



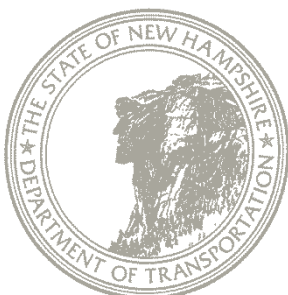
Bridge Design Manual

Chapter 6

Substructure

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Chapter 6 Substructure

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Chapter 6**Substructure**

6.1 General Substructure Considerations

The design of bridge substructures proceeds from preliminary to final design. The abutment, pier, and foundation type are selected during preliminary design based on bridge site information and criteria stated in Bridge Design Manual Chapter 3, Preliminary Design. In some cases, the preliminary design section also considers aesthetic criteria in Chapter 2, Aesthetic Design. The structural design and detailing are completed in final design.

6.1.1 Geotechnical Report

For each bridge site, the Geotechnical Section of the Bureau of Materials and Research or a geotechnical engineering consultant, provides Preliminary Design Recommendations for design and a Final Geotechnical Report. The Final Geotechnical Report typically includes the following:

- Site description.
- Summary of subsurface conditions and materials that may be encountered.
- Design considerations such as type of foundation support (shallow or deep foundation with their related design considerations).
- Engineering properties of the subsurface materials relevant to shallow foundation design, such as nominal bearing resistance, resistance factors, sliding resistance, etc.
- Estimated driven pile and drilled shaft lengths, size, type protection, capacity, resistance factors, rock socket dimensions, etc., as applicable to deep foundation designs.
- Estimated bottom of footing elevation or footing embedment depth below the adjacent ground surface to minimize the potential for settlement or frost heave concerns.
- Slope stability, settlement or other geotechnical factors that may affect the bridge structure.
- Recommendations for ground improvement to increase bearing resistance and reduce settlement.
- Discussion of conditions that may be encountered during construction with recommendations.
- Exploration logs.
- Any other pertinent design and construction information.
- Recommendations for any special notes that may be required on the contract drawings or any specific contractual provisions to address specific geotechnical conditions.

The successful integration of the geotechnical design recommendations into the bridge design will require close coordination between the Geotechnical Section and the Bureau of Bridge Design.

6.1.2 Substructure and Foundation Loads

In all cases substructure components need to be designed for vertical and lateral loads, settlement, stability, and economy considering the complete bridge structure. The design process requires an iterative collaboration to provide cost-effective constructible substructures. Input is required from multiple sources and departments. Prior to the design of the substructure, the bridge

designer must have knowledge of the environmental, climatic, hydraulic and loading conditions expected during the life of the proposed bridge.

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

Substructure elements are designed to carry all of the loads specified in *AASHTO LRFD Specifications* and Chapter 4 of this manual. Selecting the controlling load conditions requires good judgment to minimize design time. All anticipated dead load (*DC & DW*) conditions shall be accounted for during a substructure design. Sidesway effect shall be included where it tends to increase stresses. For live loads (*LL*), the dynamic allowance (*IM*) shall be applied in accordance with *AASHTO LRFD 3.6.2* and is not included in the design of buried elements of the substructure. Portions of abutments in contact with soil are considered buried elements.

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

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

A. Service Limit State

Foundations, including abutments and piers, shall be evaluated at the Service Limit State considering the effects of scour due to the design flood. In addition to other serviceability criteria, tolerable deflections and rotations are calculated at the Service Limit State. The overall stability investigation shall be in accordance with *AASHTO LRFD 11.6.2.3*.

Total tolerable movements shall be determined on a project to project basis and must consider the following:

- Structure type and function
- Consequences of the movements and tolerances to differential movements
- Structure detailing (such as roadway joints and bearings)
- Economy
- Rideability

B. Strength Limit States

The Strength Limit States shall consider the structural and geotechnical resistance of the foundation components as well as the loss of lateral and vertical support due to scour. Structural and geotechnical resistance shall include axial, lateral, and flexural resistance.

For spread footings, the Strength Limit States shall also consider bearing resistance, overturning and sliding.

For deep foundations (driven piles, micro piles, and drilled shafts), the strength limit states shall also consider single and group axial compression resistance, single and group uplift resistance, and single and group lateral resistance.

C. Extreme Event Limit State

Structures must remain stable for an Extreme Event II limit state that considers scour due to the check flood. This limit state need not include ice loads, vehicle collision loads and vessel collision loads simultaneously. See Chapter 2, Section 2.7.7 for additional information regarding scour analysis.

Structures must also remain standing for the Extreme Event I and remaining Extreme Event II limit states without consideration for the effects of scour.

D. Loads and Load Factors

Table 6.1.2-1 is a general application of minimum and maximum load factors as they apply to a generic footing design. Footing design must select the maximum or minimum load factors for various modes of failure for the Strength and Extreme Event Limit States. For each load combination both positive and negative effects shall be investigated. In load combinations where one force effect decreases another effect, the minimum value shall be applied to the load reducing the force effect.

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

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

Strength I Limit State using maximum load factors for the expansion abutment produces the maximum soil stress in the toe. Since a single load factor shall be used for each load type the toe, heel, and stem shall all have the same factor. Use of a maximum load factor for the DC and EV loads produces the maximum soil pressure but also produces the maximum resisting moment and shear since the overburden soil and footing toe resist the soil pressure. Use of a minimum load factor for the DC and EV loads reduces soil pressure but produces the minimum resisting moment or shear. Whether a maximum or minimum load factor produces the maximum moment and shear is not always obvious, resulting in the need to analyze each possible combination of maximum and minimum load factors for all the loads.

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

**Possible Load Factor Combinations
for a Spread Footing**

(Note: Other combinations may need to be analyzed.)

6.1.3 Constructability

The design of all substructure types must consider the effects of the anticipated method of construction, including the construction sequencing. Such considerations shall consist of, but not be limited to: the need for shoring, the use of cofferdams, tremie seals, dewatering, excavation stability, downdrag considerations for driven piles, and the need for permanent or temporary casing for drilled shafts or micropiles.

6.1.4 Foundation Type Selection

Each substructure location of a bridge needs to be evaluated for the appropriate foundation type. It is not uncommon to have different foundation types (i.e. shallow and deep) for the various substructure units in a bridge. The use of different deep foundation elements is also possible, such as using driven piles for one substructure and drilled shafts for another substructure of the bridge. The following items need to be assessed in the selection of a site-specific foundation type:

- Bridge configuration and substructure location.
- Magnitude and direction of loading.
- Depth to suitable bearing material and available bearing resistance.
- Potential for undermining or scour.
- Seismic site classification and liquefaction assessment.
- Embedment requirements for frost protection.
- Performance requirements, including deformation (settlement), global stability and resistance to uplift, and lateral, sliding and overturning forces.
- Ease and cost of construction.
- Environmental impact of construction, including need and costs of temporary structures.
- Site constraints, including restricted right-of-way, overhead and lateral clearance, construction access, existing traffic, utilities, and proximity of existing or vibration-sensitive structures.

Shallow foundations, such as spread footings, are typically initially considered to determine if this type of foundation is technically and economically viable. Often foundation settlement and lateral loading constraints govern over bearing resistance. Other significant considerations for selection of shallow foundations include requirements for cofferdams, tremie seals, dewatering, temporary excavation support, overexcavation of unsuitable material, slope stability, available time to dissipate consolidation settlement prior to final construction, scour susceptibility, environmental impacts and water quality impacts.

When shallow foundations are not viable for the substructure site, ground improvement methods and deep foundations are then considered. Ground improvement methods such as aggregate piers, vibro-compaction or simply excavate and replace, if they are viable for the site, can improve conditions sufficiently to allow construction of a spread footing. Deep foundations can transfer foundation loads through shallow deposits to competent deeper bearing material. Deep foundations are generally considered to mitigate concerns about scour, lateral spreading, excessive settlement, and other site constraints.

Common types of deep foundations for bridges include driven piles and drilled shafts. Micropiles are less common and are usually only applicable when certain conditions exist. Specialty deep foundation types such as augercast piles, etc. could be considered, but are also only applicable when certain conditions exist. Driven piles are by far the most common type of deep foundation because competent bearing soils or bedrock are usually within reasonable depths, they can be

installed relatively quickly, and their capacity can be tested directly. Drilled shafts may be advantageous for uplift or lateral resistance, where obstructions may result in premature pile driving refusal, where piers need to be founded in areas of shallow bedrock, or where deep water exists. A drilled shaft may be more cost effective than driven piling when the drilled shaft and pier column can be constructed as a continuous unit to eliminate the need for a pile footing and cofferdams. Drilled shaft capacities are difficult to test and integrity testing of the shaft is used instead to assure the shaft is constructed properly. Micropiles may be the best foundation alternatives where headroom is restricted or foundation retrofits are required at existing substructures, or site access limits more traditional foundation systems.

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6.2 Spread Footing

6.2.1 Footing Design Considerations

The following design considerations apply to shallow foundations:

- Scour must not result in the loss of bearing or stability.
- Frost protection must be provided in the form of minimum embedment.
- External or surcharge loads must be accounted for in the design.
- Deformation (settlement) and angular distortion must be within tolerable limits.
- Bearing resistance must be sufficient.
- Overturning requirements must be satisfied.
- Sliding resistance must be sufficient.
- Overall (global) stability must be satisfied.
- Uplift resistance must be sufficient.
- The effects of groundwater must be mitigated and/or considered in the design.

6.2.2 Minimum Footing Depth

The geotechnical report shall provide guidance on the minimum embedment for spread footings that takes into consideration frost protection and adequate bearing materials. The hydraulic report shall provide information regarding scour potential and maximum scour depth.

A. Scour Vulnerability

Scour is a hydraulic erosion process caused by flowing water that lowers the grade of a water channel or riverbed. For this reason, scour vulnerability is an essential design consideration for shallow foundations. Scour can undermine shallow foundations or remove sufficient overburden to redistribute foundation forces, causing foundation displacement and detrimental stresses to structural elements. Excessive undermining of a shallow foundation leads to gross deformation and can lead to structure collapse.

Foundations for new bridges and structures located within a stream or river floodplain shall be located at an elevation below the maximum scour depth that is identified by the hydraulics report. In addition, the foundation shall be designed deep enough such that scour protection is not required. If the maximum calculated scour depth elevation is below the practical limits for a shallow foundation, consideration shall be given to a deep foundation system for support of the structure. See Chapter 2, Section 2.7.7 for additional information regarding scour analysis.

B. Frost Protection

Spread footings must be embedded below the maximum depth of frost potential (frost depth). This embedment is required to prevent foundation heave due to volumetric expansion of the foundation subgrade from freezing and/or to prevent settling due to loss of shear strength from thawing.

