40°, and γ = 95 pcf. Consider a semi-integral abutment configuration or seat type abutment if there is uncertainty about the development of a pin, insufficient flexibility, or if integral abutment design criteria cannot be met.

Drilled shafts may be used for integral abutments provided a pin detail such as that shown on Figure 11-2 is specified at the top of caisson. Extending fully developed drilled shaft reinforcing around the perimeter into the bearing cap prevents a pin from forming and is not permitted. Design dowels connecting the drilled shaft to the bearing seat for seismic loading.

To ensure that girder ends will rotate during the deck pour, the Designer shall add a note to the plans requiring the Contractor to pour the deck within two hours of the integral diaphragms.

The depth of the integral abutment, measured from top of deck to bottom of pile cap, shall typically be less than or equal to 13 ft. The maximum pile cap depth shall be less than or equal to 6 ft. and the minimum shall be 3.5 ft. These maximum limits prevent framing action on an integral abutment from occurring and ensure it acts like the intended pin by controlling bending and torsional forces. Designs that require greater abutment depth will need a special design with considerations for torsional and passive earth pressure bending forces.

The bottom of the bearing cap shall be embedded 1.5 ft. minimum into the embankment and provide 2 ft. minimum from the top of the embankment to the bottom of the girder. If the bridge is curved, the maximum degree of curvature shall be less than or equal to 5°.

Skewed bridges induce biaxial bending into the foundation elements from passive soil pressure. Unless otherwise approved by Unit Leader, limit skew angles to 30° or less. The Designer shall also include in the analysis all forces that rotate the structure.

On skewed bridges, the Designer shall provide 3 in. minimum clearance from the girder flanges to the back face of abutment. If sufficient clearance is not provided, the flange shall be coped or the abutment width increased. The coping shall parallel the centerline of abutment and not extend across the girder web.

For pre-tensioned or post-tensioned concrete bridges, use methods to increase foundation flexibility when the girder contraction due to elastic shortening, creep, shrinkage and temperature fall exceeds 1 in. Methods include temporarily sliding elements between the diaphragm and bearing cap, details that increase the foundation flexibility, or other details approved by the Unit Leader. Take steps to ensure that the movement capability at the end of the approach slab is not exceeded.

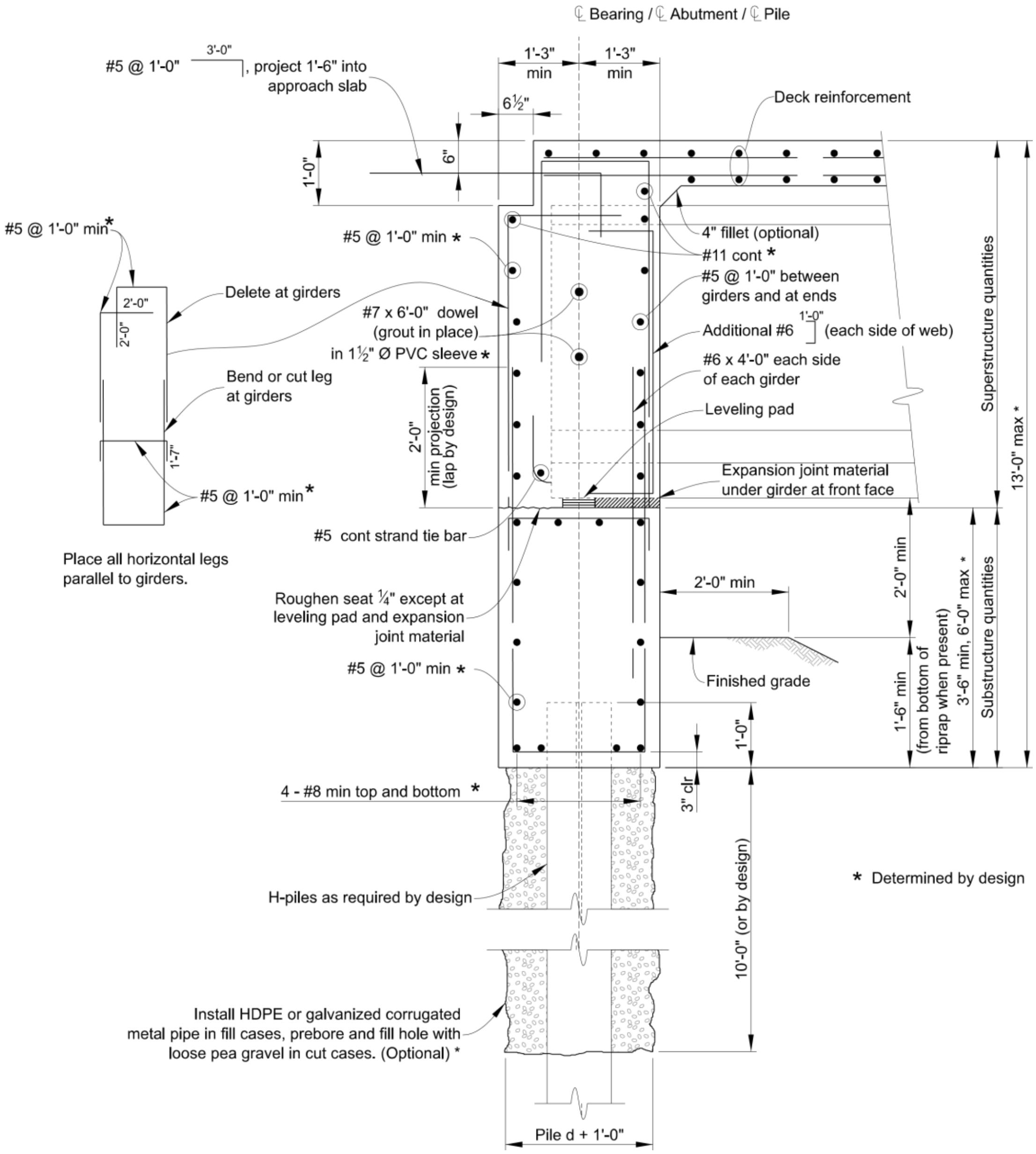


Figure 11-1: Integral Abutment on H-Piles

<u>Notes:</u>

1. All abutment and wingwall concrete shall be Class DF or D (Bridge).
2. Extend strands, per design, from the bottom of precast sections into the abutment. See Staff Bridge Worksheets.
3. Anchor the bottom of steel girder sections to the abutment with studs, bearing stiffeners, anchor bolts, or diaphragm gussets.
4. Pour the deck and portion above the bearing seat within 2 hours of each other.

5. Reinforcing steel shall be determined by design.
6. All reinforcing shall be epoxy coated or corrosion resistant.
7. Place all horizontal reinforcement legs above the bearing seat parallel to girders.
8. For integral abutments on drilled shafts, height of gap between top of caisson and bottom of diaphragm shall be verified to ensure that girder rotation will not cause the gap to close.
9. Use a leveling pad designed per Section 14.5.7 of this BDM on integral type abutments.
10. For thermal stress relief, H-Pile should have the weak axis aligned with centerline of abutment. Strong pile axis alignment is allowed provided thermal modeling with a refined method of pile-soil interaction analysis to determine actual movement is used and full thermal movement is accommodated.
11. Include the cost of pipe (CMP/HDPE), prebore, and fill material inside pipe (pea gravel or alternative approved by Unit Leader) in the work.
12. The field splice weld zones defined in Section 10.5.4 of this BDM shall be noted in the plans.
13. Grout #7 Bars into the PVC sleeve prior to the diaphragm pour. The girder worksheet should show the cast in PVC sleeve instead of a coil tie.

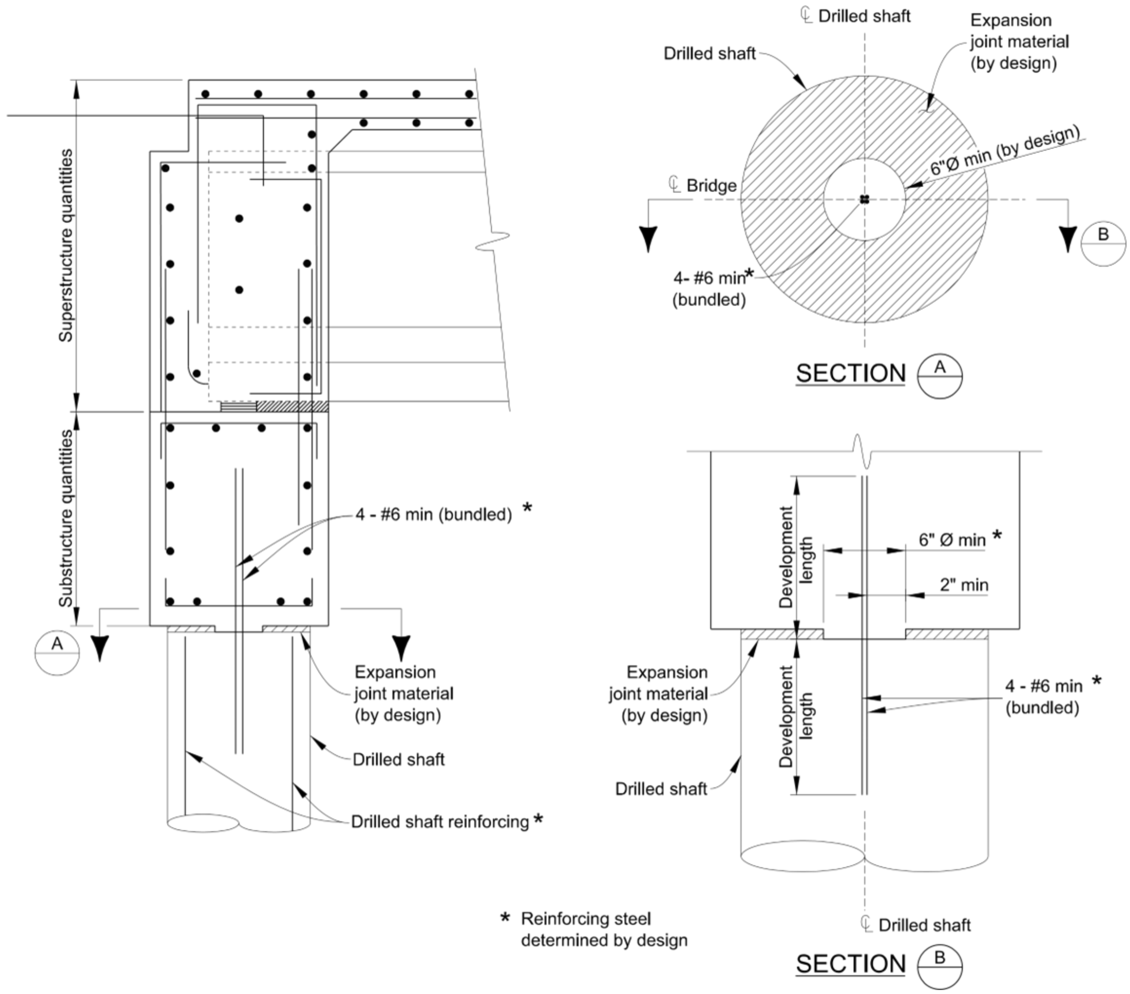


Figure 11-2: Integral Abutment on Drilled Shafts

(For details of reinforcement, refer to Figure 11-1. See Notes 1–13 with Figure 11-1.)

11.3.2 Semi-integral Abutments

Semi-integral abutments are like integral abutments because both eliminate the expansion joints at supports and encase the girder ends in concrete. The difference is that the pin for a semi-integral abutment is located at the top of bearing seat via a bearing device and the foundation element connection at the bottom of bearing cap is fixed. The bearings accommodate the rotational and horizontal movements. Using spread footings, footings on piles or drilled shafts, multiple rows of piles, or drilled shafts can establish abutment fixity.

When semi-integral abutments are used, intermediate shear blocks between girders or end blocks beyond the edge of deck shall allow a means for lateral load distribution to the substructure. If a shear block is not practical, use anchor bolts with a sole plate. The Designer shall provide an area to allow for jacking the superstructure and bearing replacement per Section 14.5.6 of this BDM.

Figure 11-3 and Figure 11-4 show semi-integral abutments on drilled shafts.

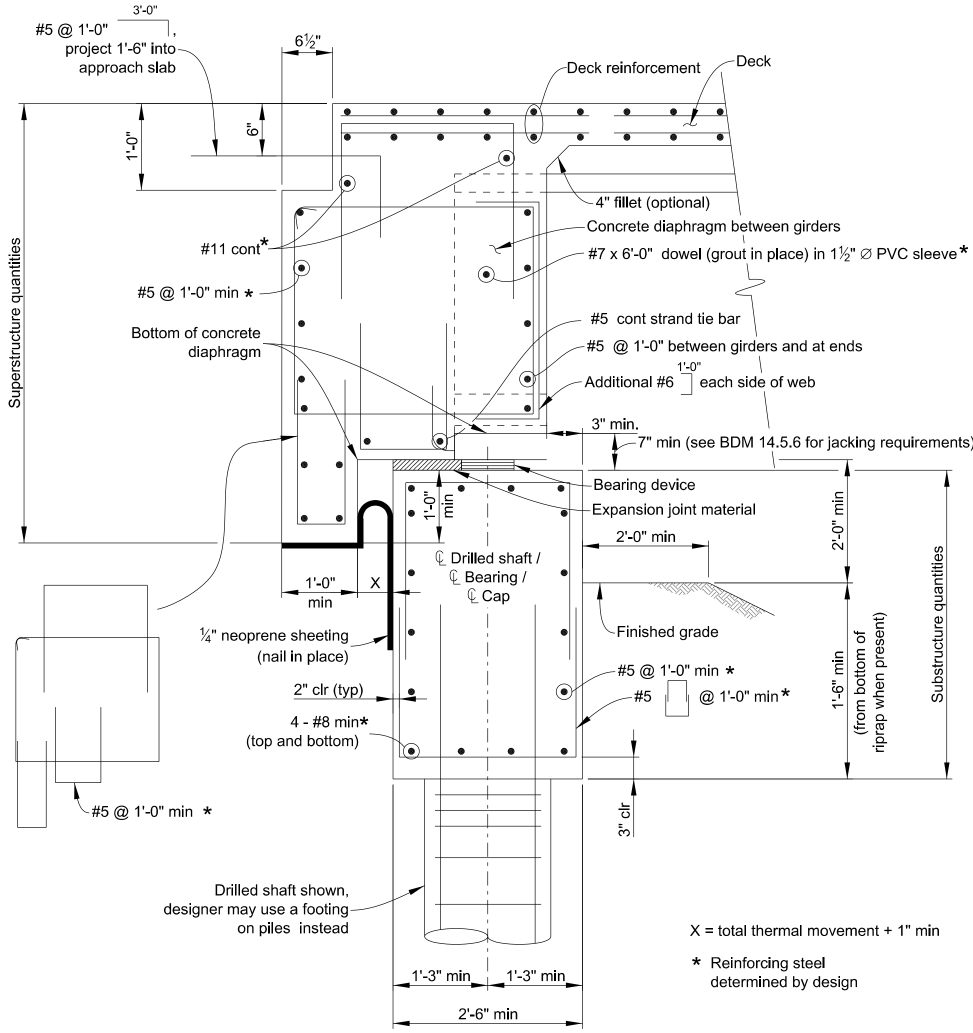


Figure 11-3: Semi-Integral Abutment (Alternative 1)

<u>Notes (For Figures 11-3 & 11-4):</u>

1. All abutment and wingwall concrete shall be Class DF or D (Bridge).
2. Extend strands, per design, from the bottom of precast sections into the abutment. See Staff Bridge Worksheets.
3. Anchor the bottom of steel girder sections to the abutment with studs, bearing stiffeners, anchor bolts, or diaphragm gussets.
4. Pour the deck and portion above the bearing seat within 2 hours of each other.
5. Reinforcing steel shall be determined by design.
6. All reinforcing shall be epoxy coated or corrosion resistant.
7. Place all horizontal reinforcement legs parallel to girders.
8. Provide lateral restraint with anchor bolts and/or intermediate or end shear blocks.