In New Hampshire, the bottom of spread footings shall be placed at a *minimum* depth stated below, unless recommended otherwise by the geotechnical engineer:

- 5-ft. (1.5-m) below lowest finished grade if the footing is bearing on soil or structural fill (measured perpendicular to finished grade).
- 4-ft. (1.2-m) below lowest finished grade if the footing is supported on piles (measured perpendicular to finished grade).

- Deep enough to allow the full depth of foundation protection (for riprap) over the top of the footing.

6.2.3 Spread Footing Design

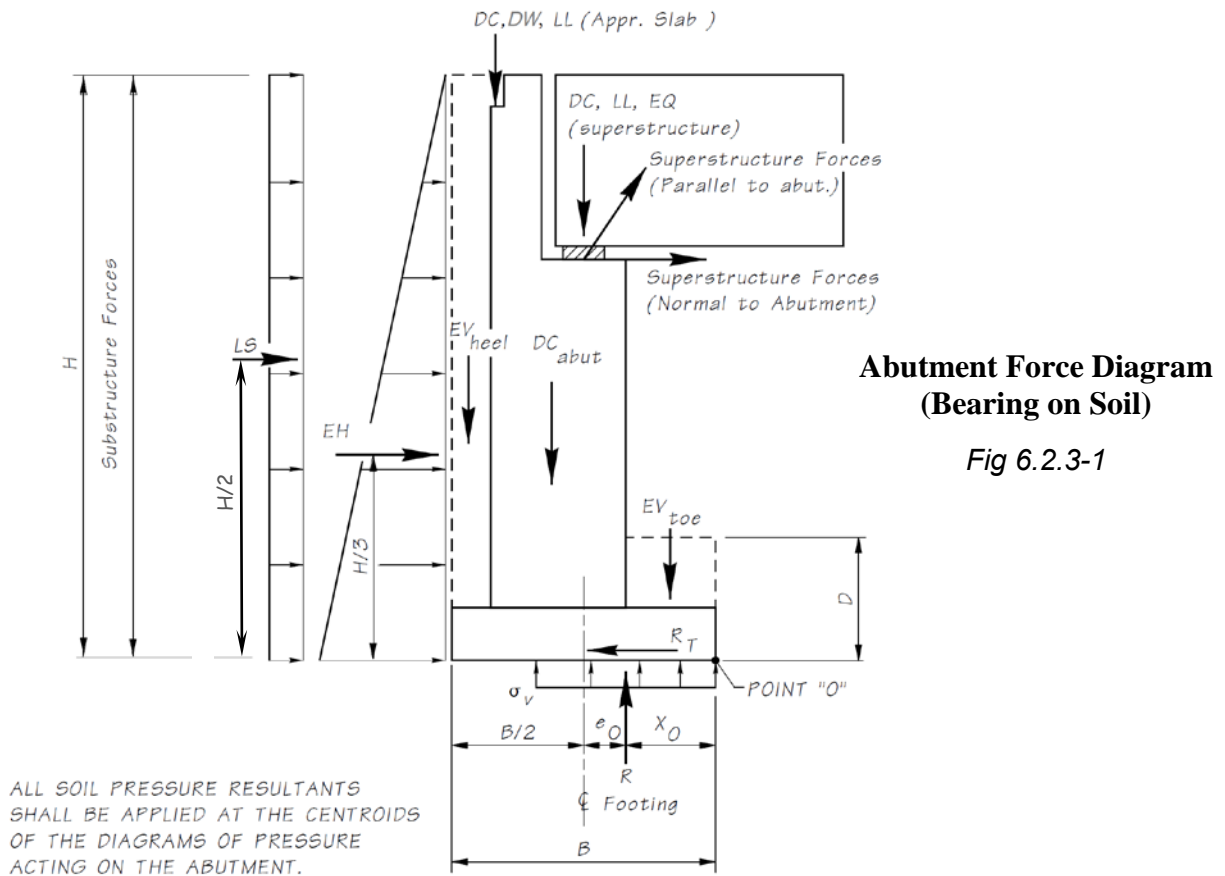
The following section is oriented toward abutment spread footing design. Spread footing designs for intermediate piers or other applications use the same concepts with the appropriate structural analysis.

A spread footing foundation will be evaluated, considering expected scour, for the following failure conditions:

1. Bearing Resistance – Strength Limit States
2. Settlement – Service I Limit State
3. Sliding Resistance – Strength Limit States
4. Load Eccentricity (Overturning) – Strength Limit States
5. Overall Stability – Service I Limit State
6. Structural Resistance – Service I and Strength Limit States

A. Abutment Spread Footing Force Diagram

Figure 6.2.3-1 shows the forces that act on abutment footings. Each limit state design check will require calculation of a reaction (R) and the location (X_0) or eccentricity (e_0).



**Abutment Force Diagram
(Bearing on Soil)**

Fig 6.2.3-1

B. Bearing Stress

The factored bearing resistance (q_R), is often the controlling parameter for which a footing is sized to assure adequate performance. The nominal bearing resistance (q_n) is influenced by several factors, including the footing length to width ratio, the embedment depth-to-width ratio, the strength and compressibility of the foundation support material, as well as the applied loading inclination and eccentricity. The Geotechnical Report may document some assumptions or qualifications on the use of the recommended values for design that do not fully account for some of the factors discussed above for a particular project. As such, in cases where the design footing dimensions or loadings are not typical, the designer may wish to contact the geotechnical engineer to verify or recalculate the nominal bearing resistance using the proposed configuration and loads.

The nominal bearing resistance (q_n) is the resistance available from the soil to support the footing assuming it is uniformly loaded. Since a true uniform loading is rarely the case for design, an equivalent uniform factored bearing stress (σ_v) is calculated and compared to the factored bearing resistance (q_R) to determine the required footing size of a footing. For footing designs on soil, bearing stress (σ_v) is based on a uniformly distributed bearing pressure. For footing designs on rock, the bearing stress (σ_v) is based on a triangular or trapezoidal bearing pressure distribution. See Figure [6.2.3-1](#) for typical loads and eccentricity and *AASHTO LRFD 10.6.1.4, 10.6.3, 11.6.3.2* and *Figures 11.6.3.2-1 & 2* for bearing pressure distribution.

Structures shall be designed not to exceed the factored bearing resistance (q_R) under footings in accordance with the recommendations of the Geotechnical Engineer. The bearing resistance is checked at the strength limit state. The appropriate strength limit states are I, III and V. The maximum bearing stress will be found by applying the maximum load factors to each applicable load. The design nominal bearing resistance (q_n) and geotechnical resistance factor (ϕ_b) shall be stated in the project notes on the contract plans.

For design of footing bearing pressures, as a minimum the following load case shall be considered:

- Bridge is complete and approach slab is in place. Use maximum load factor for all loads.

C. Failure by Sliding

Sliding failure occurs if the force effects due to the factored horizontal component of loads exceed the more critical of either the factored sliding resistance of the soils or the factored sliding resistance at the interface friction between the soil and the foundation along the base of the foundation. Structures shall be designed not to exceed the sliding resistance (R_R) under footings in accordance with the recommendations of the Geotechnical Engineer. The value of the internal friction angle of the bearing soil (ϕ_f) and the shear resistance factor (ϕ_t) will be provided in the Geotechnical Report.

Sliding resistance is determined differently depending on whether the spread footing is founded on granular soil, cohesive soil, or rock. For granular soils, the nominal sliding resistance (R_t) is calculated as the vertical resultant (V) times the tangent of the internal friction angle of soil (ϕ_f), for concrete cast against soil.

$$R_t = (V)(\tan\phi_f) \quad (\text{for concrete against soil})$$

$$R_t = (V)(0.8\tan\phi_f) \quad (\text{for precast concrete footing})$$

For cohesive soils, sliding resistance (R_t) is calculated as cohesion times the effective footing width (B). Lower strength clays require special attention to ensure adequate sliding resistance. Cohesive soil can be excavated and replaced with structural fill to improve sliding resistance.

The Strength Limit States are used for this check. Since the resistance is based on the reaction, minimum factors are used for all vertical loads and the vertical weight of the live load surcharge is ignored on the footing heel. The maximum factors are used with the earth pressure forces (horizontal and vertical).

Except for the Extreme Limit States, the effects of passive soil resistance in front of the footing shall not be included as part of the shear resistance required to resist sliding because the soil may be removed due to scour or during future construction. The effects of passive soil resistance may be included for the Extreme Limit States.

Sliding resistance on rock is evaluated in a similar manner as that for a cohesive or a granular soil, depending on rock type and weathering. Planes of weakness in rock can control the design. Keying the footing into rock and the use of vertical steel dowels drilled into bedrock can provide substantial added sliding resistance in addition to the friction or adhesion along the base of the footing. The Geotechnical Engineer will provide guidance on whether the spread footing shall be keyed into bedrock.

D. Global Stability

Overall stability analysis is performed by the Geotechnical Engineer. The global stability of spread footings on or near an inclined slope, or close to an embankment, excavation, or retaining wall, shall be evaluated by limiting equilibrium methods of analysis in accordance with *AASHTO LRFD 10.5.2.3 and 10.6.2.5*. Global stability of walls and abutments shall be investigated at the Service Limit State.

E. Settlement

Settlement analysis is a Service Limit State and is performed by the Geotechnical Engineer in accordance with *AASHTO LRFD 3.12.6, 10.5.2.2, 10.6.2.2 and 10.6.2.4*. Tolerable settlement should generally be limited to a maximum of 1 inch at each substructure after girder placement. If the Geotechnical Engineer estimates that the differential settlement is ½ inch or less, the bridge designer may usually ignore the effects of differential settlement in the structural design of the bridge. Generally, due to the methods used by NHDOT to proportion foundations, settlements are within a tolerable range and, therefore, force effects due to differential settlement need not be investigated.

If varying settlement conditions exist the following effects shall be considered:

- **Structural.** The differential settlement of substructures causes the development of force effects in continuous superstructures. These force effects are directly proportional to structural depth and inversely proportional to span length, indicating a preference for shallow, long-span structures.
- **Joint Movements.** A change in bridge geometry due to settlement causes movement in deck joints that shall be considered in their detailing, especially for deep superstructures.
- **Profile Distortion.** Excessive differential settlement may cause a distortion of the roadway profile that may be undesirable for vehicles traveling at high speed.
- **Appearance.** Viewing excessive differential settlement may create the impression of an unsafe structure.
- **Mitigation.** Ground improvement techniques may be used to improve the subsurface conditions below a spread footing to address bearing resistance deficiencies, liquefaction potential, and overall settlement or differential settlement concerns. These techniques include but are not limited to:

- Vibrocompaction or dynamic compaction
- Over-excavation and replacement with structural fill or subfooting concrete
- Preloading or surcharging with a waiting period
- Construction of stone columns or aggregate piers, high modulus concrete columns
- Compaction grouting

F. Load Eccentricity

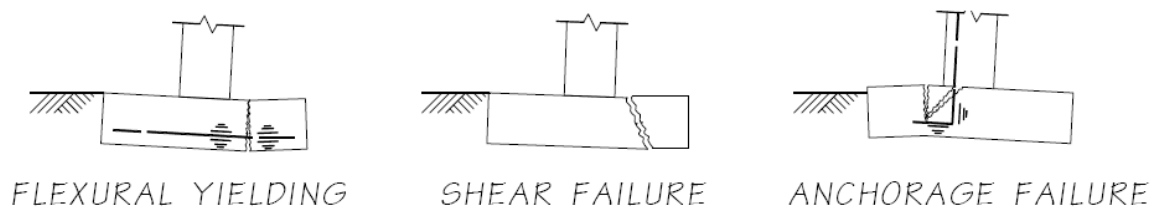
For foundations on soil, the location of the resultant of the reaction forces shall be within the middle 2/3 of the base width. For foundations on rock, the location of the resultant of the reaction forces shall be within the middle 9/10 of the base width (*AASHTO LRFD 11.6.3.3*).

$$e = B/2 - \frac{(\sum M_{RES} - \sum M_{O.T.})}{\sum F_{RES}}$$

G. Concrete Design

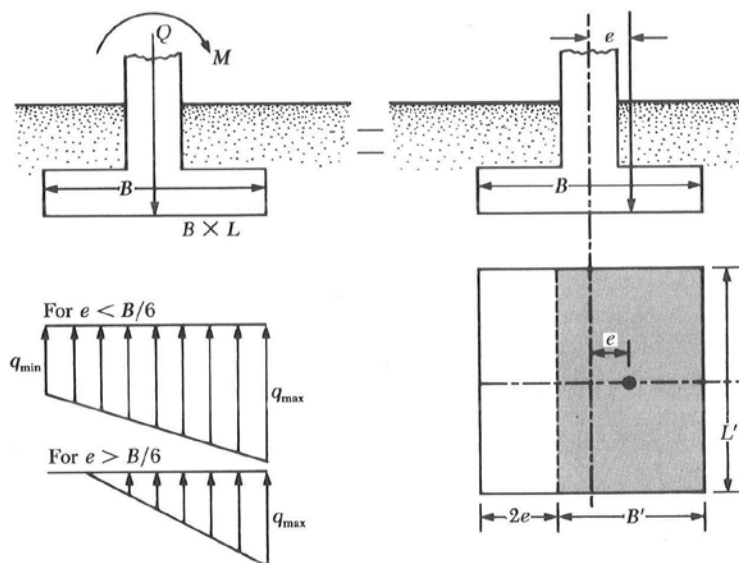
Footing design shall be in accordance with the general concrete design of *AASHTO LRFD Chapter 5*. Spread footings shall be designed for flexure, flexural shear, and punching shear as appropriate for each particular project. Figure [6.2.3-2](#) illustrates the modes of failure checked in the footing concrete design.

Unlike settlement and bearing resistance checks, where the bearing stress is based on a uniformly distributed bearing pressure for bearing on soil and a triangular or trapezoidal bearing pressure distribution bearing on rock, a triangular or trapezoidal shaped soil stress distribution is assumed for the design of structural elements, regardless if the footing bears on soil or rock. This assumption will provide the maximum moments and shears in the footing. (*AASHTO LRFD 10.6.5 & 11.6.3.2*) See Figure [6.2.3-3](#) for a triangular and trapezoidal stress distribution.



Spread Footing Modes of Failure

Fig 6.2.3-2



Eccentrically Loaded Foundation

Fig 6.2.3-3

(Fundamentals of Geotechnical Engineering, 3rd Ed.)⁴

- Footing Thickness and Shear.** The minimum footing thickness shall be 2-ft. (0.6-m). Footing thickness may be governed by the development length of the column dowels or hooked bars, or by concrete shear requirements (with or without reinforcement). If concrete shear governs the thickness, it is the engineer's judgment, based on economics, as to whether to use a thick footing unreinforced for shear or a thinner footing with shear reinforcement. Generally, shear reinforcement should be avoided but not at excessive cost in concrete, excavation, and shoring requirements. Where stirrups are required, place the first stirrup at $d/2$ from the face of the column or pedestal. For large footings, consider discontinuing the stirrups at the point where $V_u = V_c$. Typically, NHDOT designs increase the footing thickness if shear governs.
- Footing Force Distribution.** The maximum shear stress in the footing concrete shall be determined based on a triangular or trapezoidal bearing pressure distribution (*AASHTO LRFD 5.13.3.6, 10.6.5*). This is the same pressure distribution as for footing on rock.
- Vertical Reinforcement (Column or Wall).** Vertical reinforcement shall be developed into the footing to adequately transfer loads to the footing. Vertical rebar shall be bent 90° and extend to the bottom of the bottom mat of footing reinforcement (J-bar). This facilitates placement and minimizes footing thickness. Bars shall be developed in accordance with *AASHTO LRFD 5.11.2*.

The outside radius of the J-bar bend shall be 8-in. (200-mm). This meets the minimum bar bend (measured on the inside of the bar) of $8(d_b)$ for up to and including a #11 (#36) bar. An 8.5-in. (216-mm) outside radius or greater would be required for a #14 (#43) bar (*AASHTO 5.10.2.3*). The 8-in. (200-mm) outside radius, which is larger than the standard bar bend, shall be used in locations with large moments (e.g., knee of a rigid frame, abutments, retaining walls, or piers). Standard bar bends can be used for detailing other reinforcement.

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

- **Bottom Reinforcement (Toe Design).** Concrete design shall be in accordance with *AASHTO LRFD Specifications*. Reinforcement shall not be less than #5 (#16) bars at 12-in. (300-mm) on center to account for uneven soil conditions and shrinkage stresses.
- **Top Reinforcement (Heel Design).** Top reinforcement shall be used in any case where tension forces in the top of the footing are developed. Where columns and bearing walls are connected to the superstructure, sufficient reinforcement shall be provided in the tops of footings to carry the weight of the footing and overburden assuming zero pressure under the footing. This is the uplift earthquake condition described under “Superstructure Loads.” This assumes that the strength of the connection to the superstructure will carry such load. Where the connection to the superstructure will not support the weight of the substructure and overburden, the strength of the connection may be used as the limiting value for determining top reinforcement. For these conditions, the *AASHTO LRFD* requirement for minimum percentage of reinforcement will be waived. Regardless of whether or not the columns and bearing walls are connected to the superstructure, a mat of reinforcement shall normally be provided at the tops of footings.

The critical group combination for the heel design will be the load factors producing the minimum axial loads with maximum eccentricities resulting in the minimum soil pressure.

- **Materials.** Typical concrete footing designs shall use a concrete compressive strength, $f'_c = 3,000\text{-psi}$ (2.10 kg/mm^2), Concrete Class B Footings. Greater concrete compressive strengths require the approval of the Design Chief. All reinforcing steel shall be grade 60 black bars.
- **Detailing.** All footing reinforcing shall be detailed and dimensioned from working points. This reduces the chance that construction tolerances for pile driving and concrete placement will impact the final location of substructure components.
 - Layout lines shall be the centerline of bearings or the face of frame leg, and the face of wing lines. The intersections of these lines at each corner of the abutment or frame shall be designated as the “working point” (wp).
 - The minimum spacing of reinforcing steel in either direction is 6-in. (150-mm) on center; the maximum spacing is 12-in. (300-mm) on center.
 - Reinforcement shall not be less than #5 (#16) bars at 12-in. (300-mm) on center.
 - Footing widths shall be multiples of 6-in. (150-mm) to allow for placing reinforcement at 6-in. (150-mm) intervals with 3-in. (75-mm) clear cover.
 - The minimum vertical reinforcement (J-bars) shall be #5 (#16) bars at 12-in. (300-mm) spacing in each direction. J-bars shall be placed near side and far side of all abutments, wingwalls and wall piers.
 - Specify all footing elevations at the bottom of footing, except in bedrock where the top of footing elevation and minimum and maximum footing thickness shall be indicated.
 - Footing plan dimensions shall be laid out in a manner that will allow support of the formwork used to construct the substructure elements above it. This is accomplished by extending the footing at least 6-in. (150-mm) beyond the vertical face of the wall or stem.
 - See [Appendix 6.2-B1](#) for examples of spread footings details.

H. Miscellaneous

- **Steps.** Footing steps may be used and shall follow the following criteria:
 - Maximum step height shall be the thickness of the footing unless bearing on sound bedrock.
 - The bottom of the step shall be sloped on a 2H:1V when bearing on competent soil, but may be vertical when bearing on bedrock.
 - Step lengths shall be a minimum of 10-ft. (3-m).
 - Reinforcing shall tie the footing together between steps.
 - A construction joint may be used at the step.
- **Joints.** Footings do not generally require construction joints. Contraction and expansion joints shall not be used unless approved by the Design Chief.
- **Foundation Seal.** A foundation seal is a mass of unreinforced concrete placed by tremie or pumping under water inside a cofferdam. The foundation seal is designed to withstand the hydrostatic pressure produced at the bottom of the seal when the water above is removed. Once the cofferdam is constructed, excavated to the bottom of the seal elevation and foundation piles driven if applicable), the foundation seal concrete (Class T) is then placed under water by tremie pipe or by pumping. The displaced water is pumped out of the cofferdam (usually to a holding tank or settlement area). Once the concrete has set, the foundation seal provides a dry platform for construction of the structure footing.

Design of the foundation seal consists of determining a concrete thickness that will counterbalance the hydrostatic pressure with an adequate factor of safety. The foundation seal should also extend to the required bearing layer, if the foundation is configured as a spread footing. Design is done under the service limit state. Lateral forces from stream flow pressure are resisted by penetration of the sheet piling below the streambed elevation and by the bracing inside the cofferdam. The cofferdam design is the responsibility of the Contractor. See [Appendix 6.2-A1](#) for a foundation seal design example.