9. Bearings pads designed per Section 14.5 of this BDM are required for semi-integral abutment types. Leveling pads are not allowed.

10. Grout #7 Bars into the PVC sleeve prior to the diaphragm pour. The girder worksheet should show the cast in PVC sleeve instead of a coil tie.

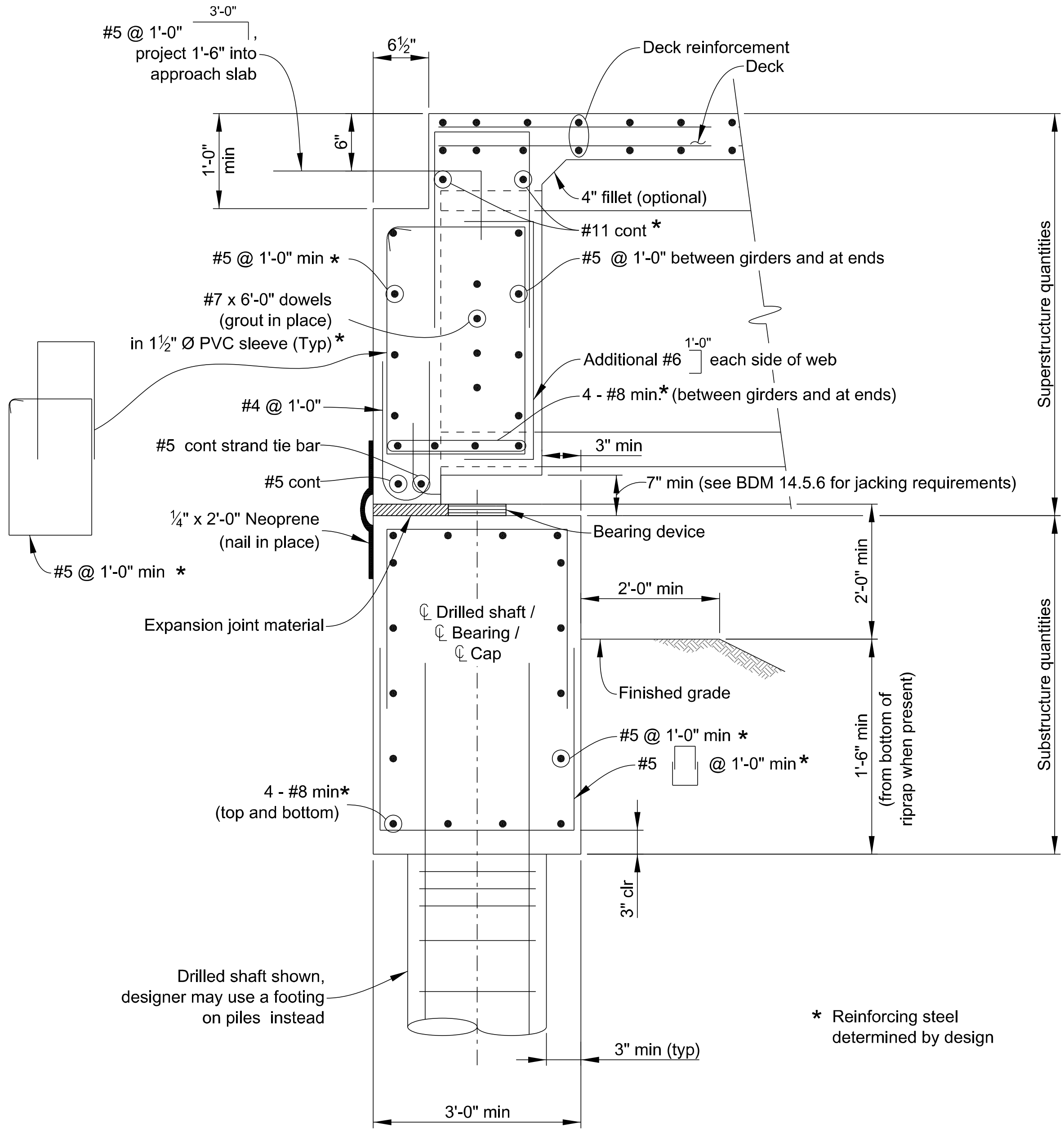


Figure 11-4: Semi-Integral Abutment (Alternative 2)

11.3.3 Seat Type Abutments

Seat type abutments have an expansion gap between the backwall and end of girders, as shown on Figure 11-5, and are typically used when large movements require a modular expansion device rather than a strip seal placed at the end of the approach slab.

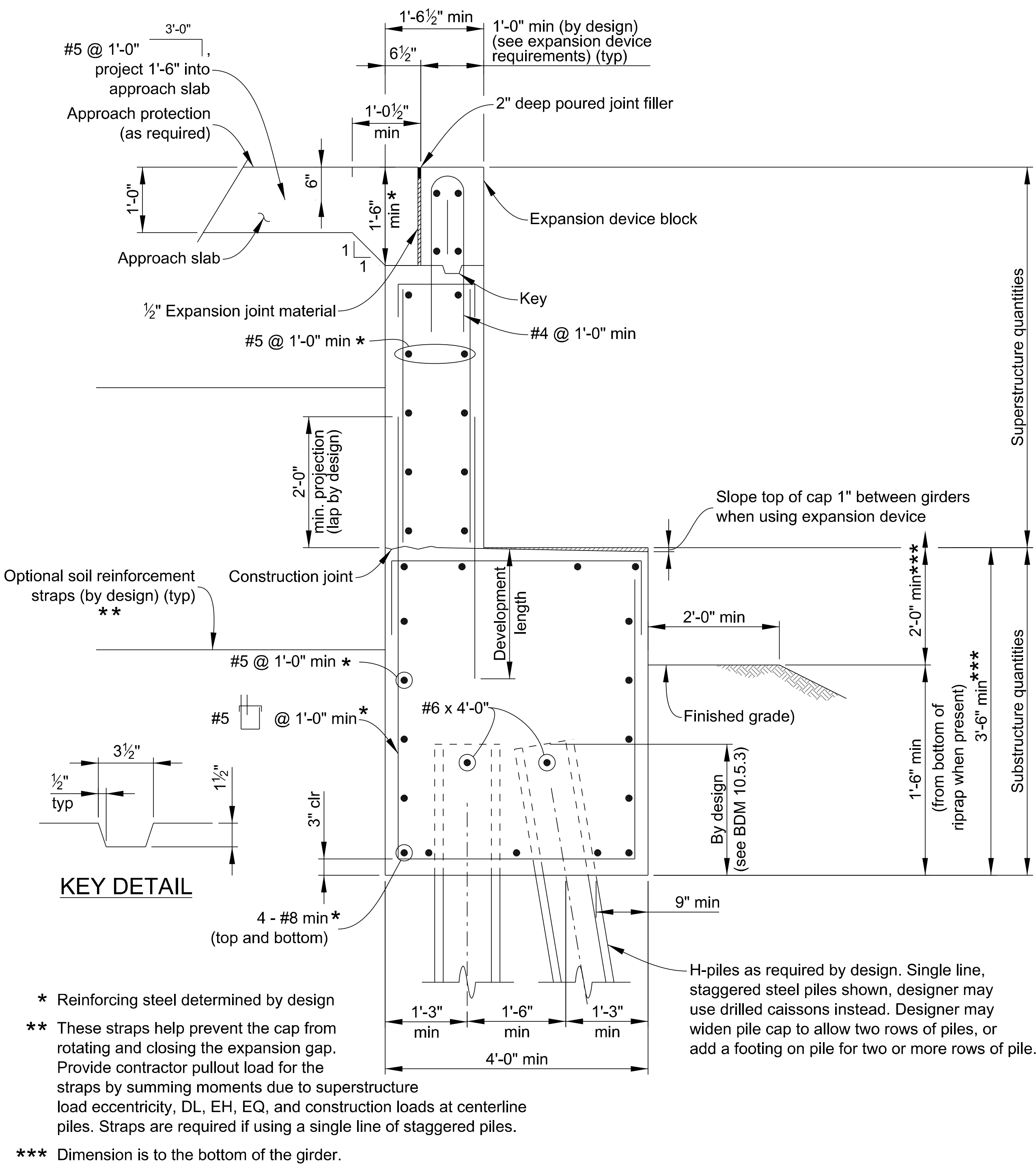


Figure 11-5: Seat Type Abutment

To provide a pinned connection between the superstructure and substructure, place the girders on bearing devices, thereby allowing rotational and horizontal movements. Using seat type abutments is discouraged due to the high maintenance costs associated with leaking expansion joints, substandard expansion device performance, and being prone to rotation and closing the expansion device.

Notes:

1. All abutment and wingwall concrete shall be Class DF or D (Bridge).
2. Reinforcing steel shall be determined by design.
3. All reinforcing shall be epoxy coated or corrosion resistant.
4. Apply an epoxy protective coating to the exposed portion of backwall, top of bearing seat, and front face of bearing cap.
5. Bearings pads designed per Section 14.5 of this BDM are required for seat type abutments. Leveling pads are not allowed.
6. To decrease a lateral load pressure on backface of abutment, a woven fabric soil reinforcing straps with 12 in. typical spacing with 3 in. low density polystyrene board or collapsible cardboard isolator may be used.

11.3.4 Tall Wall Abutments

Tall wall abutments, as shown on Figure 11-6, are used to shorten span lengths and are typically located at the approximate front toe of approach embankment. Depending on the required height, they can be founded on a single row of drilled shafts, footing on piles, or footing on drilled shafts. Due to the high cost of concrete, careful cost comparisons should be done before using this type of abutment instead of lengthening the bridge span. Architectural requirements can drive the use of this type of abutment rather than cost. The details shown in the semi-integral or seat type abutment sections can be used to connect the superstructure to the substructure.

11.3.5 Geosynthetic Reinforced Soil Abutments

Geosynthetic Reinforced Soil (GRS) is a type of retaining structure that consists of closely spaced (12 in. or less) geosynthetic reinforcement installed in granular backfill, along with a facing system approved by the Unit Leader in coordination with the Wall SMEs. GRS can be used at bridge abutments to directly support the bridge superstructure without the use of deep foundations. Geosynthetic Reinforced Soil – Integrated Bridge System (GRS-IBS) is a unique application of GRS bridge abutments. Compared to a conventional GRS abutment, which combines GRS with traditional elements of bridge design, GRS-IBS integrates the bridge approach, abutment, and superstructure to create a joint-free bridge system, without deep foundations or approach slabs.

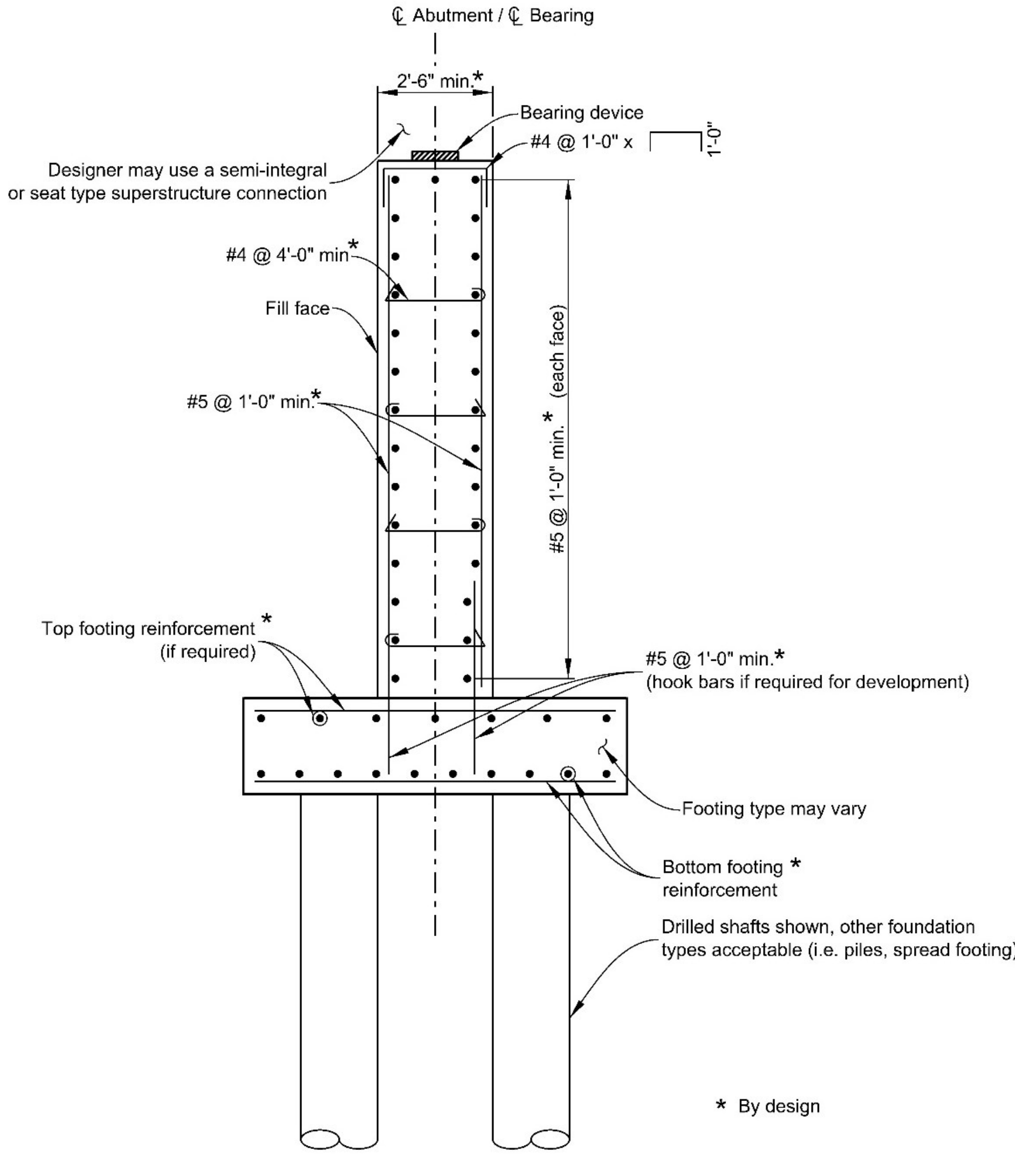


Figure 11-6: Tall Wall Abutment

The primary advantage of GRS abutments is that differential settlement between the approach fill and the bridge is minimized. The abutment fill supports the bridge, decreasing the severity of the “bump at the end of the bridge.”

Other potential advantages of GRS compared to conventional bridges supported on deep foundations include, but are not limited to:

- Decreased cost
- Accelerated construction

- Decreased reliance on specialized equipment and skilled labor for construction
- Flexible design that can be adjusted easily in the field to fit actual conditions
- Decreased maintenance due to the lack of expansion devices

GRS has been used most widely to support single-span bridges. However, the use of GRS to support continuous-span bridges is also feasible.

As discussed in the following subsections, GRS is not appropriate for sites where significant post-construction settlement or scour is expected.

11.3.5.1 Structure Selection Requirements

For bridges meeting one or more of the following structural, geotechnical, and hydraulic criteria, GRS shall be considered during the structure selection process:

a. Single or continuous span bridges where long-term foundation settlement is anticipated to be less than 1 in.
b. Single-span bridges where bearing seat elevations can be adjusted during construction to provide the required vertical clearance, accounting for the anticipated short- and long-term foundation settlement.
c. Bridges where scour is negligible or can be mitigated to a negligible level by features such as a cut-off apron wall, riprap, a reinforced soil foundation (see FHWA-HRT-11-026), or a combination thereof.

11.3.5.2 Design Criteria

GRS shall be designed in accordance with this BDM, the CDOT Geotechnical Design Manual, the FHWA publication *Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, FHWA-HRT-11-026* (FHWA, 2012), and AASHTO. The design shall be completed using LRFD methodology (see Appendix C of FHWA-HRT-11-026).

Additional geotechnical borings may be required to adequately characterize settlement of GRS abutments, particularly the settlement of the integration zone (i.e., the reinforced transition zone immediately behind the abutment). The geotechnical exploration shall be sufficient to characterize short- and long-term settlement of the GRS abutments. As appropriate, obtain relatively undisturbed thin-wall tube samples during the field investigation for consolidation testing to support the evaluation of post-construction settlement behavior.