A foundation seal design shall follow the following criteria:

- The foundation seal should be a minimum of 4-ft. (1.2-m) thick unless approved otherwise by the Design Chief.
- Foundation seals shall be designed to a safety factor of 1.1 using the bottom of seal elevation and the cofferdam vent elevation for the seal, which must be stated in the plans. The uplift resistance of any foundation piles present in the foundation seal, but not any cofferdam sheet piling, can be included in the factor of safety calculation of the foundation seal. Concrete density shall be assumed to be 145-lbs/ft³ (22.8-kn/m³).
- When a spread footing foundation is used above the seal, the foundation seal should have a minimum width equal to the width of the footing plus the added width calculated by extending a 1H:2V line from the edge of footing on all sides to the bottom of seal. The sliding resistance between the bottom of footing and the top of seal should also be checked. The bearing resistance at the bottom of foundation seal shall also be checked.
- When deep foundations are used, the foundation seal shall be shown 2-ft. (600-mm) wider than the footing dimensions on all sides.
- Where a foundation seal is used and there are no piles, the bottom footing reinforcement shall be 3-in. (150-mm) above the bottom of footing.
- Where a foundation seal is used and there are piles extending through the seal, the reinforcement shall be placed above the top of piling.

6.3 Deep Foundations

6.3.1 General

When competent bearing soil is not present near the ground surface and where ground improvement is not an option, structure loads must be transferred to a deeper soil stratum or bedrock by using a deep foundation such as driven piles, drilled shafts or micropiles.

The primary functions of a deep foundation are:

- To transmit the load of the structure through a stratum of poor bearing resistance to one of adequate bearing resistance.
- To eliminate unacceptable settlement.
- To transfer loads from a structure through a scour zone to stable underlying strata.
- To anchor structures subjected to hydrostatic uplift or overturning forces.
- To resist lateral loads from earth pressures or other loading conditions.

6.3.2 Deep Foundation Types

A. Drilled Shafts

Drilled Shafts are large diameter (greater than 24-in. [600-mm] drilled holes, usually extending into bedrock for support, filled with reinforcing steel and concrete. The reinforcement is usually in the form of a perimeter cage. Drilled shafts provide significant axial and lateral capacity. Drilled shafts can be cased or uncased, but are never battered. If not sacrificial, the casing can be used in the strength determination of the drilled shaft, which can significantly increase its axial and lateral capacity. Drilled shafts are used for specialized conditions, so their use is uncommon.

B. Precast, Prestressed Concrete Piles

Precast, prestressed concrete piles are octagonal, or square in cross-section and are prestressed to allow longer handling lengths and to resist driving stresses. This pile type is generally not used for bridge structures because of difficulties with handling, splicing and other factors.

C. Steel H-Piles

Steel H-piles are used where there are hard layers that must be penetrated in order to reach an adequate end-bearing stratum, where boulders and cobbles may be present in the soil profile, or when the piles are expected to reach bedrock for end bearing. These conditions are very common across the state, making steel H-piles the most prevalent pile type used in deep foundations. H-piles can act as friction piles due to their large surface area. Steel H-piles are also well-suited for integral or semi-integral abutment designs. The pile tips are usually protected with a hardened pile point. Steel H-piles are relatively easy to splice.

D. Steel Sheet Piles

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

E. Steel Pipe Piles

Steel pipe piles can be used as open-end or closed-end piles, the latter having a plate or tapered point attached at the tip. Pipe piles are used where both lateral and transverse forces must be

resisted since their strength properties are uniform in all directions of loading. Closed-end pipe piles can be filled with reinforcing steel and concrete. Hard layers to be penetrated, or boulders or cobbles present in the soil profile, may present difficult installation conditions for pipe piles. Pipe piles can be protected with a hardened pile point. Pipe piles can act as friction piles due to their large surface area.

F. Timber Piles

Timber piles may be untreated or treated. Untreated piles are used only for temporary applications or where the entire pile will be permanently below the water line. Where composite piles are used, the splice must be located below the permanent water table. If doubt exists as to the location of the permanent water table, treated timber piles shall be used. Timber piles are currently not considered for use in bridge structures, but may be encountered in older structures or used by municipalities.

F. Micropiles

Micropiles are small diameter (typically 12-in. [300-mm] or less) drilled holes, usually extending into bedrock for support, filled with reinforcing steel and concrete. The reinforcement is usually a centralized steel bar, which provides significant axial capacity, but limited lateral capacity in bending. Micropiles can be cased or uncased and battered like a driven pile. If not sacrificial, the casing can be used in the strength determination of the micropile, which can increase its axial and lateral capacity. Micropiles are used for specialized conditions, so their use is very uncommon

G. Piles can be classified as follows:

- **Friction Pile:** A driven pile whose support capacity is derived principally from soil resistance mobilized along the side of the embedded pile in proportion to the surface area of the pile. Also, a drilled shaft or micropile that relies on side friction in soil or in a rock socket for support.
- **End Bearing Pile:** A driven pile whose support capacity is derived principally from the bearing resistance of the foundation material on which the pile tip rests in proportion to the cross section size of the pile. Also, a drilled shaft or micropile that relies on end bearing resistance on soil or in a rock socket for support.
- **Combination Friction and End Bearing Pile:** A driven pile that derives its capacity from contribution of both friction resistance mobilized along the embedded pile length and end bearing developed at the pile tip. Displacement piles are generally combination piles.
- **Test Pile:** A pile, which should be installed vertically, designated for static or dynamic testing. A test pile can be a support pile intended to be incorporated into the foundation.
- **Production Pile:** A term used for a pile not designated as a test pile, although a production pile could be tested if ordered.
- **Battered Pile:** A friction, end bearing, or combination pile driven at an angle from vertical to provide higher resistance to lateral loads.

6.3.3 Selection of Pile Types

The selection of a pile type for a given foundation application is made on the basis of subsurface conditions, stability under vertical and horizontal loading, settlement potential, pile installation constraints, substructure type, cost comparison, length of pile, and any other factors affecting the foundation or presented by the Geotechnical Engineer. Frequently more than one type

of pile meets the physical and technical requirements for a given site. The performance of the entire structure controls the selection of the foundation. Primary considerations in choosing a pile type are the evaluation of the foundation materials and the selection of the substratum that provides the best foundation support.

Piling is also used at piers where scour is possible, even though the streambed may provide adequate support without piling. In some cases, it is advisable to place footings at greater depths than minimum and specify a minimum pile penetration to guard against excessive scour beneath the footing and piling. Shaft resistance (skin friction) and lateral support within the maximum depth of scour is assumed to be zero. When a scour depth is estimated, this area of lost frictional support must be taken into account in the pile driving operations and capacities.

Subsurface conditions at the structure site also affect pile selection and details. The presence of artesian water pressure, soft compressible soil, cobbles and/or boulders, loose/firm uniform sands or deep water all influence the selection of the optimum type of pile for deep foundation support. There may also be difficulty driving closely spaced displacement piles in denser sands within cofferdams, due to compaction of the in-situ sand during consecutive pile placement.

If boulders or cobbles are anticipated within the estimated length of the pile, consideration shall be given to the potential for pile damage. Options include increasing the pile cross-sectional area, using pile point protection or using a drilled deep foundation

Environmental factors may be significant in the selection of the pile type. Environmental factors include areas subject to high corrosion, abrasion due to moving debris or ice, wave action, deterioration due to cyclic wetting and drying, strong currents, driving through hazardous materials could provide a wick to other soil layers, and gradual erosion of riverbed due to scour. Concrete piles are susceptible to corrosion when exposed to alkaline soil or strong chemicals, especially in rivers and streams. Steel piles can suffer serious electrolysis deterioration if placed in an environment near stray electrical currents or located in a zone subject to cyclical wetting and drying, such as in a tidal river.

Foundations located in deep and/or swiftly moving water may be subject to large lateral forces requiring piles that can sustain large bending forces. Battered piles can be used for high lateral loading conditions. For extremely high lateral load conditions, large diameter drilled shafts may be more appropriate.

Piles may be installed vertical or battered. The path of battered piles shall be checked to insure the piles remain within the right of way and do not interfere with piles from adjacent and existing substructure units, nor conflict with temporary sheeting or cofferdams.

6.3.4 Pile Spacing

A wide spacing of piles reduces heaving and possible uplifting of the pile, damage by tension due to heaving and the possibility of crushing from soil compression. Wider spacing more readily permits the tips of later-driven piles in the group to reach the same depths as the first piles and results in more consistent bearing pressure and settlement. Large horizontal pressures are created when driving in relatively incompressible strata, and damage may occur to piles already driven if piles are too closely spaced. In order to account for this, use a minimum center-to-center spacing of 2.5 times the pile diameter or 30-in. (760-mm) as stated in *AASHTO LRFD 10.7.1.2*. The maximum pile spacing shall be 10-ft. (3-m) or as approved by the Design Chief.

Care shall be exercised in locating piles to avoid interference with other piles, both in the final position and during the driving process. If a plumb pile in the back row is located directly behind a battered pile in the front row, the Contractor may be forced to plan his sequence of pile

driving and cut-offs in a less efficient manner than if the back row of piles were staggered with the front row.

6.3.5 Battered Piles

Lateral loadings applied to pile foundations are typically resisted by pile bending resistance or by battering selected piles, or a combination of these two methods. The magnitude of lateral loads to be resisted will determine whether battered piles are required.

Use of battered piles is the most common way to obtain lateral resistance with minimal lateral deflection. Batters typically range from 1H:12V to 4H:12V. Piles with batters in excess of this become very difficult to drive, and the bearing values become difficult to predict. The path of battered piles shall be checked to ensure the piles remain within the right of way, do not interfere with piles from adjacent and existing substructure units, or conflict with temporary sheeting or cofferdams.

In seismic regions, the use of battered piles is not normally desirable as they produce relatively stiff foundations (as compared to vertical piles) that perform in a less ductile manner during an earthquake. In some cases, site conditions limit the amount of batter or preclude batter piles entirely. Regardless of the amount of batter specified, there is almost always some remaining lateral loading that the piles are required to resist in flexure (fixity). The transferred hammer energy is reduced when piles are battered. The effect of soil consolidation around battered piles should be assessed, and may require preloading or other method to address settlement prior to pile driving.

Battered piles are frequently used in combination with vertical piles. The lateral resistance of battered piling is a function of the vertical load applied to the pile group. Since the sum of the forces at the pile head must equal zero, increasing the number of battered piles does not necessarily increase the lateral load capacity of the pile group. Both the lateral passive resistance of the soil above the footing and the sliding resistance developed at the base of the footing are generally neglected in design.

6.3.6 Pile Splices

Full-length piles shall be used whenever practical. Pile splices within the estimated pile length are not paid, if the estimated length is less than 60-ft. (18-m). Splices are designed to develop the full strength of the pile section. Splices shall be full penetration welds. Mechanical splice sleeves shall not be used to join sections of steel pile.

6.3.7 Pile Corrosion Loss

Piling shall be designed with sufficient corrosion resistance to assure a minimum design life of 75 years. Experience indicates that corrosion is not a problem for steel piles driven in natural soil, due primarily to the absence of oxygen in the soil. However, in fill material at or above the water table or in marine environments, moderate corrosion may occur and protection may be required. Protection can consist of concrete encasement or increased steel thickness. Concrete piles are prone to deterioration from exposure to excess concentrations of sulfate and/or chloride. At potentially corrosive sites, encasement by cast-in-place concrete can provide the required protection for piles extending above the ground surface. All exposed piling shall be painted.

6.3.8 Pile Points

The primary advantages for specifying pile points are for penetrating or displacing boulders, or driving through dense granular materials and hardpan layers to minimize damage for all steel piles. Piling can generally be driven faster and in straighter alignment when points are used.

Pile points can be used to help drive piles through soil that has gravel and/or cobbles or presents other difficult driving conditions. They can also be used to get a good 'bite' when ending piles on sloping bedrock surfaces. Points cannot be used to ensure that piles penetrate into competent bedrock. They may assist in driving through weathered zones of rock or soft rock but will generally not be effective when penetration into hard rock is desired.

The Geotechnical Report recommends the type of pile tip to be used. The bridge designer shall note this in the contract documents.

6.3.9 Pile Preboring

If embedment into rock is required or minimum pile penetration is doubtful because of possible obstructions or dense soil conditions, preboring shall be considered. When the pile is planned to have end resistance on rock, preboring may be advanced to plan pile tip elevation, and the pile shall then be proofed with an impact hammer.

The Final Geotechnical Report will state if preboring is required and provide the project-specific requirements if applicable.

6.3.10 Contract Plan Pile Information

Normally, all piles in a foundation unit are of the same size and are driven to the same nominal required bearing. The pile data to be included in the Project Notes on the Contract plans shall include:

1. Pile Type and Size. This is provided so the Contractor can bid and furnish the piles required at each foundation location. Whether pile points are required should also be specified along with the type and size. Examples of typical pile type and size callouts are as follows:

Steel: HP ___x ___ AASHTO M270, Grade 50 (*note if pile points are required*)

2. Maximum Factored Design Load for the controlling Strength and Service Limit States. This is provided in tons to document the net long term axial factored load at the top of the pile for the current and future design/rehabilitation work, as adjusted for downdrag or other conditions that may affect the factored load.

Maximum Factored Design Pile Load = xx Tons (Strength)
= xx Tons (Service I)
= xx Tons (Extreme Event II)

3. Nominal Geotechnical Resistance Factor. The geotechnical resistance factor is used to determine the pile driving criteria at the time of installation. Sometimes a minimum driving resistance in kips may also be stated because the factored design load and resistance factor may not result in an adequate driving resistance to get the piles seated in an acceptable bearing stratum.
4. Estimated Pile Length and Number of Splices. This is provided to give Contractors a bid quantity, and to define the pile length furnished by the Contractor. It also is used as a

reference by the inspectors to identify when pile problems, such as lack of set up or improper hammer performance, are causing piles to stop short or run long. In some cases, a minimum tip elevation will be specified in addition to the estimated pile length. Normally, the minimum tip elevation will only be necessary when the piles have the potential to stop shorter than estimated resulting in inadequate lateral load strength or penetration below any geotechnical losses such as scour.

5. Number and Location of Production Piles. This is the total number and location of production piles required at the substructure or foundation covered by the pile data. When test piles are specified, the number of production piles shall be decreased by the number of test piles since they will be driven in production locations, unless it is specified that the test pile will not be incorporated into the foundation as a support pile.
6. Number of Test Piles and Location. This number shall always be stated. Test piles are employed at a project site for two purposes: (1) For testing during driving (dynamic testing); (2) For load testing (static load test), to verify actual pile geotechnical resistance versus the design loads.

Test piles with dynamic monitoring are typically required to verify the design nominal resistance. During the installation of production piles, dynamic pile monitoring ensures that driving occurs in accordance with the established criteria. It provides information on soil resistance at the time of monitoring and on the adequacy of the driving system. Dynamic pile monitoring also measures driving stresses, which helps prevent pile damage. Static pile load tests may also be considered. The need for test piles is evaluated by the Geotechnical Engineer and will be included in the Final Geotechnical Report
7. Estimated Top of Rock Elevation, Rock Socket Depth, and Rock Socket Diameter. This information shall be shown in the pile data on the plans when piles are set in rock.

6.3.11 Pile Design Considerations

The Geotechnical Engineer is responsible for recommending when driven piles can be considered. The Geotechnical Engineer is responsible for the following:

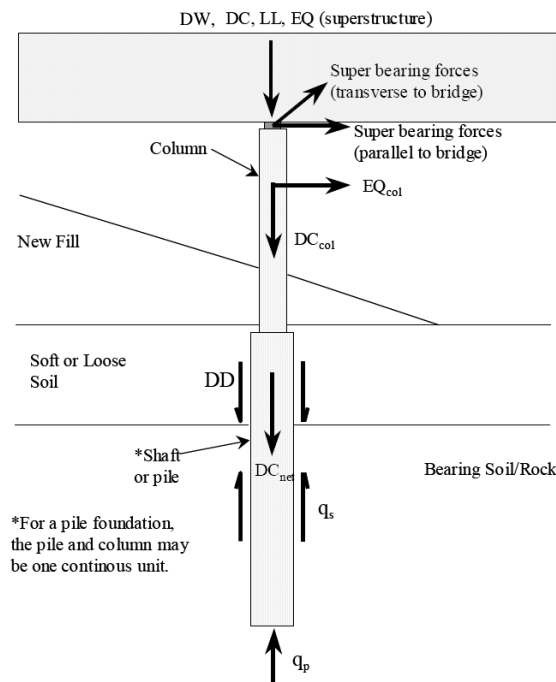
- Design considerations:
 - Axial and lateral soil resistance
 - Downdrag load
 - Lateral squeeze
 - Uplift resistance
 - Pile setup and relaxation
 - Drivability analysis
 - Scour
 - Service, strength or extreme event limit states capacity of the pile (axial, bearing, lateral load resistance)
- Type of driven pile to be used.
- Estimated pile tip elevation and any special requirements necessary to drive the piles.
- Resistance factors.
- Estimated pile length, number of splices, pile points.
- Construction considerations:
 - Pile Hammers
 - Driving formulas

- Field testing (dynamic testing and static load tests)

The bridge designer is responsible for ensuring that the axial capacity and the lateral capacity of the pile or pile group are not exceeded for any limit states group loadings in accordance with *AASHTO LRFD 10.7*.

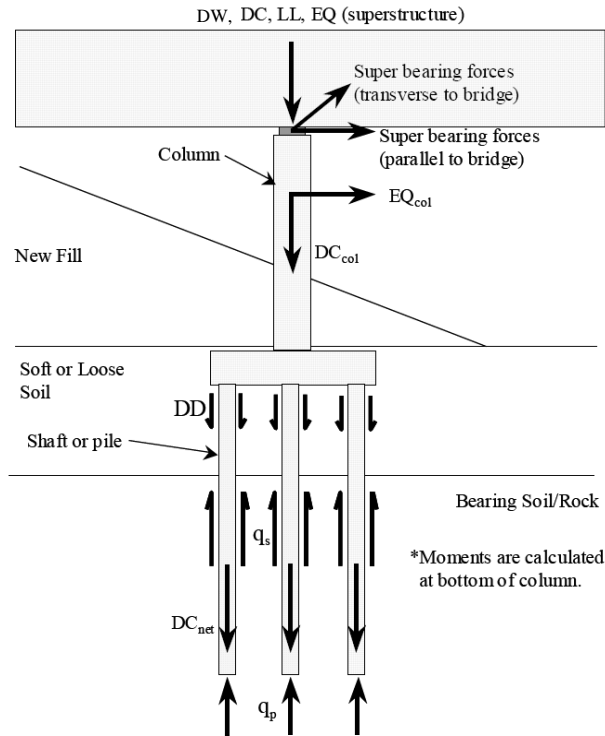
6.3.12 Loads and Load Factors for Pile Design

Figures [6.3.12-1](#) and [6.3.12-2](#) provide definitions and typical locations of the forces and moments that act on deep foundations such as driven piles. Table [6.3.12-1](#) identifies when to use maximum or minimum load factors for the various modes of failure for the pile (bearing, uplift, and lateral loading) for each force, for the strength limit state.



Integral Shaft Column or Pile Bent Forces

Fig 6.3.12-1



Pile or Shaft Supported Footing Forces

Fig 6.3.12-2

Load	Load Factor		
	Bearing Stress	Uplift	*Lateral Loading
DC, DC _{col}	Use max. load factor	Use min. load factor	Use max load factor
LL	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)	Use transient load factor (e.g., LL)
DC _{net}	Use max. load factor	Use min. load factor	N/A
DD	Use max. load factor	Treat as resistance, and use resistance factor for uplift	N/A

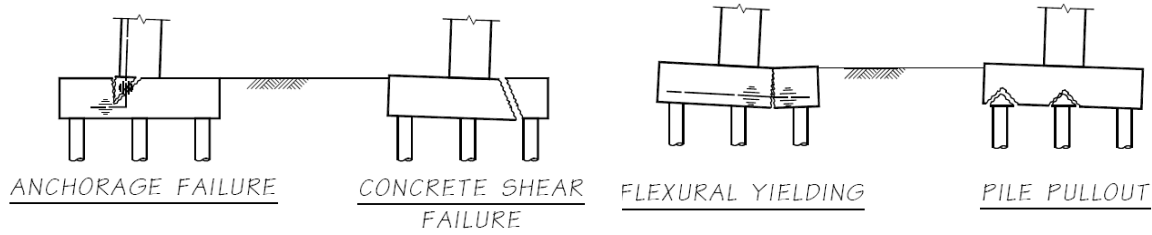
*Use unfactored loads to get force distribution in structure, then factor the resulting forces for final structural design.

Max./Min. Deep Foundation Load Factors

Table 6.3.12-1

6.3.13 Pile Supported Footing Design

Figure 6.3.13-1 identifies the modes of failure that shall be investigated for general pile cap/ footing design.



Pile Supported Footing Modes of Failure

Fig 6.3.13-1

A. Pile Embedment, Clearance, and Rebar Mat Location

All piles shall have an embedment in the concrete sufficient to resist moment, shear, and axial loads. Steel to concrete working stress shear bond strength can be estimated at 40-psi (276 kPa) for dry conditions and 20-psi (138 kPa) for tremie conditions of contact surface in checking pile embedment. The addition of shear studs to the embedded pile section can increase shear resistance.