The design of GRS abutments is an iterative procedure 11-13equireing coordination among the structural, geotechnical, and hydraulics engineers, e.g., the Geotechnical Engineer must know footing dimensions and bearing pressures to estimate settlement values. Therefore, the design disciplines should coordinate as necessary for the evaluation and design of GRS abutments.

11.3.5.3 Settlement

The tolerable settlement is defined in terms of angular distortion between supports. Without a refined superstructure and substructure interaction analysis, use the angular distortion requirements stipulated in AASHTO as a guide.

AASHTO LRFD 10.5.2.2, C10.5.2.2

The primary factor in the design of a GRS abutment is tolerable settlement, which is closely related to superstructure continuity (simple or continuous). Achieving and maintaining vertical clearance requirements must also be considered.

Settlement of GRS abutments includes short-term settlement (occurring during construction) due to the elastic compression of foundation materials and long-term (post-construction) settlement, which can occur due to time-dependent consolidation of clay soils. Settlement also includes compression of the GRS itself.

Consider the estimated short- and long-term settlement when establishing abutment girder seat elevations. Evaluate actual loads and loading sequences before and after girder placement. For phased construction, evaluate the settlement between abutment phases to determine if a closure pour is needed. Surcharging and/or subgrade improvement measures can also be used to limit the differential settlements between phases.

During construction, monitor and record settlements before and after placement of girders and deck. Provide these settlements to the Bridge Designer and Geotechnical Engineer for their information. Due to the variability in methods available for settlement monitoring, write a Project Special Provision to indicate the method to use, minimum number of points to monitor, preservation of points, reporting frequency, and measurement and payment criteria.

Uncertainty in the calculation and estimation of settlement values can contribute to the risk of unsatisfactory long-term performance of a structure. However, the risk can be managed by considering the likelihood and consequences of settlement that are greater than the estimated values. For example, a single-span bridge can tolerate more angular distortion than a continuous-span bridge. Similarly, settlement of granular soils occurs relatively quickly and could be compensated for during construction. Post-construction settlement could also be corrected by adding an asphalt overlay, but the weight of the additional overlay should be considered in the design. The risk of long-term settlement can also be reduced by surcharging or pre-loading.

11.3.5.4 Approach Slabs and Pre-camber

For single-span bridges less than 150 ft. long and continuous-span bridges with a total length less than 250 ft., CDOT prefers to use asphalt-paved approaches and no expansion joints. See Figure 11-12.

To compensate for long-term differential settlement of the abutment and the adjacent roadway, a pre-camber (increase in proposed profile to account for settlement) of 1/100 longitudinal grade is allowed at either the expansion joint at the end of the approach slab or, for bridges without an approach slab, at the back face of abutment, as shown on Figure 11-11 and Figure 11-12, respectively. The asphalt pavement camber can be accomplished with added asphalt during construction or post-construction resurfacing if the actual settlement is greater than that estimated.

The amount of pre-camber should be sufficient to compensate for long-term differential settlement and to eliminate ponding near the expansion joint, if used. Depending on the abutment height, a ½ in. to ¾ in. pre-camber has typically been specified over the approach slab length. In addition to the pre-camber, a 4 in. PVC trough (a PVC pipe cut in half and daylighted at the edge of roadway), matching the roadway cross slope, should be used under the expansion joint to capture surface run-off and reduce infiltration into the GRS.

11.3.5.5 Design and Detailing Requirements

Figure 11-7 through Figure 11-15 provide example details for GRS abutment design. The following represent additional requirements and considerations:

a. Connect the soil reinforcement directly under the girder seat spread footing to the facing with either a frictional or a mechanical connection.

b. Limit the nominal soil bearing resistance beneath the spread footing to 14,000 pounds per square foot or as stated in the project geotechnical report. Higher bearing pressures may be feasible depending on the maximum grain size of the backfill and the spacing and properties of the reinforcement.

c. Require a setback equal to H/3, with a minimum value of 3 ft., from the back of the facing to the centerline of the Service I resultant, where H is the height from the bottom of the spread footing to the roadway. See Figure 11-9 and Figure 11-10.

d. Use reinforced concrete for the girder seat and back wall.

e. Provide a GRS slope face with the reinforcement wrapped up and around the face of the individual soil layers and anchored (burrito wrap) behind the abutment and wingwalls.

f. Require a minimum vertical clearance of 2 ft. from the top of wall facing to the bottom of girder (see Figure 11-7 through Figure 11-10 and Chapter 11 in the Bridge Detail Manual).

g. Use concrete for the leveling pad at the base of the GRS abutment.

h. Provide drainage measures to reduce the likelihood of water accumulating in the GRS backfill. Appropriate drainage features could include encapsulating the top of the reinforced soil zone with dual-track seamed thermal welded geomembrane or providing an internal drainage system.

i. Provide a 3 in. minimum thick low-density polystyrene, collapsible cardboard void, or a void space with burrito wrap geosynthetic reinforcement behind the abutment back wall to isolate the back wall from the GRS backfill and to allow thermal expansion of the bridge.

j. Provide a 6 in. wide polystyrene spacer or 3 in. minimum clear space between the back of wall facing to the toe of abutment spread footing to accommodate thermal movement.

k. Extend the length of abutment soil reinforcement as a stiffness transition zone into the roadway embankment with a 1H(min):1V slope for cut or 2H(min):1V slope for fill to mitigate differential settlement caused by dissimilar foundations.

l. Use GRS abutments with a truncated base (minimum reinforcement length of 0.35DH, where DH is the design height measured from the top of the leveling pad to the roadway) and cut benches with a maximum height of 4 ft. if the global stability requirements are met (see Figure 11-7). GRS abutments with a truncated base are more likely to meet global stability requirements in cut conditions rather than fill conditions.

m. For bridges with a non-yielding foundation at the pier(s) and a semi-yielding reinforced soil/foundation at abutment(s), there is a possibility that cracks will appear in the top of the deck over the first pier near the abutment. Cover these cracks with waterproofing membrane and asphalt overlay; however, with bare concrete decks, check the crack size and rigorously control or mitigate with FRP top reinforcement in the deck.

11.3.6 Wingwalls

11.3.6.1 Wingwall Design Length

The wingwalls, as shown in the Bridge Detail Manual, shall be laid out from a working point defined as the intersection of abutment back face and wingwall fill face to 4 ft. minimum beyond the point of intersection of the embankment slope with the finished roadway grade. In most situations, using the working point provides the Contractor economy of design by having the same wingwall length at opposite corners. It is preferred that the wingwall be constructed parallel to girders to minimize the soil pressure against the wingwalls. The maximum integral wingwall length from the working point shall be 20 ft. If a longer wingwall is required, as shown in the Bridge Detail Manual, the Designer should use a maximum of a 10 ft. long integral wingwall in conjunction with an independent wingwall to achieve the required design length. It is not desirable to add a footing or support at the end of wingwalls for integral abutments unless provision for movement and rotation are provided. It is acceptable to support the wingwall ends on seat type abutments, on semi-integral abutments if the wingwall is not attached to the superstructure, or where no abutment rotation is expected.

The Designer needs to be aware of the various effects of soil on wingwalls and design for the anticipated loading due to the downdrag from fill settlement or uplift due to expansive soils. These forces can cause cracking of the wingwalls and abutment if they are not accounted for. If significant movement is predicted, the Geotechnical Report shall provide design recommendations and coordinate with the Designer on possible solutions. The Designer should analyze the torsional effects from the soil on the wingwall abutment connection and determine if 135° hooked stirrups are required. For wingwalls on box culverts, see Section 12.

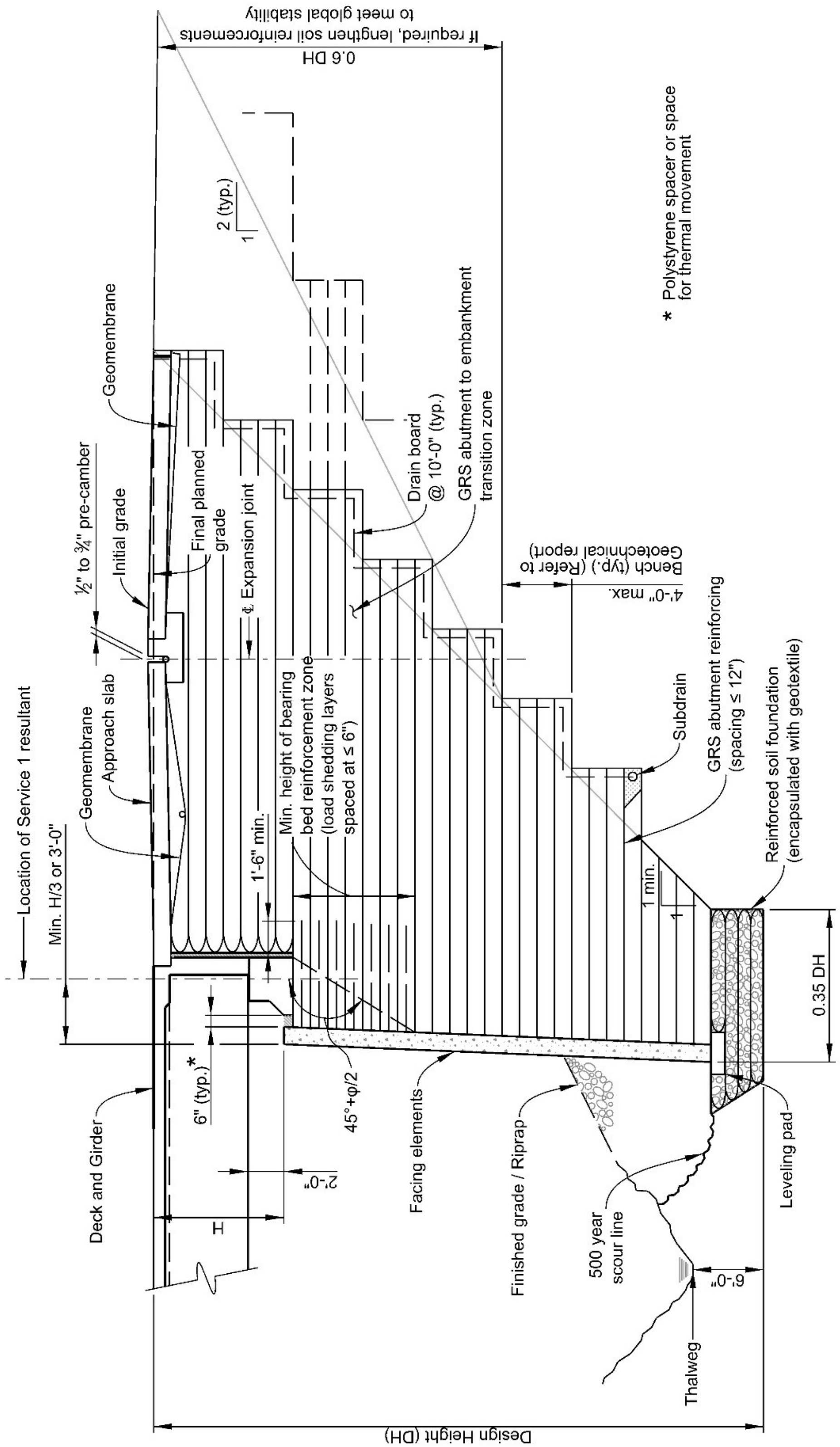


Figure 11-7: GRS Abutment (Cut Case)

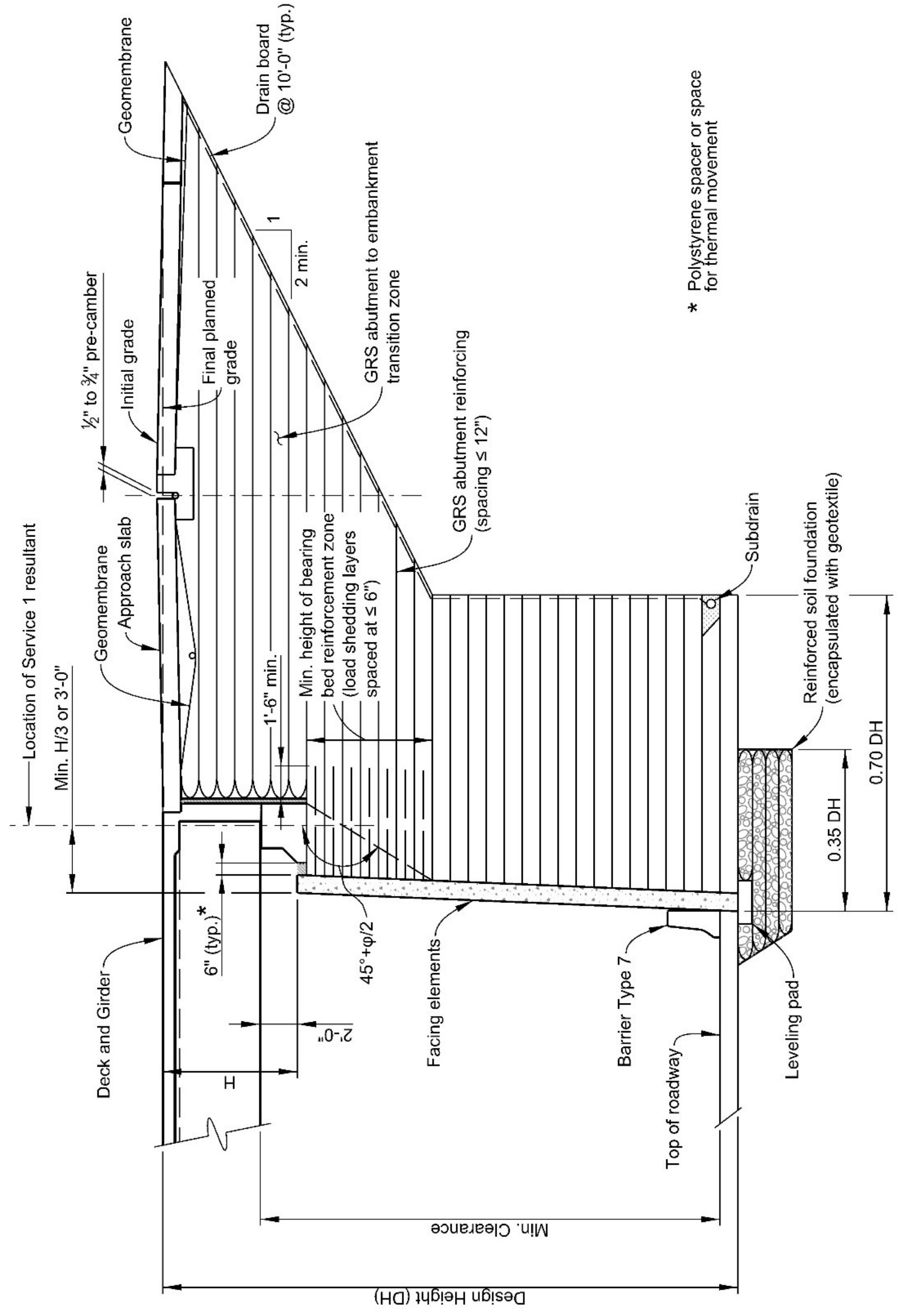


Figure 11-8: GRS Abutment (Fill Case)