Typically, extend piles 1.5-ft. (0.450-m) into stub abutments, 2-ft. (0.6-m) into integral abutments, and 1-ft. (0.3-m) into pier or other footings including through foundation seals. Cast-in-place concrete piles with reinforcing extending into footings are embedded a minimum of 6-in. (150-mm). Piles should be a minimum of 9 inches from any edge of the footing (*AASHTO LRFD 10.7.1.2*) with 12 inches being typical to accommodate the layout tolerance.

Reinforcing mat shall be placed at the top and bottom of the footing with a 3-in. (75-mm) clear distance and placed to clear piles.

B. Concrete Design

In determining the proportion of pile load to be used for calculation of shear stress on the footing, any pile with its center 6-in. (150-mm) or more outside the critical section shall be taken as fully acting on that section. Any pile with its center 6-in. (150-mm) or more inside the critical section shall be taken as not acting for that section. For locations in between, the pile load acting shall be proportioned between these two extremes. The critical section shall be taken as the effective shear depth (d_v) as defined in *AASHTO LRFD 5.8.2.9*. The distance from the column/wall face to the allowable construction centerline of pile (design location plus or minus the tolerance) shall be used to determine the design moment of the footing.

C. Footing Thickness

The minimum footing thickness shall be 2.0-ft. (0.6-m). Footing thickness may be governed by the development length of the column dowels, or by concrete shear requirements.

6.3.14 Concrete Piles

The three principal types of concrete piles are cast-in-place (CIP), precast reinforced, and prestressed reinforced; only the CIP piles should be considered for use. CIP concrete pile types include piles cast in driven steel casings that remain in-place and piles cast in unlined drilled holes or shafts. Driven-type concrete piles are discussed later in this section. Concrete piles cast in drilled holes are discussed later in this chapter and include drilled shafts, and micropiles.

Depending on the type of concrete pile selected and the foundation conditions, the load carrying capacity of the pile can be developed by shaft resistance, end resistance or a combination of both. Generally, driven concrete piles are employed as a displacement type pile.

When embedded in the earth, plain or reinforced concrete piles are generally not vulnerable to deterioration. The water table does not affect pile durability, provided the levels of acidity, alkalinity, chemical salt, acidic groundwater, and organic acids are not excessive. Laboratory testing of soil and ground water samples for sulfates and pH is usually sufficient to assess pile deterioration potential. A full chemical analysis of soil and groundwater samples is recommended when chemical wastes are suspected (*AASHTO LRFD 10.7.5*). Concrete piles that extend above the water surface are subject to abrasion damage from floating objects, ice debris and suspended solids. Deterioration can also result from frost action, particularly in the splash zone and from concrete spalling due to internal corrosion of the reinforcement steel. Generally, concrete spalls are a concern for reinforced concrete piles more than prestressed piles because of micro-cracks due to shrinkage, handling, placement and loading. Prestressing reduces crack width. Concrete durability increases with a corresponding reduction in aggregate porosity and water/cement ratio.

A. Driven Cast-in-Place Concrete Piles

Driven cast-in-place (CIP) concrete piles are formed by pouring concrete into a thin-walled closed-end steel casing that has been previously driven into the ground. A flat, oversize plate is typically welded to the bottom of the steel casing. The minimum thickness of the steel casing shall be that required for pile reinforcement and to resist driving stress. The Contractor may elect to furnish steel casing with greater thickness to better withstand driving stresses. A thin-walled casing must be carefully evaluated so that it does not collapse from soil pressure or deform from adjacent pile driving. Deformities or distortions in the pile casing could constrict the flow of concrete into the pile and produce voids or necking that reduce pile capacity. It is standard construction practice to inspect the open casing prior to concrete placement. Care must be exercised to avoid intermittent voids over the pile length during concrete placement.

Driven CIP concrete piles are considered displacement-type piles because of their closed end. Driven CIP piles are frequently employed in slow flowing streams and areas requiring pile lengths of 50-ft. (15-m) to 120-ft. (37-m). Driven CIP piles are generally selected over timber piles because of the availability of different diameters and wall thicknesses, the ability to adjust driven lengths, and the ability to achieve greater resistances.

Any structural strength contributed by the steel casing is neglected in driven CIP concrete pile design. Therefore, steel corrosive sites do not affect driven CIP concrete pile designs. However, CIP piles shall not be used for exposed pile bents in corrosive environments.

Driven cast-in-place concrete piles are generally the most favorable displacement pile type since inspection of the steel casing is possible prior to concrete placement, and more reliable control of concrete placement is attainable.

Driven CIP concrete piles are designed as reinforced concrete beam-columns, as described in *AASHTO LRFD 5.7.4.5 and 6.9.5.2*, when the cross-sectional area of the steel casing is less than 4 percent of the total cross-sectional area of the pile. When the effective cross-sectional area of

the steel casing is at least 4 percent of the total area of the pile, the pile is classified as a composite section and is designed as a steel pipe pile filled with concrete (*AASHTO LRFD 6.9.2, 6.9.5 and 6.15.3*).

Additional guidelines include the following:

- Concrete piles shall use a 28-day concrete compressive strength f'_c of 5,000 psi (3.5-kg/mm²).
- The steel casing is ignored when computing the axial structural resistance of driven CIP concrete piles that are symmetrical about both principal axes. This nominal (ultimate) axial structural resistance is computed using *AASHTO LRFD equation 5.7.4.4-3*, neglecting the contribution of the steel casing to resist compression.
- For piles subject to large lateral loads, the structural pile capacity must also be checked for shear and combined stress against flexure and compression. Piles subject to uplift must be checked for tension resistance.
- Since the diameter of the concrete portion of the pile is dependent on the steel casing thickness, the as-built diameter will not be known during design (since the casing thickness is determined by the Contractor). As such, a casing thickness must be assumed for design. The structural engineer shall work closely with the geotechnical engineer to determine a suitable casing thickness to assume based on expected driving conditions. A pile drivability analysis may be required for this. Otherwise, the following can typically be assumed:
 - ¼-in. for piles less than 14-in. diameter
 - ⅜-in. for piles 15-in. to 18-in. diameter
 - ½-in. for larger piles
- Steel casing for 24-in. (610-mm) diameter and smaller CIP piling shall be designated by nominal diameter rather than inside diameter. Steel casing must meet ASTM A252 Grade 2 requirements, and are purchased by nominal diameter (outside diameter) and wall thickness. As stated previously, the *Standard Specifications* require the Contractor to determine the pile casing thickness required for driving.
- Transverse spiral reinforcement shall be designed to resist the maximum shear in the pile.
- Resistance factors for Strength and Extreme Event Limit States shall be per *AASHTO LRFD 5.5.4.2.1*.
- Piles are typically assumed to be continuously supported. Normally, the soil surrounding a foundation element provides sufficient bracing against a buckling failure. Piles that are driven through very weak soils shall be designed for reduced lateral support, using information from the Geotechnical Section as appropriate. The unbraced length should also be considered if scour is a factor. *AASHTO LRFD 10.7.3.13.4* may be used to estimate the column length for buckling. Piles driven through firm material normally can be considered fully supported for column action (buckling not critical) below the ground.
- The axial load along the pile varies due to side friction. It is considered conservative, however, to design the pile for the full axial load plus the maximum moment. The entire pile is then typically reinforced for this axial load and moment.
- In all cases of uplift, the connection between the pile and the footing must be carefully designed and detailed. The bond between the pile and the seal may be considered as contributing to the uplift resistance. The pile must be adequate to carry tension throughout its length. See Section [6.3.13 A](#). for bond values.

6.3.15 Steel Piles

Steel piles generally consist of either H-pile or pipe pile types. Both open-end and closed-end pipe piles can be used depending upon conditions and capacities required. Pipe piles may be left open or filled with concrete, and can also have a structural shape inserted into the concrete. Steel pile can be socketed into bedrock with preboring.

Steel piles are typically top driven at the pile butt. The initial installation of a steel pile may be performed using a vibratory hammer, but their final driving to seat the pile shall be done with an impact hammer.

Steel piles can be used in friction, end-bearing, a combination of both, or rock-socketed piles. One advantage of a steel pile is the ease of splicing or cutting to accommodate differing final constructed lengths.

The nominal (ultimate) structural compressive resistance of steel piles is determined in accordance with *AASHTO LRFD 10.7.3.13.1* for either non-composite or composite sections. Composite sections include concrete-filled pipe piles and steel piles that are encased in concrete. The nominal structural compressive resistance for non-composite and composite steel piles is further specified in *AASHTO LRFD 6.9.4 and 6.9.5*, respectively. The effective length of horizontally unsupported steel piles is determined in accordance with *AASHTO LRFD 10.7.3.13.4*. Resistance factors for the structural compression limit state are specified in *AASHTO LRFD 6.5.4.2*.

A. H- Piles

Steel piles are generally used as end-bearing piles and employ what is known as the HP-section (often called H-piles for brevity). Steel H-piles are rolled sections with wide flanges such that the depth of the section is approximately equal to the width of the flanges. The cross-sectional area and volume displacement are relatively small and, as a result, H-piles can be driven through compact granular materials and slightly into soft rock. Also, steel piles have little or no effect in causing ground swelling or raising of adjacent piles. Because of the small volume of H-piles, they are considered “non-displacement” piling.

H-piles are available in many sizes and lengths. Typical pile lengths range from 40 to 120-ft. (12-m to 37-m). Common H-pile sizes vary between 10 and 14 in. (250-mm to 355-mm).

Always assume ($\phi_c = 0.5$) for severe driving conditions (*AASHTO LRFD 6.5.4.2*). The driving resistance may be more than the maximum axial resistance.

Since granular soil is largely incompressible, the principal action at the tip of the pile is lateral displacement of soil particles. Although it is an accepted fact that steel piles develop extremely high loads per pile when driven to end-bearing on rock, some misconceptions still remain that H-piles cannot function as friction piles. Load tests indicate that steel H-piles can function satisfactorily as friction piles in sand, sand-clay, silt-and-sand or hard clay. However, they will typically drive to greater depths than displacement piles.

H-Piles used for bridge foundations shall be composed of rolled-steel sections of ASTM A572, Grade 50 steel (345-MPa). For conventional abutments and mass piers, H-piles shall be oriented with the flange perpendicular to the substructure axis in the direction of the maximum applied lateral load if necessary. For integral bridges, the piles are oriented in either their weak or strong axis perpendicular to the longitudinal axis of the bridge, depending upon the requirements of the design (See Section [6.4.6](#), Design/Analysis for Integral Abutments).

Preferred sections are HP 12x53 and HP 10x42. The yield strength shall be based on 50-ksi (345-MPa).