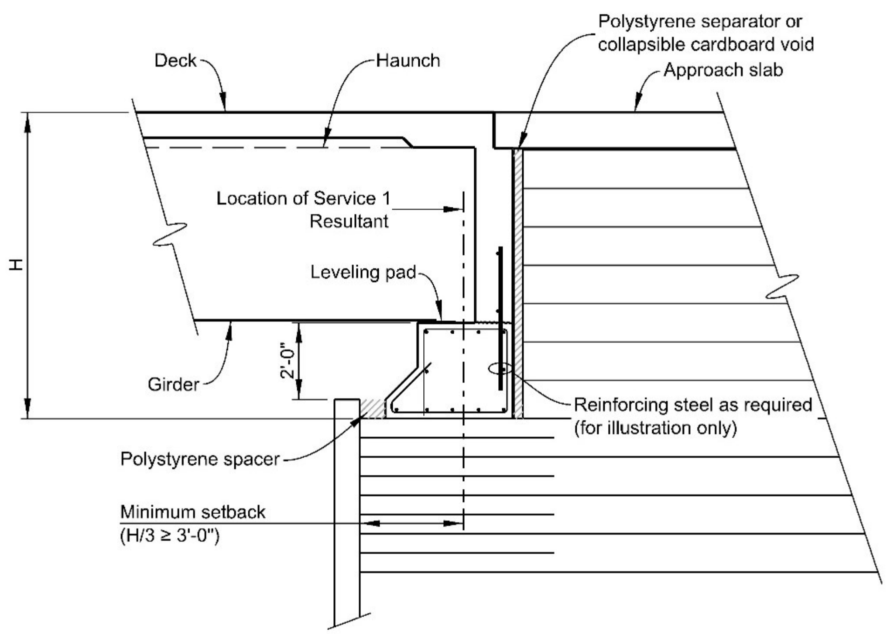


Figure 11-9: Integrated Girder Seat with Footer

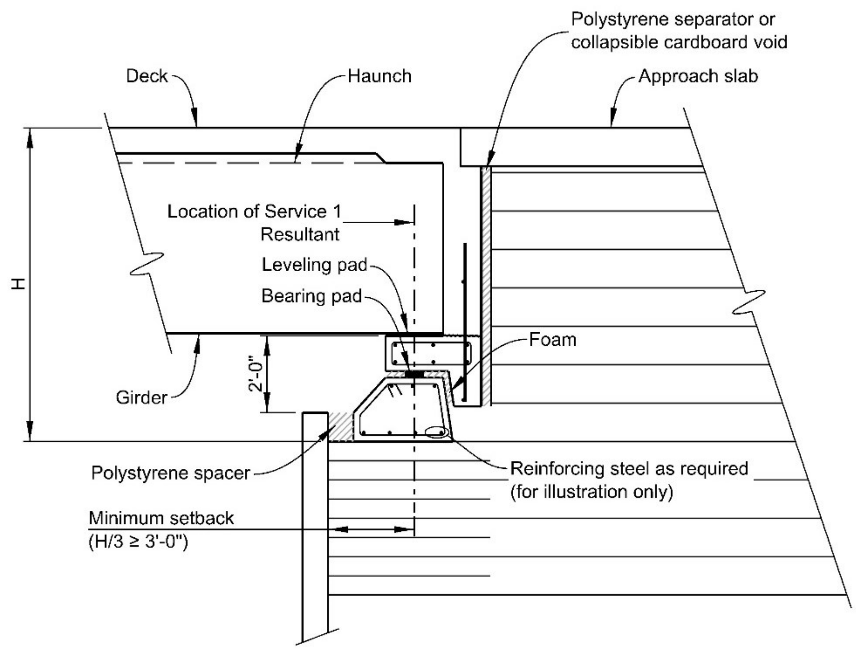


Figure 11-10: Separated Girder Seat with Footer

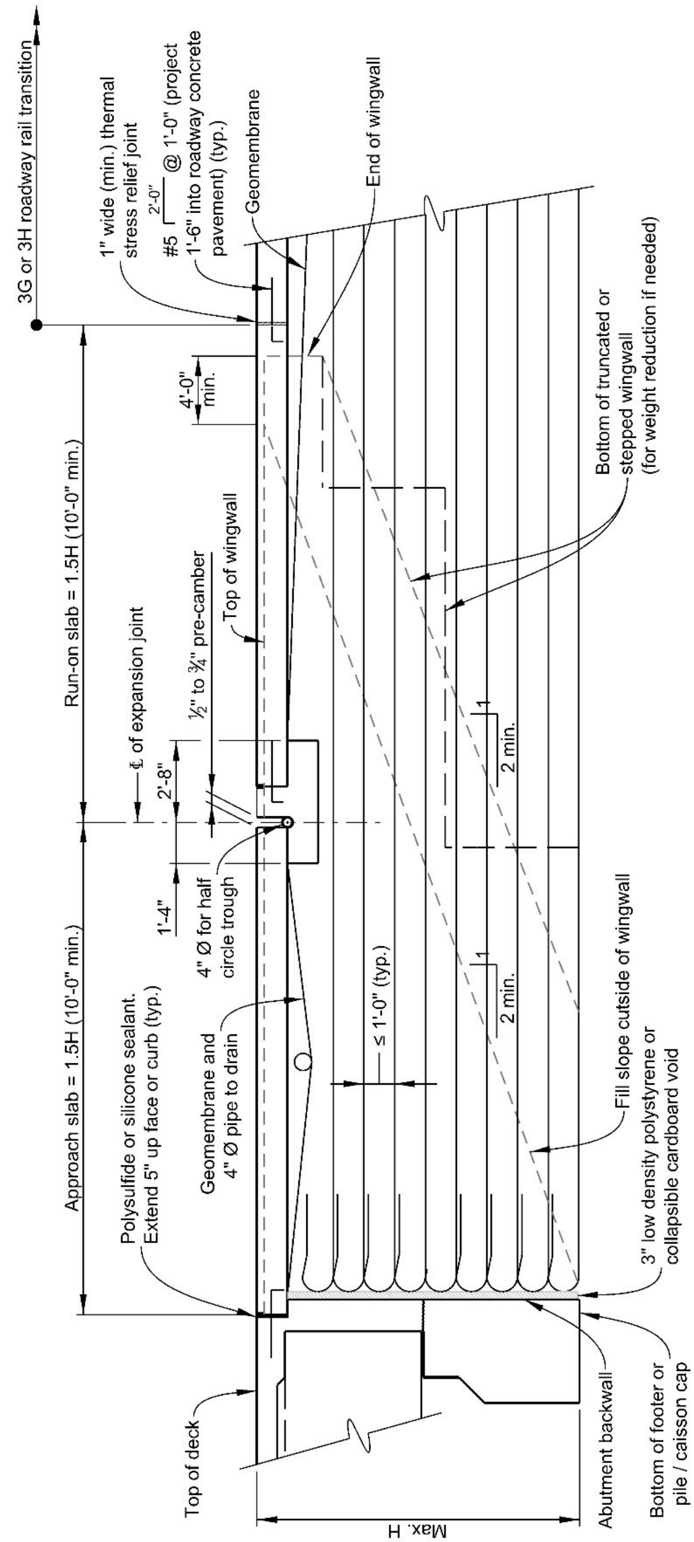


Figure 11-11: Transition Zone Behind Abutment Backwall (With Expansion Joint, Concrete Slab, and Roadway Pavement)

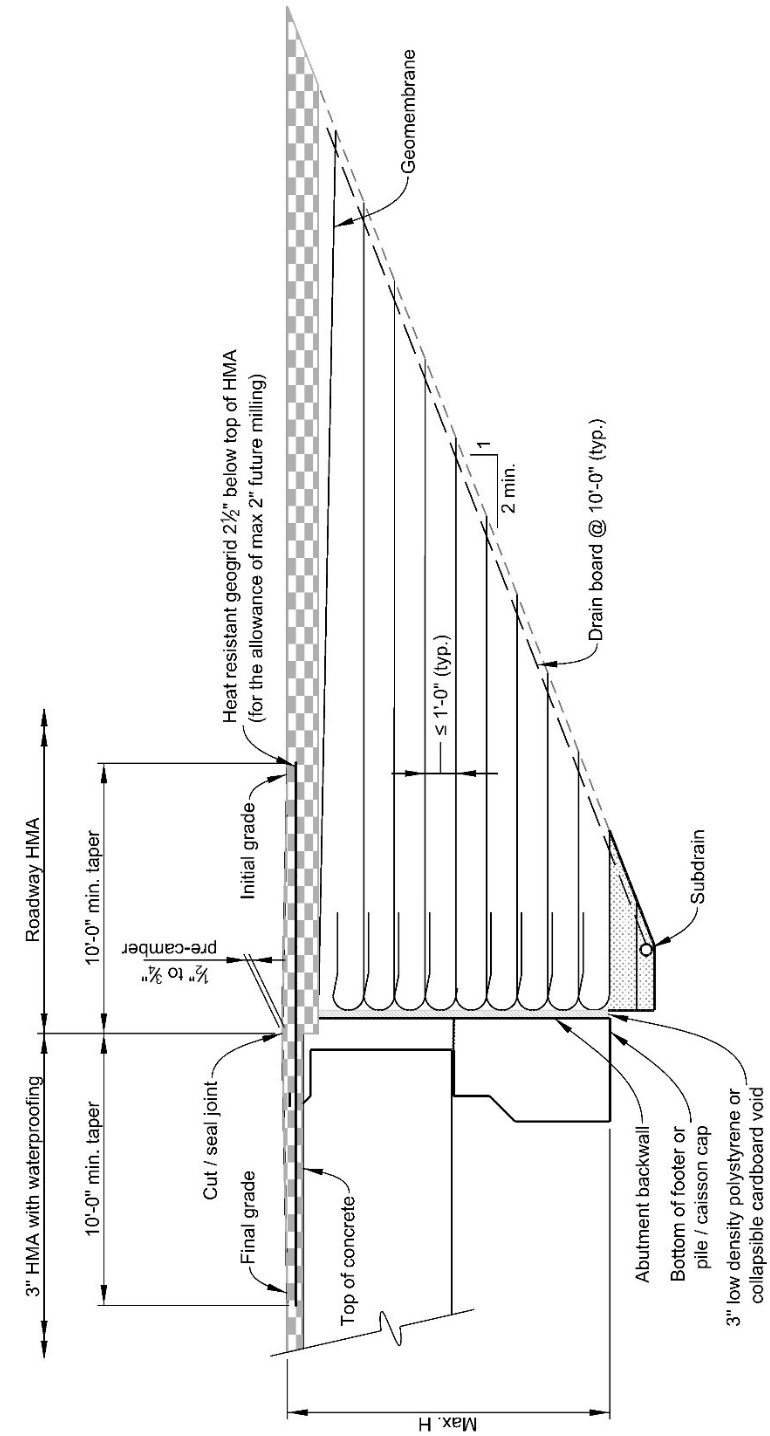


Figure 11-12: Transition Zone Behind Abutment Backwall (With Asphalt Pavement, No Approach Slab and No Expansion Joint)

11.3.6.2 Wingwall Design Loads

Design cantilevered wingwalls for tangent, non-skewed bridges for an active equivalent fluid pressure as recommended in the Geotechnical Report but not less than 36 psf. Design all other wingwalls for an at-rest equivalent fluid pressure recommended in the Geotechnical Report but not less than 57 psf. At-rest pressure is recommended for design in most cases because wingwalls on non-square bridges may undergo a transverse deflection into the backfill during longitudinal bridge movements, which could increase the pressure above active level.

The wingwall analysis shall include a live load surcharge load per AASHTO 3.11.6.4, regardless of the presence of an approach slab. Do not include vehicular collision unless the barrier is attached to the top of the wingwall.

Due to equilibrium of fill pressures on each side of the wingwall, the Designer may ignore the earth pressure below a line that extends from a point 3 ft. below the top of the wingwall at the end of the wingwall to another point at the bottom of the wingwall at the back face of the abutment. For erosion along the outside of the wingwall, 3 ft. is an assumed depth. This trapezoidal loading condition applies to wingwall design only and is not to be used for foundation stability analyses. Refer to Example 8: Cantilever Wingwall Design Loads for sample calculations and equations.

11.3.7 Approach Slabs

Construct approach slabs to match the required roadway width and sidewalk approaches. When a guardrail transition is required, the Designer shall provide 6 in. between the outside face of the bridge rail and the inside face of the wingwall, refer to the Bridge Detail Manual. This clearance may be eliminated when no guardrail transition is required or when rail anchor slab is used.

AASHTO Sections 3 & 5

The approach slab worksheet is a predesigned worksheet based on the Staff Bridge policy of designing approach slabs for 50% of the slab span based on a PCI methodology. Additional design is not necessary unless project requirements dictate a length not shown by the worksheet. This same methodology is required on approach slab lengths other than shown in the worksheet. The approach slab inlet worksheet is also a predesigned worksheet primarily based on ACI methodology. A variance will be required for changing the design. Limit post-construction settlement at the free end of the slab to 1 in. If the Geotechnical Engineer anticipates settlement greater than 1 in., the Designer shall incorporate plan details to mitigate the amount of settlement to 1 in. or less. One possible mitigation detail would be to raise the end of approach slab by the anticipated long-term settlement. For additional information on approach slabs, see Section 2.13 of this BDM and Staff Bridge Worksheets.

11.4 PIERS

Bridge piers provide intermediate support to the superstructure and a load path to the foundation. Suitable types of piers include, but are not limited to, the following:

- Solid Wall Piers

- Multi-Column (Frame) Piers
- Single Column (Hammerhead) Piers
- Straddle Bent Piers

AASHTO 11.7.1

Forces acting on the pier in the vertical, longitudinal, and transverse direction shall be per AASHTO. The connection between the superstructure and pier should be pinned by use of bearings or a key detail, allowing rotation in the longitudinal direction of the superstructure and eliminating longitudinal moment transfer to the substructure. Fixed or integral connections between the superstructure and substructure are not desirable. If the bridge is being designed with staged construction, each stage shall meet AASHTO.

The bearing cap should be a sufficient width and length to support the superstructure, meet support length requirements, and provide adequate bearings edge distances. A recommended pier width to depth ratio is less than or equal to 1.25. If the depth of the cap varies more than 18 in. from each end, slope the bottom of the cap. For precast prestressed concrete girder superstructure types, place the bearing lines a minimum of 12 in. normal to the centerline of cap. The minimum cap size shall be 3 ft. by 3 ft. and should increase thereafter by 3 in. increments. In section, the cap should overhang the column by 3 in. minimum. The length of the cap should not extend past the drip groove and should be rounded down to the nearest inch.

When designing the pier cap for negative moment, the preferred design plane is located at the face of the column or equivalent square for a round column.

To properly model the column / pier cap connection, provide a rigid link from the centerline column to the face of the column. If a rigid link is not provided, use the maximum moment at the centerline of column. See Section 5.4.11 of this BDM for pier cap reinforcing details.

When Strut & Tie Models are used for the design, they must be shared with the design checker to obtain concurrence on the models. Refer to Section 37.5 of this BDM for more details.

To ensure that the girder ends will rotate during the deck pour, the Designer shall add a note to the plans requiring the Contractor to pour the deck within two hours of the integral diaphragms.

Coordinate the selection of column type with the architect and CDOT. Possible column types include, but are not limited to, round, square, rectangular, tapered, and oblong. Standard forms should be used whenever possible and shall be 2 ft-6 in. minimum. To match standard form sizes, round, rectangular, and square columns should have length and width dimensions in 3 in. increments. When the columns are tall, place construction joints at approximately 30 ft. spacing. The preferred method of analysis for columns is moment magnification.

In lieu of moment magnification analysis, a second-order analysis is required. If magnification factors computed using AASHTO exceed about 1.4, then a second-order analysis will likely show significant benefits. The second-order analysis of the frame can be modeled using nonlinear finite element analysis software. AASHTO Seismic 4.11.5 discusses P-Δ effects and when they should be considered in the design.

Unless in a seismic zone as defined in Section 5.4.9 of this BDM or requested otherwise, tied hoops are preferred for transverse reinforcement, rather than spirals. The column spacing on framed piers should balance the dead load moments in the cap. ***AASHTO 4.5.3.2.2***

When setting the foundation location, the Designer shall provide 2 ft. minimum cover on top of the foundation element. To protect from frost heave, place the bottom of any footing below the frost depth indicated in the Geotechnical Report and no less than 3 ft. minimum below finished grade. The minimum depth of a footing on pile/drilled shafts and spread footings is 2 ft.-6 in. See Section 10.4.2 of this BDM for additional details.

When placing a pier in the floodplain, the Designer should align the pier with the 100-year flood flow. The preferred pier location is outside the floodplain whenever possible. To prevent drift buildup and when recommended by the Hydraulics Engineer, provide web walls between columns. The Designer shall consider the effects of uplift due to buoyancy forces when designing piers located in floodplains. Final pier locations should be coordinated with the Hydraulics Engineer.

When checking cracking, all caps and columns shall use Class 1 exposure condition. Foundation elements shall use Class 2 exposure condition.

The Structure Selection Report shall document the selected pier type and its location for the project.

If the pier has bearings that may need future maintenance or replacement, the Designer should show jacking locations and loads on the drawings. CDOT Standard Specification 503.20 provides the following horizontal tolerances for drilled shaft construction:

- 3 in. for shafts with diameters less than or equal to 2 ft.
- 4 in. for shafts with diameters greater than 2 ft. and less than 5 ft.
- 6 in. for shafts with diameters 5 ft, or larger.

These construction tolerances must be accommodated in the pier cap design to prevent a need to adjust the pier cap location during construction. In situations where the column steel has a contact lap splice with projected drilled shaft reinforcing, the column is required to follow the drilled shaft if the drilled shaft is misaligned. Therefore, provide pier cap overhang (distance from the column to the face of the cap) equal to or greater than the construction tolerance above to allow column location adjustment while the pier cap remains in place.

Also provide adequate dimensional tolerance between the column and drilled shaft via a non-contact lap splice, either by oversizing the drilled shaft or by oversizing the column. The inside cage should be able to move laterally by the amount of specified allowed construction tolerance without compromising the design or details of the members.

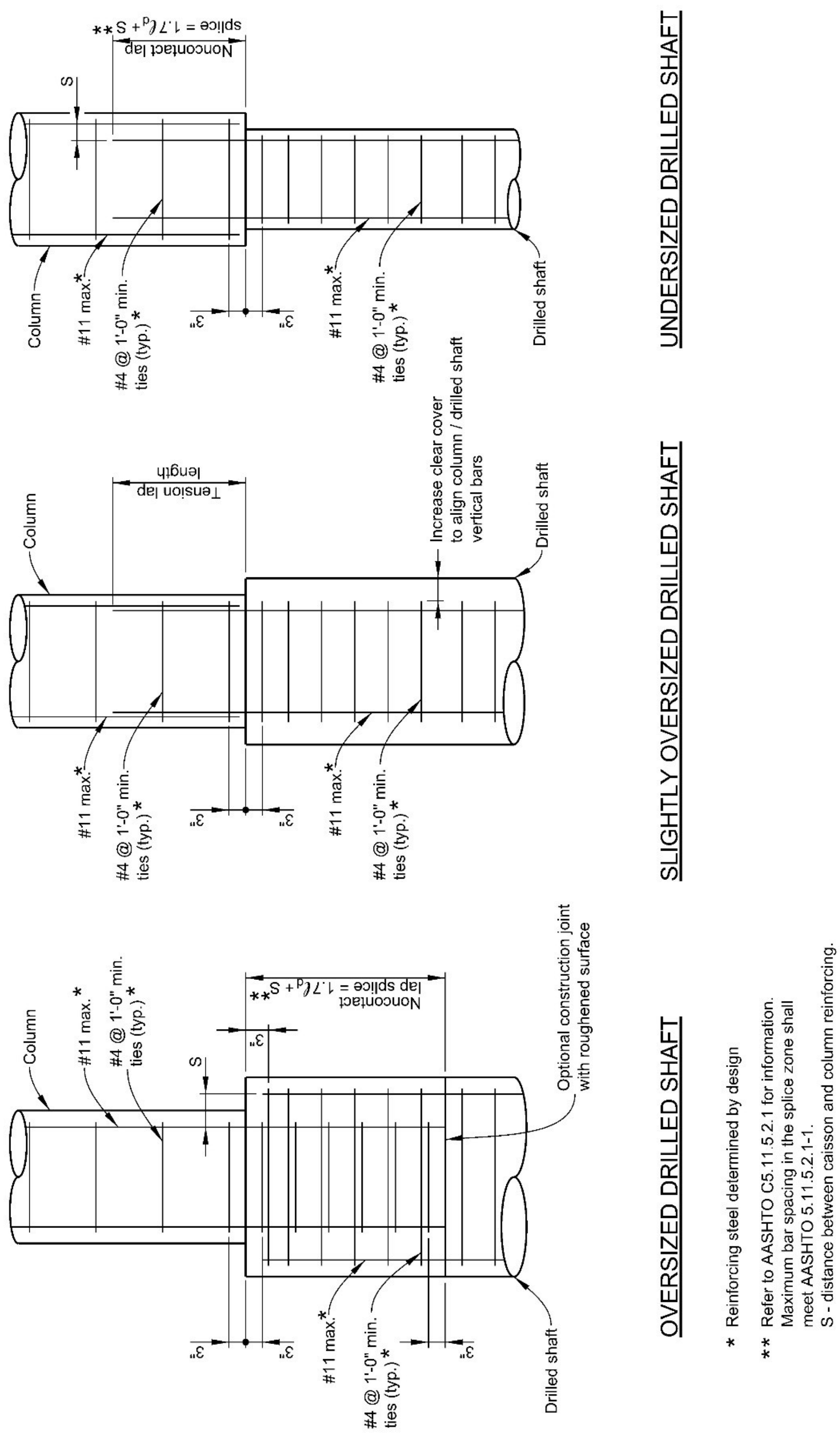


Figure 11-13: Column-Drilled Shaft Connection Details

11.4.1 Multi-Column Piers

Multi-column piers, the most commonly used pier type, consist of two or more transversely spaced columns. This type of pier is designed as a frame about the transverse direction (strong axis of the pier). The columns are usually fixed at the base and supported by one of the following foundation types: spread footing, footing on pile/drilled shafts, or drilled shafts.

11.4.2 Single Column (Hammerhead) Piers

Single column (Hammerhead, Tee) piers are usually supported at the base by a drilled shaft, spread footing, or footing on pile/ drilled shafts. Either the pier cap can be pinned in the longitudinal direction to the pier diaphragm and the diaphragm poured monolithically with the superstructure or the pier cap can be poured integrally with the superstructure. The column cross section can be various shapes and can be either prismatic or flared to form to the pier cap.

It is recommended that hammerhead style piers be modeled using the strut-and-tie method. This method creates an internal truss system that transfers the load from the bearings through the cap to the columns. The truss uses a series of compressive concrete struts and tensile steel ties to transfer the loads. Place nodes at each loading and support point. The angle between truss members should be between 25° minimum and 65° maximum with a preferred angle of 45°. If a wide column is used, place two or more nodes at points along the column.

11.4.3 Solid Wall Piers

Design solid wall piers per AASHTO. Assume the top of pier wall to be pinned or free at the top. Support the bottom of wall on either a spread footing or footing on piles/drilled shafts.

AASHTO 5.11.4.2

11.4.4 Straddle Bent Piers

Use straddle bent piers where there is a geometrical constraint in placing the piers. Such geometrical restrictions can be one or more of the following:

- Spanning a wide roadway
- Right-of-way (ROW) issues not permitting placing columns under the bridge
- Presence of railroad tracks to span over
- Presence of underground utilities where relocating them can be cost prohibitive
- Other

Straddle bent piers are non-redundant structures that can be conventionally reinforced, pre-tensioned or post-tensioned. Consider constructability, cost, span, and construction schedule when selecting the type of bent style.

Steel straddle bent caps are not permitted due to corrosion issues, inspection access concerns, fracture critical designation, high cost, and maintenance issues.

11.4.5 Aesthetics

Special corridor projects and signature bridges can have variations of the standard pier types or entirely unique pier designs. Coordination with Staff Bridge is essential at the preliminary phase of the project to determine the aesthetic requirements. The Structure Selection Report should document all aesthetic treatments required by the project.

11.4.6 Details

When a footing on pile is used, refer to Figure 11-14.

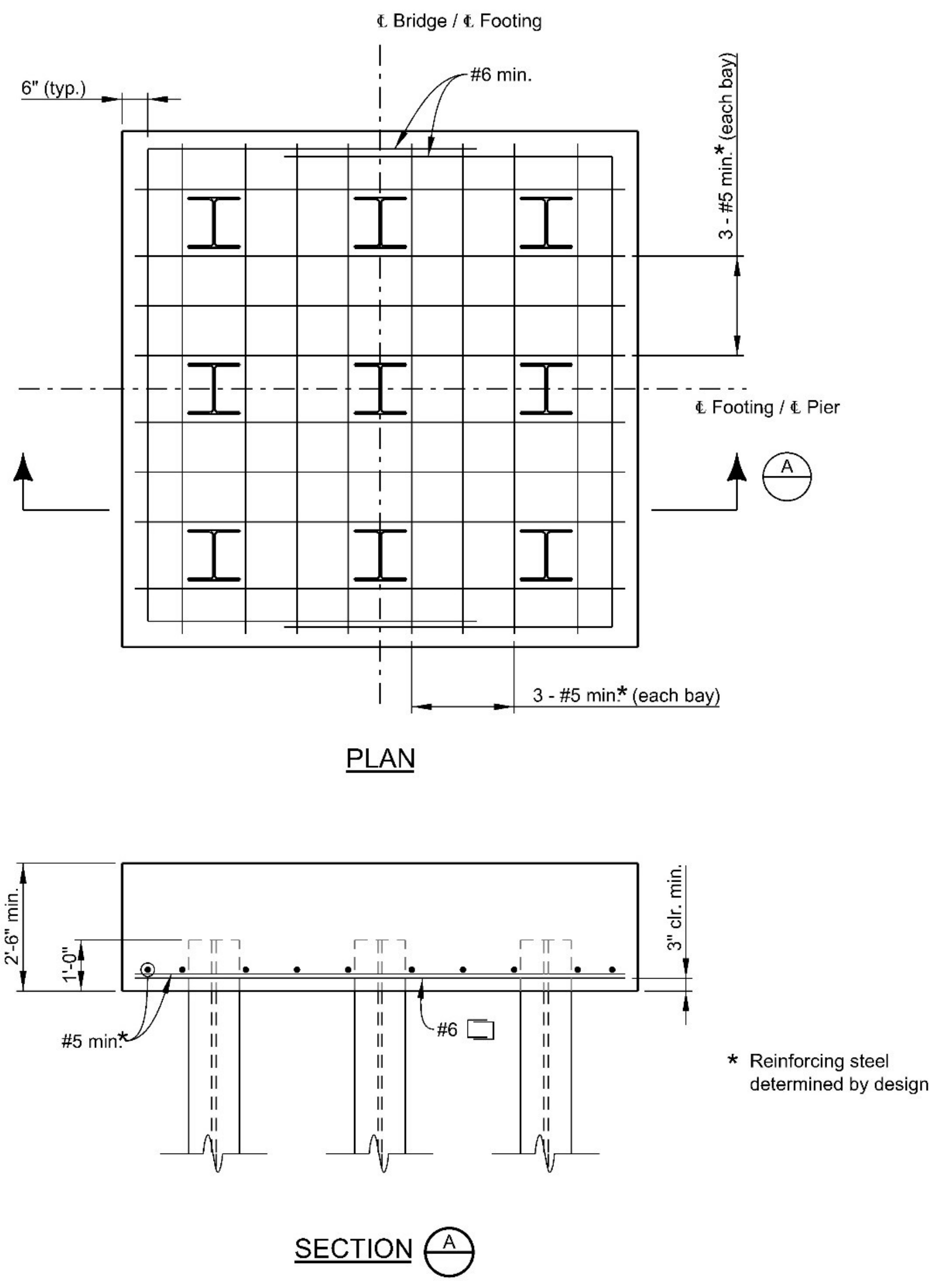


Figure 11-14: Footing on Pile

11.5 RETAINING WALLS

Design permanent retaining walls for a service life based on AASHTO. Design retaining walls for temporary applications for a service life of 3 years. ***AASHTO Section 11***

Retaining walls can be classified into three categories according to their basic mechanisms of soil retention and source of support. Externally stabilized systems use a physical structure to retain the soil. Internally stabilized systems involve reinforcement (e.g., soil nails and geosynthetics) to support loads. The third system is a hybrid that combines elements of both externally and internally stabilized systems.

Calculate earth pressures in accordance with AASHTO. The Designer shall use Coulomb's earth pressure theory to determine the active coefficient of lateral earth pressure. The minimum equivalent fluid due to soil pressure shall be 36 pcf. If the wall design height is less than 4 ft. and a geotechnical report is not required or has not been provided, the Designer may assume a nominal soil bearing capacity of 6 ksf. ***AASHTO Section 3.11.5***

Settlement criteria will depend on the wall type and project constraints, such as nearby structures and the project schedule. The structural and geotechnical engineers should coordinate to select and design an appropriate wall system capable of meeting project requirements. For instance, the bearing resistance of wall footings will depend on the footing size.

Most walls that support vertical loads, unlike columns, do not require the 1% minimum longitudinal steel. When the vertical load becomes so great that buckling is a concern, walls should be treated like columns and meet compressive member requirements. A ratio of the clear height to the maximum plan dimensions of 2.5 may be used per AASHTO to differentiate between walls and columns (C5.11.4.1), but it should primarily be behaviorally based. Some references use b/d ratios of 3 to 6 to differentiate between walls and columns. See Section 11.4.3 of this BDM for more information on solid pier walls. ***AASHTO 5.11.4.1***

Provide weep holes or a drainage system behind the wall stem to prevent water accumulation. The Designer should reference Staff Bridge Worksheets for required size and spacing of weep holes or provide drainage system details in the project plans. The final drainage system selected will depend on the amount of water anticipated to infiltrate into the backfill and shall consider groundwater conditions.

Runoff shall not be permitted to pass freely over the wall; rather, a wall coping, drain system, or a properly designed ditch shall be used to carry runoff water along the wall to be properly deposited. Where this is not feasible, such as soil nail walls in steep terrain, the Designer shall coordinate with Staff Bridge to develop a solution that has concurrence from Region Maintenance and Bridge Asset Management.

When laying out walls, if possible, provide a 10 ft. inspection zone in front of the wall. The Designer must consider ROW limits for placement of the footings and if temporary easements are needed for excavation. Any wall footings, straps, soil anchors, or other wall elements shall be contained within the established ROW limits unless a permanent easement is obtained. The

Designer shall coordinate with the Roadway Engineer to determine final wall layouts and grading requirements.

The Wall Structure Selection Report shall be provided per Section 2.10.4 of this BDM. Appendix 11A contains worksheets to assist in developing wall selection options.

The following are the most common retaining walls used in Colorado:

11.5.1 Cantilever Retaining Wall

Cast-in-place and precast cantilever retaining wall systems are considered semi-gravity walls. Conventional cantilever walls consist of a concrete stem and a concrete footing, both of which are relatively thin and fully reinforced to resist the moment and shear to which they are subject. A cantilever wall foundation can be either a spread footing or a footing on deep foundations. Document the recommendation of the soil parameters and preferred foundation type in the Geotechnical Report and include in the plan set.

For retaining walls without concrete curb or barrier attached to the top of the wall, top of the wall shall be a minimum of 6 in. above the ground at the back face.

If a shear key is required to provide adequate sliding resistance, place it approximately one-third of the footing width from the heel to the centerline of the key. If additional depth for development length of the reinforcing is needed, it may be shifted to under the stem in lieu of increasing the footing thickness. Passive resistance shall be neglected in stability calculations and shall not be counted on for sliding resistance unless a shear key below frost depth is provided. Soil that may be removed due to future construction, erosion, or scour shall not be included in determining passive sliding resistance. The Designer shall, at a minimum, ignore the top 1 ft. of front face fill when determining sliding resistance. See Figure 11-15 for the passive resistance loading due to the shear key.

Protect retaining wall spread footings from frost heave by placing the bottom of the footing a minimum of 3 ft. below finished grade at front face. Top of footings shall have a minimum of 1.50 ft. of cover.

Sloped footings are permitted with a maximum slope of 10 percent.

Stepped footings may be used with maximum step of 4 ft.

Reinforcement should be as shown on Figure 11-16.