B. Pipe Piles

Pipe piles consist of seamless, straight or spirally butt-welded metal shells. Steel pipe piles may be driven in groups, to support ground-level pile caps, or in-line to form pile bents. They are available in a wide range of diameters.

Pipe piles may be driven either open or closed end. If the end bearing capacity from the full pile toe area is required, the pile toe shall be closed with a flat plate or a conical tip.

If obstructions are expected, the pile shall be open-ended, so that it can be cleaned out and driven further. Open-ended piles driven in sands or clays will form a soil plug at some stage during driving. At this stage, the pile acts like a closed ended pile which can increase the pile toe resistance.

Concrete filled pipe piles have a high load-carrying capacity and provide high bending resistance where an unsupported length is subject to lateral loads.

Steel pipe pile material shall conform to ASTM A252 Grade 2 or Grade 3. Open-ended piles shall be reinforced with steel cutting shoes to provide protection against damage. End closures for close end pipe piles shall be cast steel, conforming to the requirements of ASTM A27 (grade 65-35) or ASTM A148 (grade 90-60).

For high vertical or lateral loads, open-ended pipe piles may be socketed in bedrock. They can also have a structural shape such as an H-section inserted into the concrete and socketed into bedrock. Anchoring pipe piles with rock dowels or anchors is not recommended.

Pipe piles can be spliced using full penetration groove welds that provide full strength in bending.

6.3.16 Timber Piles

Timber piles shall not be used for bridge structures.

6.3.17 Drilled Shafts

Drilled shafts are generally large diameter, cast-in-place, open ended, concrete circular shafts that can be designed to carry heavy loads. Drilled shafts can be the most economical foundation alternative at sites where foundation loads are carried to bearing on dense strata or bedrock. They are also cost effective in water crossings with very shallow bedrock, where cofferdams are difficult or expensive to construct, and where high overturning moments must be resisted. Drilled shafts may also be used to enhance the stability of piers adjacent to a navigation channel. See FHWA publication *Geotechnical Engineering Circular No. 10- Drilled Shafts Construction Procedures and LRFD Design Methods* for additional information on design and construction of drilled shafts (http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm).

Drilled shafts are installed by removing soil and rock using drilling methods or other excavation techniques and constructing the foundation element in the excavated hole. The excavated hole may be supported using temporary or permanent casing, drilling slurry or other methods. The hole is then filled with a reinforcement cage and cast-in-place concrete. Drilled shafts are non-displacement elements since the soil volume required for the element is physically removed prior to installation. Thus the effective normal stress adjacent to the pile remains unchanged or is reduced (due to expansion of the soil into the hole before insertion/construction of the load bearing element), and the soil properties and pore water pressure adjacent to the foundation elements are not significantly impacted.

Because drilled shafts do not require a hammer for installation and do not displace the soil, they typically have much less impact on adjacent structures. Depending on the excavation technique used, they can penetrate significant obstructions. Because the method of construction often allows a decrease in the effective stress immediately adjacent to and beneath the tip of the foundation element, the resistance developed will often be less than an equivalently sized driven pile.

A. Drilled Shaft Usage

Drilled shafts can be an advantageous foundation alternative when:

- Spread footings cannot be founded on suitable soil, or bedrock, within a reasonable depth or when driven piles are not viable.
- Traditional piles would result in insufficient embedment depth.
- Scour depth is large.
- Foundations are required in stream channels. Drilled shafts will avoid expensive construction of cofferdams. Advantages are the reduction of the quantities and cost of excavating, dewatering, and sheeting, and in limiting environmental impact.
- The foundation is required to resist high lateral loads or uplift loads.
- There is little tolerance for deformation.
- The cost of seals and caps for pile supported structures is high or constructability is difficult.

B. Drilled Shaft Design

Information used for the design of drilled shafts is determined by the geotechnical engineer. This information includes depth (length) of the earth and rock portions of the shaft, and maximum load resistance for a given diameter. The geotechnical load resistance of drilled shafts is provided by sidewall friction and end bearing in soil or rock. End bearing resistance may not be included in the design, if there are concerns with bottom cleanliness.

Unless noted otherwise in the Geotechnical Report, the following criteria shall be used in designing drilled shaft foundations:

- Drilled shafts shall be designed for limit states and resistance factors as specified in *AASHTO LRFD 10.5*.
- Drilled shafts shall be designed to provide adequate axial and structural resistance consistent with the anticipated construction method. The drilled shaft design must also satisfy the tolerable vertical and lateral displacement criteria. All design shall be in accordance with *AASHTO LRFD 10.8*.
- Strength limit states for drilled shafts are evaluated in the same way as for driven piles. Drivability does not need to be evaluated. The structural resistance of drilled shafts is evaluated in accordance with *AASHTO LRFD 5.7 and 5.8*. This includes evaluation of axial resistance, combined axial and flexure, shear and buckling. It is noted that the critical load case for combined axial and flexure may be a load case that results in the minimum axial load or in tension.
- For the design of drilled shafts for piers and abutments, dynamic load allowance shall be excluded from the vertical loads (*AASHTO LRFD 3.6.2.1*).
- In cases where drilled shafts are placed below the water table, loads due to buoyancy shall be considered.
- For typical drilled shaft foundations for bridges the load modifier shall be taken as 1.0.

- Drilled shaft concrete only needs to extend a minimum depth (6-in. [100-mm]) into the footing or abutment. Drilled shaft reinforcement shall extend into the footing or abutment sufficiently to develop the required structural resistance.
- If the lateral load at the top of a drilled shaft is greater than the lateral geotechnical resistance of the shaft, the shaft diameter shall be increased to provide the needed resistance, or additional shafts shall be placed and tied together with a footing. Battered shafts are not permissible (*AASHTO LRFD 10.8.1.4*).
- The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, etc.), but the structural capacity of permanent steel casing shall not be considered for structural design of drilled shafts. Permanent casing shall extend a sufficient distance into rock to form a seal.
- If a drilled shaft penetrates soil layers subject to subsequent consolidation, the designer shall consider the effect of downdrag.
- Concrete strength for the drilled shafts shall be specified on the plans or special provision.
- Drilled shafts are normally designed as columns with longitudinal and spiral reinforcement.

C. Drilled Shaft Detailing

Detailing of the reinforcing steel in a drilled shaft must consider the constructability of the shaft. The reinforcing cages must be stiff enough to resist bending during handling and concrete placement. In addition, the spaces between reinforcement bars must be kept large enough to permit easy flow of the concrete from the center of the shaft to the outside of the shaft.

Unless noted otherwise in the Geotechnical Report, the following criteria shall be used in detailing drilled shaft foundations:

- Drilled shafts for bridge support shall be at least 36-in. (915-mm) in diameter. For bridge piers with columns directly above the shafts, the columns are detailed 2-ft. (600-mm) smaller in diameter than the shafts in soil. This accommodates some field adjustment when shafts are constructed out of plan location.
- Preferably, the diameter of a drilled shaft should be one of the following: 36, 48, or 60-in. (915, 1219, 1524-mm), larger sizes are possible.
- The designer shall consider the permissible construction tolerance for the plan position of a drilled shaft which is typically 3-in. (75-mm).
- The minimum distance from the nearest edge of the footing (or pier cap) to the face of any drilled shaft shall be 1-ft. (0.300-m) per *AASHTO LRFD 10.8.1.2*.
- Do not use keys in the design of construction joints for drilled shafts.
- Drilled shafts shall be reinforced full height.
- Black reinforcing bars shall be used unless the top portion of a shaft will be permanently exposed, or if the bars will be extended into an exposed portion of the structure. In this case, corrosion resistant bars (epoxy coated or stainless steel) shall be used only at the top of the shaft unless it is more practical to use them throughout.
- Reinforcing bars shall be detailed to provide a clear distance between bars as noted in *AASHTO LRFD 5.13.4.5.2*.

- Longitudinal bars shall be #5 (#16) or larger. Bundled bars should be avoided. Longitudinal reinforcement shall be a minimum of six bars. The designer shall avoid use of double cages. Longitudinal bar splices shall be staggered. Splice bars shall be alternated and staggered so that no more than 50% of reinforcement is spliced at one location.
- The diameter of the reinforcement cage shall be the same throughout all portions of the shaft (above ground, through soil, and in rock). Shaft diameter in rock shall generally be specified as 6-in. (150-mm) smaller than the portion in soil. Concrete cover both in soil and rock shall be a minimum of 6-in. (150-mm). Non-corrosive spacers are used to ensure that the annular space around the cage is maintained.
- If the structure design requires a reinforced connection between the top of the shaft and the structure above, hooked bars shall not be used because they will interfere with concrete placement tools.
- The contract plans shall also provide the following items when applicable: estimated top of rock elevations, minimum bottom of permanent casing, and Estimated Water Surface Elevation (EWSE).
- Access tubes for Crosshole Sonic Logging (CSL) testing shall be provided in all shafts. One tube shall be furnished and installed for each foot of shaft diameter, rounded to the nearest whole number, and shown in the plans. The number of access tubes for shaft diameters specified as “X feet 6 inches” shall be rounded up to the next higher whole number. The access tubes shall be placed around the shaft, inside the spiral or hoop reinforcement and three inches clear of the vertical reinforcement, at a uniform spacing measured along the circle passing through the centers of the access tubes. If the vertical reinforcement is not bundled and each bar is not more than one inch in diameter, the access tubes shall be placed two inches clear of the vertical reinforcement. If these minimums cannot be met due to close spacing of the vertical reinforcement, then access tubes shall be bundled with the vertical reinforcement.
- The drilled shaft design will need to consider the type of drilled shaft excavation support that will be used during construction. Excavation support methods include temporary and permanent casing, and slurry support methods. The type of excavation support has a direct effect on the side friction value that is used to determine the geotechnical side friction resistance. Permanent casing should be specified when shafts are constructed through water. The Geotechnical Section will provide guidance regarding the construction method and the geotechnical axial and lateral resistance.

6.3.18 Micropiles

Micropiles are used for structural support of new structures, underpinning existing structures, scour protection, and seismic retrofit at existing structures. In areas of restricted access, close proximity to settlement-sensitive existing structures, or difficult geology, micropiles may be considered when determining the foundation type. Although typically more expensive than driven piles, constructability considerations may warrant selection of micropiles as the preferred foundation type. A micropile is constructed by drilling a borehole with drill casing, placing reinforcement, and grouting the hole. Drill casing permits installation in poor ground conditions. Micropiles are installed with similar equipment that is used for ground anchor and grouting projects. Micropiles can be either vertical or battered.