11.5.2 Counterfort Retaining Wall

Counterfort retaining walls, another type of semi-gravity wall, are an economical option for wall heights 25 ft. and taller. They are designed to carry loads in two directions. The horizontal earth pressure is carried laterally to the counterfort through the stem. The counterfort is a thickened portion that extends normal to the stem and is used to transfer the overturning loads directly to the foundation.

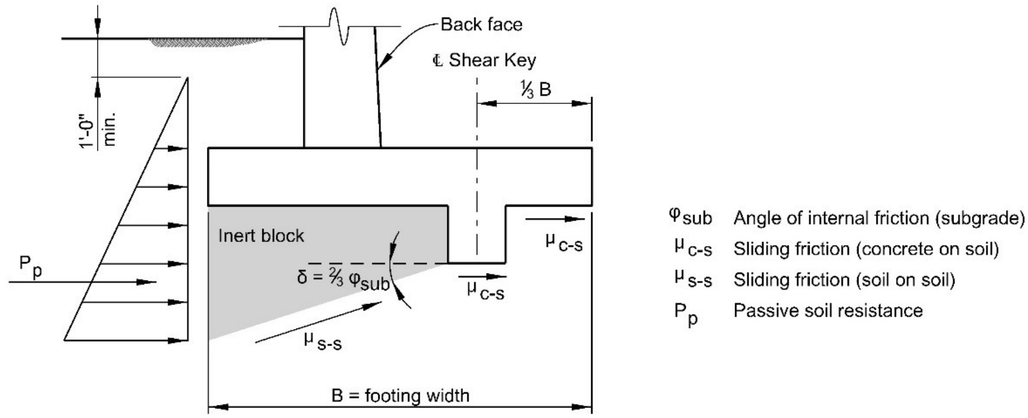


Figure 11-15: Shear Key

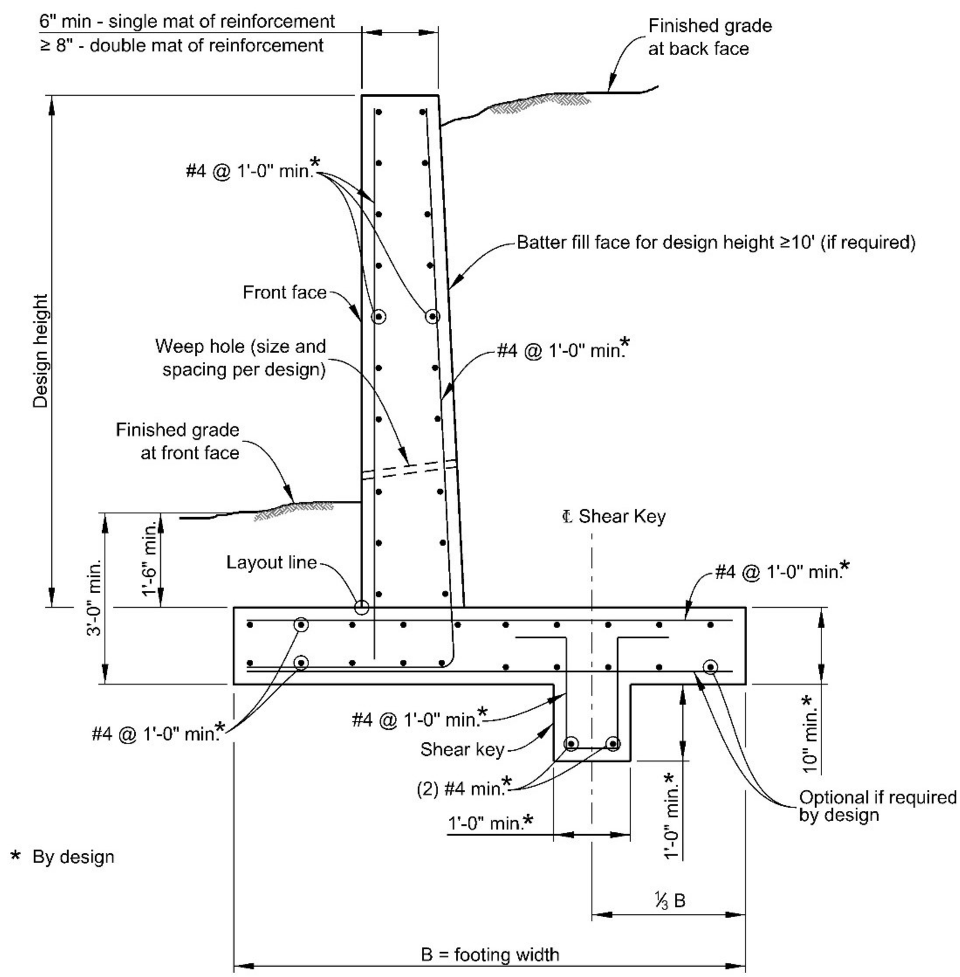


Figure 11-16: Cantilever Retaining Wall Reinforcement

11.5.3 Mechanically Stabilized Earth Wall

MSE walls, as detailed in the Staff Bridge Worksheets, are reinforced soil retaining wall systems that consist of vertical or near vertical facing panels or blocks, metallic or polymeric tensile soil reinforcement, and granular backfill. MSE walls are typically classified into one-stage and two-stage, where two-stage are used for large long-term settlements as outlined in Section 11.5.3.1 of this BDM. The strength and stability of MSE walls derive from the composite response due to the frictional interaction between the reinforcement and the granular fill. MSE systems can be classified according to the reinforcement geometry, stress transfer mechanism, reinforcement material, extensibility of the reinforcement material, and type of facing.

Sufficient ROW is required to install the reinforcing strips that extend into the backfill area 8 ft. minimum, 70 percent of the wall height or as per design requirements, whichever is greater. Truncated base or linearly varied reinforced zone per Staff Bridge Worksheets is allowed in cut conditions; they can be used when space constraint is a concern. Barrier curbs constructed over or in line with the front face of the wall shall have adequate room provided laterally between the back of the curb or slab and wall facing so that load is not directly transmitted to the top wall facing units. For more details, refer to Staff Bridge Worksheets B-504-V1. ***AASHTO 11.10.2.1***

For block walls and partial height panel facing walls, set the leveling pad a minimum of 18 in. from finished grade at front face to top of pad. When using full height panels, set them a minimum of 3 ft. below finished grade at front face to top of pad. If the front face fill is sloped in either direction, the Designer shall provide a 4 ft. minimum horizontal bench measured from the front face of facing. MSE structures are considered earth structures and are not subject to the minimum depth requirements for frost heave. The concrete leveling pad shall be reinforced along its entire length per the worksheet details.

For a retaining wall with a rail anchor slab placed at the top of the wall, allow a minimum 11 ft. wide (including rail), 40 ft. long monolithically constructed reinforced concrete barrier and slab system to carry and spread loads. Rail anchor slab with lesser widths may be allowed if a thicker or longer slab is designed by the Engineer to maintain stability. See Example 12, Rail Anchor Slab Design, for additional information on the design of a rail anchor slab.

Attach a minimum 12 in. wide geotextile to the back face of all joints in facing panels to reduce the loss of backfill through the joints.

The Designer must be aware of the possibility of the presence of an abutment, soil nail wall or other additional loads near the MSE wall affecting the design of the wall. Additional tie backs or straps may be necessary to deal with the loads on the facing. It is the Designer's responsibility to determine if an MSE wall is in the influence zone of an abutment, thus adding surcharge loads per AASHTO 3.11.6.3, and to adjust the design accordingly per AASHTO 11.10 as required.

The Designer shall reference the Standard Special Provisions, Standard Specifications for Road and Bridge Construction, and Staff Bridge Worksheets for the most current design requirements and material properties required for design. The Staff Bridge Worksheets were created based on the AASHTO Simplified Method, which is CDOT's preferred method of design. Any other design method requires approval by the Unit Leader.

11.5.3.1 Two-Stage MSE Walls

One-stage MSE wall detail shown on Staff Bridge MSE Wall Worksheets can accommodate up to 1 in. of differential settlement between soil mass and the panels. If this limit is exceeded, the wall shall be evaluated for use of modified details or a two-stage MSE Wall. Geotechnical Engineer shall provide wall type recommendations for every project.

Two-stage MSE walls are constructed in two stages. During the first stage, the reinforced soil mass is constructed and left to settle until the remaining settlement is within the tolerances of the permanent facing. Settlement could be accelerated by installing wick drains, if necessary. The second stage is the installation of the permanent wall facing.

Other options to mitigate the long-term settlement, such as excavation and replacement of soil, deep foundations, and ground improvement, may be more expensive than a two-stage wall. In the Structure Selection Report, all alternatives should compare settlement mitigation, schedule, constructability, and cost.

11.5.3.2 Precast Concrete Panel Wall

MSE walls often use a fascia consisting of precast concrete panels. Full height or segmental panels based on the corridor architectural requirements are allowed.

Full height panel width is limited to 10 ft. and the height to 30 ft. The use of larger panel dimensions will require the approval of Unit Leader in coordination with the Wall SMEs and must be documented in the Structure Selection Report.

The segmental panel area is limited to a maximum of 50 sf. with a minimum panel height of 2.5 ft.

The segmental panel will tolerate more differential settlement than the full height panel.

11.5.3.3 Modular Block Wall

Block wall facing is made of various shapes and colors of concrete block units that will fit many architectural needs and has been specifically designed and manufactured for retaining wall application. Two types of blocks are available for use: dry cast and wet cast. Dry cast blocks have shown a propensity to degrade with age and exposure to weather and salts and can be difficult and expensive to repair. Wet cast blocks have been shown not to have many of these issues.

This type of retaining wall will tolerate greater differential settlement between the blocks than a segmental panel or full height panel.

Use of dry cast blocks in a wall is not a preferred option adjacent to a roadway due to challenges of repair in the event of vehicular collision, water intrusion, and deterioration from de-icing chemicals and therefore their use requires Unit Leader approval. Dry cast blocks are an acceptable facing solution for landscape walls and around detention basins.

CDOT has experienced wall failures when using blocks in front of soil nail walls with inadequate block anchoring. To prevent future failures, the Designer shall apply the full earth pressure to the block anchorage connection.

11.5.3.4 Cast-in-Place and/or Shotcrete Facing

MSE walls can also have a cast-in-place (CIP) facing in front of the reinforced soil mass. The CIP facing can be either CIP and/or shotcrete concrete.

11.5.3.5 GRS Walls

This type of wall is generic (non-proprietary) and has a single grade of woven geotextile spaced at 8 in., including 4 ft. of tail soil reinforcement. Every modular block facing in a GRS wall is connected with a layer of soil reinforcement, Reinforcement-to-block connection mechanism is primarily based on friction and clamping action. Soil reinforcement-to-block pullout test is waived for this type of MSE wall; thus no soil reinforcement schedule or shop drawing submittal is required.

The design Engineer of Record shall thoroughly check internal, external, and global stability. The geotechnical report shall address temporary cut slope stability.

11.5.3.6 Truncated Base Walls

For a MSE wall within a cut condition, a truncated base soil reinforcing zone can provide an economical space constrained solution. The truncated base of trapezoidal soil reinforcing zone shall be 45 percent of design wall height or 4 ft., whichever is greater. The linearly varied soil reinforcement length and its maximum length at top depend on temporary cut slope stability. Use of this type of MSE wall is determined by geotechnical stability.

11.5.3.7 Collision on MSE Walls

MSE wall panels are considered sacrificial and do not require design for the vehicular collision force (CT), unless directed otherwise.

Current interpretation of AASHTO and federal design guidance for collision loads is shown on Staff Bridge Worksheets (B504H or B504O series). Depending on selected barrier type or moment slab use, the application of force may vary. Standard design practice will use a TL4 transverse design force of 80 kips. This force shall be distributed horizontally along the barrier and vertically into the resisting elements as appropriate. For rail slab applications, the 80 kip force is distributed along a 10.83 ft. length and linearly from a maximum to zero at a depth of 15.1 ft. When using a moment slab a factored impact load of 900 lbs is applied to each of the top two reinforcing layers. See Staff Bridge MSE Wall Worksheets for additional information. The findings of NCHRP 663 may be used but have been updated in a later report.

NCHRP Document 326 based on NCHRP Project 22-20(02) may be used as a reference for collision design and indicates a static load of 28 kips may be a sufficient TL-4 design load depending on the configuration of the moment slab and wall facing.

Test Designation	L_d (1) (kips)	L_s (2) (kips)	H_{min} (3) (in.)	H_e (4) (in.)	W_{min} (5) (ft)	B_L (6) (ft)
TL-3 (7)	70	23	32	24	4	10
TL-4-1	70	28	36	25	4.5	10
TL4-2	80	28	>36	30	4.5	10
TL-5-1	160	80	42	34	7	15
TL-5-2	260	132	>42	43	12	15

(1) Dynamic Load L_d

(2) Equivalent static load (Ls) applied at height He, calculated based on the static resistance deemed more critical for the barrier as follows: the overturning resistance for TL-3, TL-4 and TL-5-1 barriers and the sliding resistance for TL-5-2 barrier.

(3) Minimum barrier height H_{min}

(4) Effective barrier height H_e

(5) Minimum moment slab width W_{min}

(6) Minimum length of the precast barrier B_L

(7) Revised from the recommendations in NCHRP Report 663, Figure 7.1 (*2*)

11.5.4 Drilled Shaft Walls

Drilled shafts walls, also known as secant or tangent pile walls, consist of drilled shafts spaced along the wall alignment with an attached precast or CIP facing. They are typically used in areas where excavation limits are restricted due to ROW or there is an obstruction such as a building or utility. Micropiles can also be used when access is limited for drill rigs. The micropiles can be a single row or two rows with one battered to form an A-frame configuration.

11.5.5 Anchored Walls

Anchored walls (externally stabilized), although not routinely used in Colorado, may be appropriate for relatively high cuts or sites with stringent deformation criteria, particularly in situations where top-down construction is required. Anchored wall systems use ground anchors (e.g., tiebacks bonded into the ground, deadman anchors) to resist earth pressures acting on the wall. Anchored systems may include soldier pile and lagging, sheet pile, and drilled shaft walls.

The design of anchored walls should follow AASHTO. ***AASHTO 11.8***

11.5.6 Soil Nail Walls

Soil nail walls (internally stabilized) are frequently used as top-down, permanent retaining structures in Colorado. Soil nail walls are best suited to sites with adequate "stand-up" time, i.e., the ability of the soil to stand unsupported during wall construction. ***AASHTO 11.9***

The FHWA publication S*oil Nail Walls Reference Manual* (FHWA-NHI-14-007) provides guidance for the design of soil nail walls and is the recommended design manual for soil walls used on CDOT projects.

The Geotechnical Engineer shall be responsible for the entirety of the wall design, except for structural components such as the permanent facing, or as otherwise identified by the Geotechnical Engineer and shown in the Structure Selection Report.

When soil nail walls extend past the existing bridge abutment, future widenings need to be considered. To allow room for future pile installation, diamond patterns shall not be used within the ultimate configuration of the bridge (Figure 11-17).

Soil nail walls are typically designed with the assumption of dry soil conditions. For dry conditions, the typical soil nail bond strength is 10 to 15 psi with a maximum of 30 psi. However, for a high ground water table, spring water seepage, or heavy storm water runoff conditions, bond strength is reduced significantly. Without rigorous temporary drainage measures required during construction, wet condition bond strength must be considered and designed for by the Contractor's design Engineer of Record.

11.5.7 Gravity Walls

Rigid retaining walls of concrete or masonry stone that derive their capacity through the dead weight of their mass may be used for earth retention. Due to increases in material costs, conventional types of these walls made from concrete or stone are expensive. More affordable gravity walls, such as gabion baskets, have become more prevalent and are easily constructible.

11.5.8 Landscape Walls

Landscape walls retain soil less than 4 ft. in height from the finished grade to the top of the wall at any point along the length of the wall.

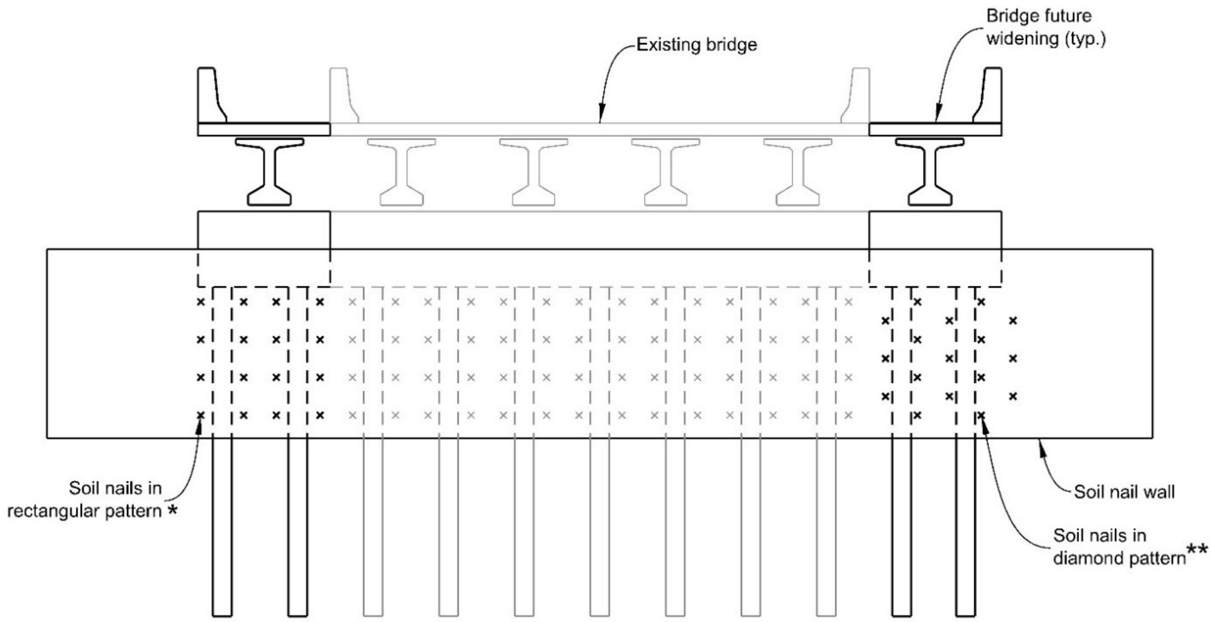
11.5.9 Load Combinations

AASHTO 3.4.1

Table 11-2 summarizes the load combinations used for wall design. Use Strength Ia and Extreme Event II to check sliding and overturning and to minimize resisting loads and maximize overturning loads. Use Strength Ib and Extreme Event II to check bearing and maximize loads for both overturning and resisting.

Note that live load surcharge (LS) and horizontal earth load (EH) are not included in the Extreme Event load case for vehicle collision load (CT). It can be assumed that the horizontal earth pressure is not activated due to the force of the collision deflecting the wall away from the soil mass at the instant of collision.

Use the service limit state for the crack control check.



* Rectangular pattern is preferred in areas of future widenings. Other patterns that provide sufficient clearance for future piles may be allowed.

** Do not use diamond pattern in areas of future widenings to avoid potential pile interference.

Figure 11-17: Soil Nail Wall in Future Bridge Widening Area

Table 11-2: Load Factors for Retaining Wall Design

Combination	γ_{DC}	γ_{EV}	γ_{LS_V}	γ_{LS_H}	γ_{EH}	γ_{CT}	Application
Strength Ia	0.90	1.00	–	1.75	1.50	–	Sliding, Eccentricity
Strength Ib	1.25	1.35	1.75	1.75	1.50	–	Bearing, Strength Design
Strength IV	1.50	1.35	–	–	1.50		Bearing
Extreme II	1.00	1.00	–	–	–	1.00	Sliding, Eccentricity, Bearing
Service I	1.00	1.00	1.00	1.00	1.00	–	Wall Crack Control

11.5.10 Resistance Factors

Resistance factors shall be per AASHTO or as given in the Geotechnical Report. Resistance factors for sliding and bearing are given in AASHTO Table 11.5.7-1. Resistance factors for passive pressure resistance are given in AASHTO Table 10.5.5.2.2-1. If an extreme event affects the wall, the resistance factors shall be per AASHTO 11.5.8.

AASHTO 10.5, 11.5

11.5.11 Collision with a Wall

AASHTO does not explicitly address how to design for collision load (CT) with a CIP wall or how the load is distributed. Conservatively, CT shall be applied at the end of the wall unless the barrier does not extend to the end of the wall. Figure 11-18 provides an example of the distribution. Assume that the horizontal earth pressure is not activated due to the force of the collision deflecting the wall away from the soil mass at the instant of collision.

For a Type 9 barrier, assume that the total lateral distribution will extend horizontally for 3.5 ft. and then downward at 45° from the point of collision. The length of distribution from impact force, L_t = 3.5 ft., for a TL4 rated barrier is taken from AASHTO LRFD Table A13.2-1.

For collision with a Type 10 barrier (post and rail), distribute CT horizontally between posts (3 maximum) and down from top of curb/wall to bottom of footing at 45°. At the end of a wall, assume a horizontal distribution distance from the edge distance to the first post plus one bay and then down at 45 percent.

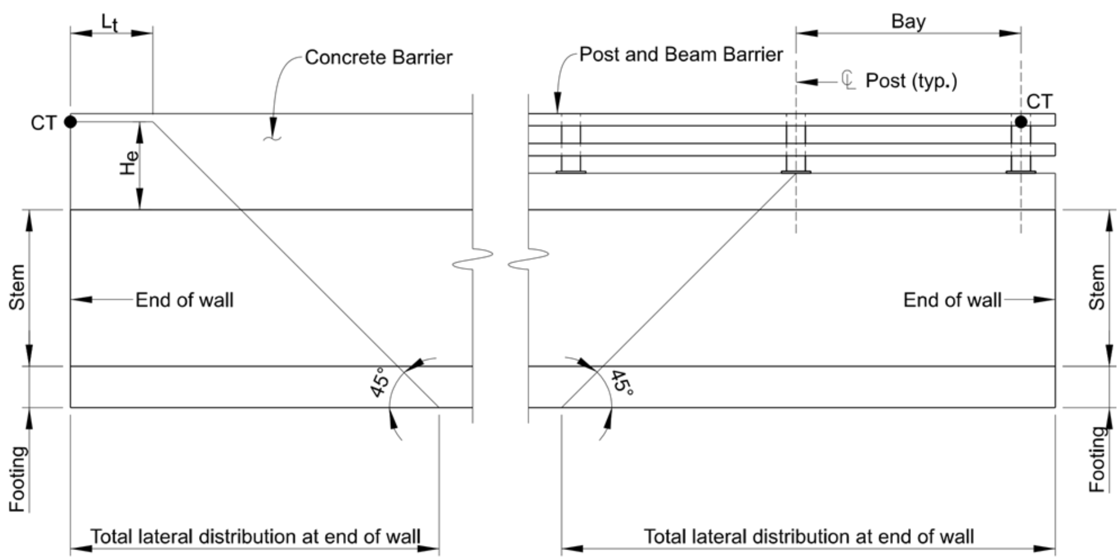


Figure 11-18: Lateral Collision Distribution

The previously described method is fairly conservative and does not always correlate with reality well since it assumes that reinforcing is similar vertically and horizontally. Walls with barrier on top should generally be designed using Chapter 13 of the AASHTO code as a very tall parapet which makes Lu a function of the relative strength vertically and transversely. For barrier with steel posts, the transferred load should be based on the capacity of the post as the impact is typically shared between 3 and 6 posts.

The findings of NCHRP 22-20(2) may be used to determine equivalent static forces for sliding and overturning stability on MSE walls. For CIP walls the load may be reduced as the section of interest goes below the riding surface due to the increased mass and reaction time as more of the wall is involved. These values can be tentatively used as 100% at the ground line, 33% at 6' below the load application and 0% at 9' below the load application. These values are extrapolated from the data in 22-20(2) with the “6' below” percentage reflecting the results for sleeper significant movement at the back of the sleeper for TL3 and 4 crashes. The value of the 0% at 9' reflects the depth at no movement for TL-5 crashes.

To mitigate the effects of live load collision with CIP or precast face panels, for all walls that use face panels (e.g., caisson walls, soil nail walls, MSE Walls), a void between soil mass (or caissons) and back face of the panel shall be filled with granular material to the minimum height of 5 ft. above the roadway surface. Wall panels shall be required to support their own weight in case of impact damage that would allow the panel to slip below precast copings or clip angles. Reinforcing spacing should be minimized on panels to limit projectile size when impacted (6 in. max spacing). Welded wired fabric may be used in addition to reinforcing to minimize projectile size.

11.5.12 Global and Compound Stability

The global stability and compound stability shall be per AASHTO and the Geotechnical Design Manual. Global stability of the wall depends on the footing width and embedment.

AASHTO 11.6.2.3

The project Geotechnical Engineer shall evaluate global stability. Minimum factors of safety for global stability shall meet the requirements of the Geotechnical Design Manual and AASHTO. The Geotechnical Engineer shall specify the minimum requirements to achieve the specified factors of safety (e.g., minimum reinforced zone length for MSE walls, minimum soil nail length, and configuration for soil nail walls).

Compound stability of MSE and soil nail walls will depend on the reinforcement type, length, and spacing. Therefore, the vendor is responsible for checking compound stability based on their submittal (see Figure 11-19).

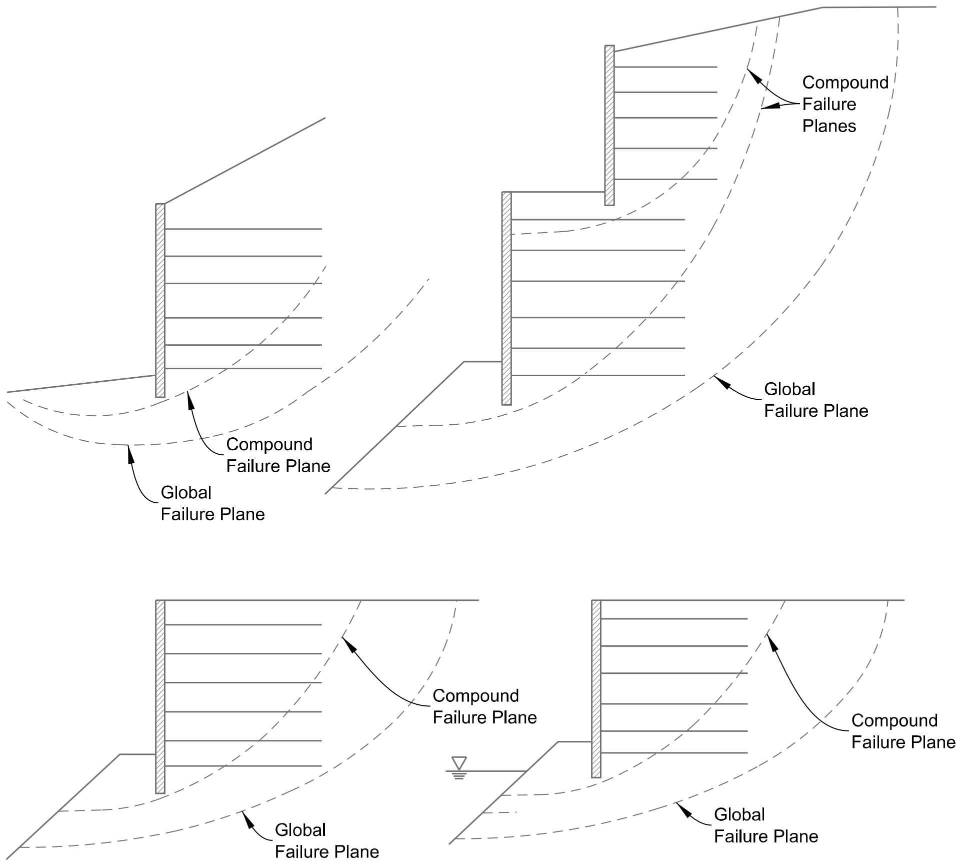


Figure 11-19: Global and Compound Failure Planes

11.5.13 Designer Responsibility for Walls

AASHTO 11.6.2.3

External stability addresses concerns with the stability of sliding masses defined by slip surfaces that pass outside the reinforced soil zone. The checks required include global stability of the structure, determination of eccentricity limits, sliding analysis, bearing capacity analysis of the foundation/supporting soils, and settlement analysis. These checks shall be performed by the Engineer of Record responsible for the design, whether that be the owner's representative, Geotechnical Engineer, Structural Engineer, or Vendor. If the wall is a vendor design, the vendor's Independent Design Engineer is responsible for submitting stamped calculations showing the external stability check for review. All walls are to be designed and built according to Standard Specifications Subsection 504.

Internal stability typically includes both pullout and rupture of the reinforcement. Responsibility for this check includes wall system components, including facing units, soil reinforcements, structural attachments, reinforcement connections to the facing units, bearing pads, and joint covering filter fabrics. Design responsibility shall fall on the engineer responsible for the design, whether that be the owner's representative, Geotechnical Engineer, Structural Engineer, or Vendor.

Global stability, compound stability, and deep seated failure conditions are closely related to external stability checks. It can be defined as the overall stability of the wall and surrounding slopes and structures. It requires the analysis of the surrounding circular slip surfaces. See Section 11.5.12 of this BDM for global stability requirements.

The Project Engineer of Record is responsible for collecting and reviewing wall submittals, which can include, but are not limited to, stamped calculations, shop drawings, etc. During the shop review process, bearing pressure, strap length and other minimum requirements from the worksheets shall be reviewed. Separate contractor designs are required when not meeting minimum requirements of the worksheets.

11.5.14 Designer Responsibility for Using MSE Wall Worksheets

CDOT MSE wall worksheets contain details such as vertical slip joints, coping, leveling pad, end of wall treatment, waterproofing membrane with drainage, and damage avoidance measures for improving wall and seismic performance. These worksheet details are CDOT minimum requirements consistent with MSE/GRS wall design criteria and policies. The Designer may provide alternatives for approval by Unit Leader to some of these details as identified in the worksheets and in Section 2.16.

Internal, external, and compounded stability are checked based on assumed soil reinforcement shown on the wall worksheets. In addition to using the worksheets, the Project Structural Engineer shall be responsible for site geometry, soil conditions, slope stability checks, and construction sequencing. For a GRS wall with only one specific grade of geotextile with a fixed spacing, the Contractor's selected supplier is only required to meet material certification and shop drawings are not required.

Alternate contracting methods may alter Designer responsibility on a project-specific basis. For example, if a project requires a complete MSE design proposal by a Contractor or appropriate Subcontractor, the Contractor is responsible for all elements of design, including reinforcement grade and placing schedule, and will provide in stamped shop drawings. The aforementioned damage avoidance details still apply.

11.5.15 Designer Responsibility for Using Soil Nail Wall Worksheets

Soil nail walls can either be designed by an in-house or consultant designer in a Design-Bid-Build situation or provided using more of a "Design Build" approach where the Contractor will design the wall based on project requirements. The soil nail worksheets provide generic details for construction as well a project example set. CDOT Geotechnical will typically design in-house soil nail walls in coordination with Staff Bridge. At a minimum, the Designer will provide the required wall alignment and determine the required project requirements. Designer shall show proposed locations of verification tests. Where geotechnical report shows varying strata or for very long walls, more than the minimum of 2 tests should be shown.

11.5.16 Seismic Design Requirements

Seismic analysis for retaining walls is not required unless they are supporting a bridge abutment or liquefaction that will affect the foundation performance is anticipated. Section 3.13 of this BDM provides additional information on seismic design requirements. Current Staff Bridge Worksheets for MSE walls use details for improved seismic performance, thus, if the worksheets are used, AASHTO 11.10.7.4 can be waived.

AASHTO 3.10.9.2, 11.6.5.6, 4.7.4

11.6 DRAINAGE REQUIREMENTS

Backfill material behind abutments and retaining walls shall be well drained and not allow water to collect. If this cannot be accomplished, the abutment and retaining walls should be designed for loads due to earth pressure plus hydraulic pressure due to water in the backfill. Class 1 backfill can have up to 20 percent fines and thus may not be classified as free draining. Design a drain system if using a Class 1 backfill.

AASHTO 11.8.8, 11.9.9, 11.10.8

If the wall or abutment includes conditions or areas that promote the trapping or intrusion of water, such as low point on a sag curve or a drainage inlet, the Designer shall create details to address the issues that may occur. The approach slab drain details used shall allow movement of the abutment while noting that the approach slab drain does not move. Add water sealers, waterproofing membranes, and protection details to the plans.

11.7 SHORING

Shoring is generally not designed by the EOR, but shall be designated in the plans and indicate which shoring areas will require an independent review. Areas that typically need review are those areas that support the roadway or could cause a safety issue.

11.8 REFERENCES

FHWA, 2012, Geosynthetic Reinforced Soil Integrated Bridge System Interim Implementation Guide, Publication No. FHWA-HRT-11-026.

FHWA, 2015, Soil Nail Walls Reference Manual, Publication No. FHWA-NHI-14-007.

APPENDIX 11A - WORKSHEETS FOR EARTH RETAINING WALL TYPE SELECTION

NOTES ON USING WORKSHEETS

1. Factors that can be evaluated in percentage of wall height:

    - Base dimension of spread footing.
    - Embedded depth of wall element into firm ground.

2. Factors that can be described as 'large (high)', 'medium (average)', or 'small (low)':

    Quantitative Measurement
    - amount of excavation behind wall.
    - required working space during construction.
    - quantity of backfill material.
    - effort of compaction and control.
    - length of construction time.
    - cost of maintenance.
    - cost of increasing durability.
    - labor usage.
    - lateral movement of retained soil.

    Sensitive Measurement:
    - bearing capacity.
    - differential settlement.

3. Factors that can be appraised with 'yes', 'no' or 'question (insufficient information)'

    - Front face battering.
    - Trapezoidal wall back.
    - Using marginal backfill material.
    - Unstable slope.
    - High water table/seepage.
    - Facing as load carrying element.
    - Active (minimal) lateral earth pressure condition.
    - Construction dependant loads.
    - Project scale.
    - Noise/water pollution.
    - Available standard designs.
    - Facing cost.
    - Durability.

4. Factors that can be approximated from recorded height:

    - Maximum wall height.
    - Economical wall height

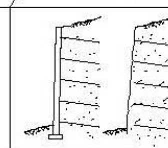
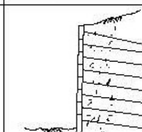
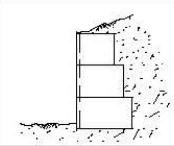
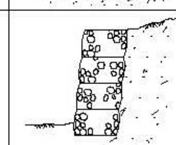
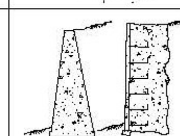
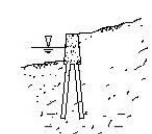
GRAVITY WALL WORK SHEET

MEASUREMENT INDICATORS:

L LARGE H HIGH Y YES

M MEDIUM M MEDIUM ?

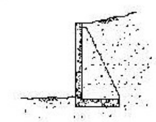
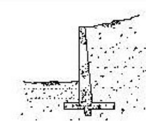
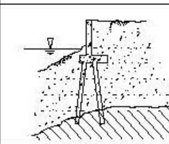
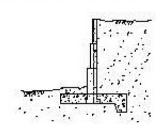
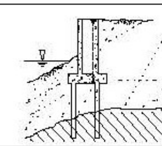
S SMALL L LOW N NO

SYSTEM NAMES AND DESCRIPTIONS			SPATIAL FACTORS						BEHAVIOR FACTORS											ECONOMIC FACTORS										
			FTG. WIDTH (EMB. DEPTH) TO HEIGHTH RATIO	MAXIMUM ECONOMICAL WALL HEIGHTH (FT.)	EXCAVATION BEHIND WALL	WORKING SPACE OF WALL	FRONT FACE BATTERING	TRAPEZOIDAL WALL BACK	SENSITIVITY OF MARGINAL BEARING CAPACITY	SENSITIVITY OF DIFFERENTIAL SETTLEMENT	LATERAL MOVEMENT	MARGINAL BACKFILL MATERIAL	UNSTABLE SLOPE	SCOUR AND FLOOD	HIGH WATER TABLE/SEEPAGE	LOAD CARRYING STRUCTURAL FACING	ACTIVE EARTH PRESSURE FACING	CONSTRUCTION DEPENDENT CONDITION	NOISE/WATER POLLUTION	QUANTITY OF BACKFILL MATERIAL	FILL COMPACTION AND CONTROL	CONSTRUCTION TIME	PROJECT SCALE AFFECTS COST	COST OF MAINTENANCE	AVAILABILITY OF STANDARD DESIGN	LABOR USAGE	FACING AS AN EXTRA COST	SYSTEM DURABILITY PROBLEM	TYPICAL ILLUSTRATIONS
EARTH WALLS	REINF. SOIL WALLS	SELECTED FILLS REINFORCED WITH TENSILE REINFORCEMENTS (EITHER METAL (INEXTENSIBLE) OR GEO-TEXTILE (EXTENSIBLE) BARS, MATS, GRIDS, ETC.)			L M S	L M S	Y ? N	Y ? N	H M L	H M L	L M S	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	L M S	H M L	L M S	Y ? N	H M L	Y ? N	H M L	Y ? N	Y ? N	
EARTH WALLS	NAILED SOIL WALLS	FACING COVERED CUTS WITH UNIFORMLY SPACED, TOP-TO-BOTTOM CONSTRUCTED NAILS.																											
MODULAR WALLS		PRECAST / PREFABRICATED MODULAR WALLS. MOST ARE PROPRIETARY PRODUCTS. SOME PATENTS HAVE RUN OUT.																											
GENERIC WALLS		PREFABRICATED WALL ELEMENTS SUCH AS MASONRY OR CONCRETE BLOCKS. ROUGH ELEMENTS SUCH AS DUMPED ROCKS, GABIONS.																											
MASS CONCRETE WALLS		CAST-IN-PLACE SOLID CONCRETE WALLS OR PRECAST CONCRETE FACINGS ANCHORED IN CEMENT STABILIZED SOIL ZONES.																											
MASS CONCRETE WALLS		C.I.P. REINFORCED CONCRETE WALL ON DEEP FOUNDATION EITHER DRILLED CAISSONS OR PILES.																											

SEMI-GRAVITY WALL WORKSHEET

MEASUREMENT INDICATORS:

L LARGE	H HIGH	Y YES
M MEDIUM	M MEDIUM	?
S SMALL	L LOW	N NO

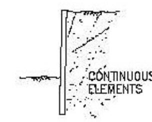
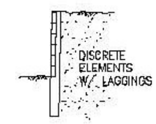
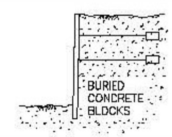
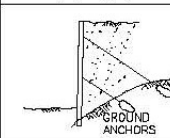
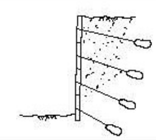
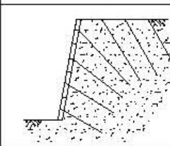
SYSTEM NAMES AND DESCRIPTIONS		FTG. WIDTH (EMB. DEPTH) TO HEIGHTH RATIO	MAXIMUM ECONOMICAL WALL HEIGHTH (FT.)	EXCAVATION BEHIND WALL	WORKING SPACE OF WALL	FRONT FACE BATTERING	TRAPEZOIDAL WALL BACK	SENSITIVITY OF MARGINAL BEARING CAPACITY	SENSITIVITY OF DIFFERENTIAL SETTLEMENT	LATERAL MOVEMENT	MARGINAL BACKFILL MATERIAL	UNSTABLE SLOPE	SCOUR AND FLOOD	HIGH WATER TABLE/SEEPAGE	LOAD CARRYING STRUCTURAL FACING	ACTIVE EARTH PRESSURE CONDITION	CONSTRUCTION DEPENDENT LOADS	NOISE/WATER POLLUTION	QUANTITY OF BACKFILL MATERIAL	FILL COMPACTION AND CONTROL	CONSTRUCTION TIME	PROJECT SCALE AFFECTS COST	COST OF MAINTENANCE	AVAILABILITY OF STANDARD DESIGN	LABOR USAGE	FACING AS AN EXTRA COST	SYSTEM DURABILITY PROBLEM	TYPICAL ILLUSTRATIONS
		SPATIAL FACTORS								BEHAVIOR FACTORS										ECONOMIC FACTORS								
CAST-IN-PLACE	L-WALLS CAN BE USED WITH COUNTERFORTS			L M S	L M S	Y ? N	Y ? N	H M L	H M L	L M S	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	L M S	H M L	L M S	Y ? N	H M L	Y ? N	H M L	Y ? N	Y ? N	
CAST-IN-PLACE	INVERT-L-WALLS CAN BE USED WITH BUTTRESSES. USE WITH TOE COVER FILLS.																											
CAST-IN-PLACE	T-WALLS ON SPREAD FOOTING, USE WITH COUNTERFORTS AND SHEAR KEY IF APPLICABLE.																											
CAST-IN-PLACE	T-WALLS OR L-WALLS ON DEEP FOUNDATIONS. USE EITHER DRILLED CAISSONS OR PILES.																											
PRECAST POST-TENSION	T-WALLS WITH PRECAST POST-TENSIONED STEM AND C.I.P. BASE ON SPREAD FOOTING.																											
PRECAST POST-TENSION	T-WALLS WITH PRECAST POST-TENSIONED STEM AND C.I.P. BASE ON DEEP FOUNDATION, EITHER DRILLED CAISSONS OR PILES.																											

NON-GRAVITY WALL WORK SHEET

SYSTEM NAMES AND DESCRIPTIONS — SPATIAL FACTORS — BEHAVIOR FACTORS — ECONOMIC FACTORS

MEASUREMENT INDICATORS:

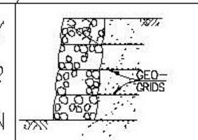
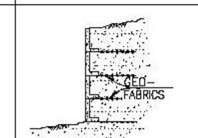
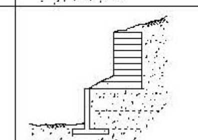
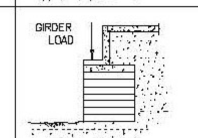
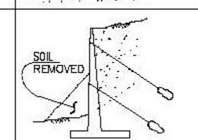
L LARGE	H HIGH	Y YES
M MEDIUM	M MEDIUM	?
S SMALL	L LOW	N NO

System	Description	FTG. WIDTH (EMB. DEPTH) TO HEIGHTH RATIO	MAXIMUM ECONOMICAL WALL HEIGHTH (FT.)	EXCAVATION BEHIND WALL	WORKING SPACE OF WALL	FRONT FACE BATTERING	TRAPEZOIDAL WALL BACK	SENSITIVITY OF MARGINAL BEARING CAPACITY	SENSITIVITY OF DIFFERENTIAL SETTLEMENT	LATERAL MOVEMENT	MARGINAL BACKFILL MATERIAL	UNSTABLE SLOPE	SCOUR AND FLOOD	HIGH WATER TABLE/SEEPAGE	LOAD CARRYING STRUCTURAL FACING	ACTIVE EARTH PRESSURE CONDITION	CONSTRUCTION DEPENDENT LOADS	NOISE/WATER POLLUTION	QUANTITY OF BACKFILL MATERIAL	FILL COMPACTION AND CONTROL	CONSTRUCTION TIME	PROJECT SCALE AFFECTS COST	COST OF MAINTENANCE	AVAILABILITY OF STANDARD DESIGN	LABOR USAGE	FACING AS AN EXTRA COST	SYSTEM DURABILITY PROBLEM	TYPICAL ILLUSTRATIONS
EMBEDDED CANTILEVER WALLS	SHEET PILES, CONTINUOUS DRILLED CAISSONS, SLURRY TRENCHED CONCRETE DIAPHRAGM WALLS.			L M S	L M S	Y ? N	Y ? N	H M L	H M L	L M S	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	L M S	H M L	L M S	Y ? N	H M L	Y ? N	H M L	Y ? N	Y ? N	
	H-PILES, WOOD PILES, CONCRETE PILES, DRILLED CAISSONS WITH LAGGINGS, TWO ROWS OF CLOSE-PACKED CAISSONS.																											
SHALLOW EMBEDDED CANTILEVERED WALLS WITH TIES OR BRACES	SHALLOW EMBEDDED CONTINUOUS OR DISCRETE CANTILEVERED ELEMENTS ANCHORED WITH BURIED CONCRETE BLOCKS CLOSE TO GROUND.																											
	SHALLOW EMBEDDED CONTINUOUS DISCRETE CANTILEVERED ELEMENTS WITH TIEBACKS ANCHORED TO THE STABILIZED ZONE.																											
MULTI-ANCHORED FACING WALLS	PRECAST CONCRETE MULTI-ANCHORED FACINGS WITH TIEBACKS ANCHORED EITHER TO THE STABILIZED ZONE, OR TO THE SELECTED FILL.																											
	CREEPING SLOPES DOWELED WITH CAISSONS OR PILES FOR STABILITY. PRECAST CONCRETE FACINGS ARE ANCHORED TO THE DOWELS.																											

HYBRID WALL WORKSHEET

MEASUREMENT INDICATORS:

L LARGE	H HIGH	Y YES
M MEDIUM	M MEDIUM	?
S SMALL	L LOW	N NO

SYSTEM NAMES AND DESCRIPTIONS	SPATIAL FACTORS						BEHAVIOR FACTORS										ECONOMIC FACTORS										
	FTG. WIDTH (EMB. DEPTH) TO HEIGHTH RATIO	MAXIMUM ECONOMICAL WALL HEIGHTH (FT.)	EXCAVATION BEHIND WALL	WORKING SPACE OF WALL	FRONT FACE BATTERING	TRAPEZOIDAL WALL BACK	SENSITIVITY OF MARGINAL BEARING CAPACITY	SENSITIVITY OF DIFFERENTIAL SETTLEMENT	LATERAL MOVEMENT	MARGINAL BACKFILL MATERIAL	UNSTABLE SLOPE	SCOUR AND FLOOD	HIGH WATER TABLE/SEEPAGE	LOAD CARRYING STRUCTURAL FACING	ACTIVE EARTH PRESSURE CONDITION	CONSTRUCTION DEPENDENT LOADS	NOISE/WATER POLLUTION	QUANTITY OF BACKFILL MATERIAL	FILL COMPACTION AND CONTROL	CONSTRUCTION TIME	PROJECT SCALE AFFECTS COST	COST OF MAINTENANCE	AVAILABILITY OF STANDARD DESIGN	LABOR USAGE	FACING AS AN EXTRA COST	SYSTEM DURABILITY PROBLEM	TYPICAL ILLUSTRATIONS
GENERIC WALLS ANCHORED WITH GEO-FABRIC GRID REINFORCEMENTS. GABION WALLS ANCHORED WITH GEO-GRIDS.			L M S	L M S	Y ? N	Y ? N	H M L	H M L	L M S	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	Y ? N	L M S	H M L	L M S	Y ? N	H M L	Y ? N	H M L	Y ? N	Y ? N	
MODULAR PRECAST L-WALLS ANCHORED WITH GEO-FABRIC GRID REINFORCEMENTS																											
GEO-FABRIC WALL(S) STACKED ON TOP OF T-WALL																											
EITHER INVERTED-L WALL STACKED ON MSE WALL FOR BRIDGE ABUTMENT APPLICATIONS, OR, L-WALL WITH RAIL STACKED ON TOP OF EARTH WALL FOR ROADWAY APPLICATIONS.																											
T-WALL WITH ANCHORS ADDED TO STABILIZED ZONE. USED FOR WALL REMODELING OR REHABILITATION, AND FOR ROADWAY WIDENING APPLICATIONS.																											
T-WALL WITH PRECAST / POST-TENSIONED MODULAR STEM ELEMENTS. ANCHORED WITH GEO-GRID OR WITH REINFORCEMENTS.