With a micropile's smaller cross-sectional area, the pile design is more frequently governed by structural and stiffness considerations. Due to the small pile diameter, end resistance is usually disregarded for design. Micropiles are typically designed to derive all their support from side friction in a rock socket or competent soil. The permanent outer casing can be used in the structural design if not considered sacrificial. High strength steel casing, 85 ksi rated pipe, is generally used for the outer casing when needed for structural design. An inner steel core pipe can be used in the micropile design to provide reinforcement in addition to the central bar where needed for high stress zones.

Grout/ground bond capacity varies directly with the method of placement and pressure used to place the grout. Common methods include grout placement under gravity head, grout placement under low pressure as temporary drill steel is removed, and grout placement under high pressure using a packer and regrout tube. Some regrout tubes are equipped to allow regrouting multiple times to increase pile capacity.

Micropiles shall be designed in accordance with *AASHTO LRFD 10.5 and 10.9*. Additional background information on micropile design may be found in the *FHWA Micropile Design and Construction Guidelines Implementation Manual, Publication No. FHWA-SA- 97-070 (Armour, et al., 2000)* (http://www.fhwa.dot.gov/engineering/geotech/library_listing.cfm).

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6.4 Abutments

6.4.1 General

Abutments are used at the ends of bridges to retain the embankment and to carry the vertical and horizontal loads from the superstructure to the foundation. The design requirements for abutments are similar to those for retaining walls and for piers; each must be stable against overturning and sliding. Abutment foundations must also be designed to prevent differential settlement and excessive lateral movements. Abutments can be cast-in-place; reinforced concrete or precast founded on a spread footing; or founded on a deep foundation such as drilled shafts or driven piles.

6.4.2 Abutment Type and Considerations

Abutment types shall be selected considering aesthetics, foundation recommendations, structure location, and the loads it must transmit to the foundation. For structures over waterways, the abutment type and location shall also be specified with consideration to hydraulic conditions at the site.

Integral and semi-integral abutments are the preferred abutment types when the limitations specified in this manual are met. All projects must therefore consider the use of integral or semi-integral bridge abutments as part of the bridge type or design alternative studies.

Semi-integral abutments are the preferred type of abutment when the wingwall length, abutment exposure or superstructure depth requirements for integral abutments cannot be met.

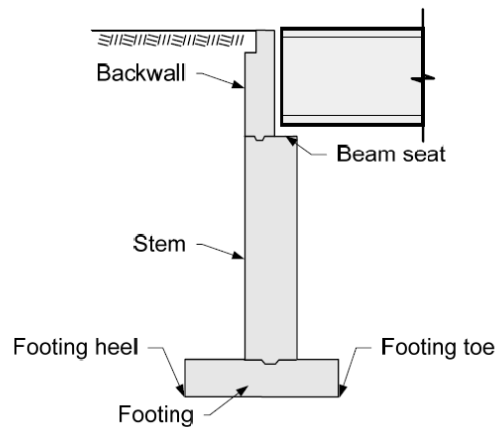
A stub abutment supported by a mechanically stabilized earth (MSE) retaining wall can be used where the abutment is to be located at or near the top of the approach fill and high abutments would otherwise be required and it would be economical to use an MSE wall.

The decision to use a cantilever abutment with a deck expansion joint shall be limited to locations where all possible solutions for an integral abutment structure or other alternatives have been fully explored and found unfeasible.

Several different abutment types are used in New Hampshire, including cantilever, integral, semi-integral, stub and special designs. Each of these abutment types is described in the following sections.

A. Cantilever

Cantilevered abutments consist of a central stem supporting the bridge seat. A backwall on top of the stem and wingwalls on either side of the stem retain the fill. The stem and wingwalls rest upon a continuous footing that can be either soil or pile supported. The superstructure length used with cantilevered abutments is not limited. The abutment shall be designed to support the applied superstructure loads. The thermal expansion of the superstructure shall be accounted for by the use of an expansion joint. The abutment is assumed to be a cantilever member fixed at the top of the footing and subjected to axial, shear, and bending loads. See Figure [6.2.3-1](#) and [Appendix 6.4-B1](#) for cantilever details and Section [6.4.4](#) for design/analysis of cantilever abutments.



Cantilever Abutment

Fig 6.4.2-1

B. Integral

In an integral abutment structure, a rigid connection is made between the primary support members of the superstructure and a pile supported substructure by an abutment diaphragm poured monolithic with the deck and encasing the girder ends. The diaphragm is connected to the stem, making the superstructure integral with the abutment. Unlike cantilevered abutments, integral abutments do not require a joint in the bridge deck or conventional bearings. An integral abutment does not have a footing, as the abutment is supported on a single row of piles extending out of the abutment stem. The piles are allowed to rotate and deflect horizontally as the abutment stem moves due to thermal expansion of the superstructure.

Integral abutment bridges offer many advantages over conventional cantilevered abutments. Joints at bridge abutments are prone to leak, allowing water containing road salts to drain onto the underlying superstructure beams, bearings, abutment backwalls and bridge seats. By doing away with these joints, future maintenance associated with joint leakage is eliminated, thereby greatly reducing the life cycle cost of the structure. Integral abutments also cost less to construct. Having no footing, no bearings, fewer piles, and relatively simple concrete forming requirements makes integral abutments a cost effective alternative to conventional abutments. Another advantage of integral abutments is that they can be constructed in a much shorter time as compared to conventional abutments.

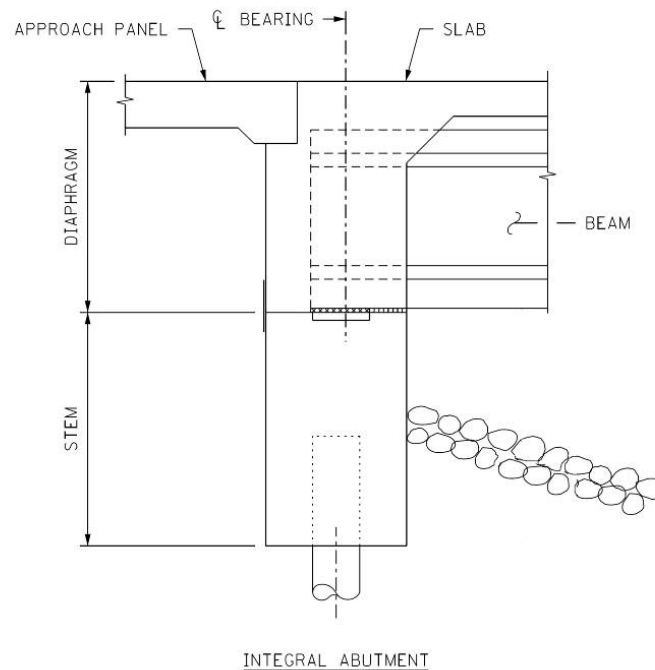
Integral abutments shall always be considered as the first choice of abutment because of their lower construction cost and superior long-term performance.

See Section [6.4.6](#) for design/analysis of integral abutments. Details of integral abutments for each type of superstructure can be found in [Appendix 6.4-B2](#). See Figure [6.4.2-2](#) for an integral abutment.

In order to use the simplified design method for integral abutments, each of the following criteria must be met, unless directed otherwise by the Design Chief:

- Skew angle less than or equal to 20 degrees.
- Straight bridge or a curved bridge with straight beams with all beams parallel.
- Grade 50 (345) steel for H-Piles.
- Parallel abutments and piers.

- Maximum abutment height of 13-ft. (4-m) to finished grade to reduce the passive earth pressure acting against each abutment, unless approved otherwise by the Design Chief.
- Strive to use equal abutment heights at each end of the bridge.
- Maximum total bridge length, as measured between centerlines of bearing at each abutment of:
 - 300-ft. (91-m) for steel bridges, and
 - 600-ft. (183-m) for concrete bridges
- Individual span length between supports less than 150-ft. (45.7-m).
- Sleeper slab shall be used with all approach slabs.
- Longitudinal slope of the bridge deck equal to or less than 5%.
- One pile per girder as a guideline.
- In-line wingwalls (parallel and cantilevered from the abutment) are the preferred arrangement; flared walls cantilevered from the abutment may be considered if approved by the Design Chief. See Figure [6.5.2-2](#) for wingwall configurations.



Integral Abutment

Fig 6.4.2-2

C. Semi-integral Abutment

Semi-integral abutments are similar to integral abutments in that the superstructure and approach slab are connected and move together. The expansion and contraction movement of the bridge superstructure is accommodated at the end of the approach slab. Unlike integral abutments, the superstructure is supported on bearings that allow movement independent from the abutment stem. The abutment stem is stationary and is supported by a spread footing or a pile cap on multiple rows of piles. In these bridges, the abutment foundations behave conventionally, while the backwall (end diaphragm) moves along a horizontal joint below ground. In general, semi-integral bridges resist excessive translation in the longitudinal directions via full-depth end diaphragms and passive earth pressure. Semi-integral abutments shall be considered for use where site conditions prevent the construction of full integral abutments.

The design allows the superstructure and the approach slab to move together independent of the abutment. Therefore wingwalls shall not be attached to the superstructure and the vertical joints between them shall be parallel with the centerline of the roadway.

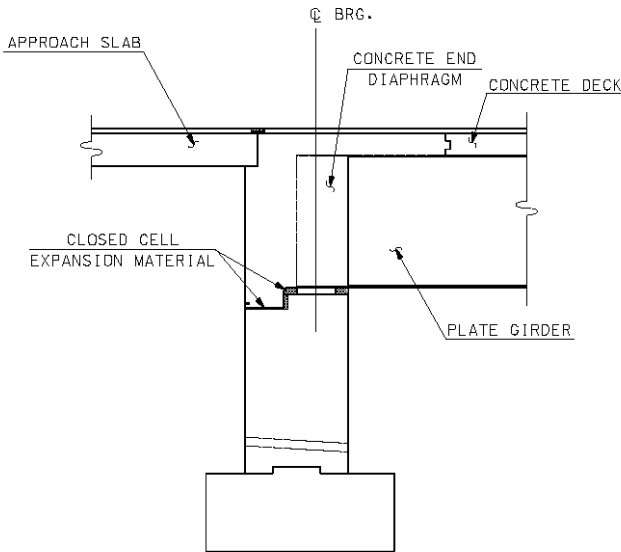
Advantages:

- Some restrictions from integral abutments can be neglected
- No wingwall length limit
- No front face exposure height limit
- No superstructure depth limit
- Allows for greater rotation than for integral bridges
- Can be placed on piles or spread footing

Disadvantages:

- More complicated design in comparison to integral abutments
- Must still meet all bridge length, skew, and horizontal alignment criteria for integral abutments
- Additional piles and concrete required

See Section 6.4.8 for design/analysis of semi-integral abutments. See Figure 6.4.2-3 and Appendix 6.4-B2 for details.



Semi-Integral Abutment

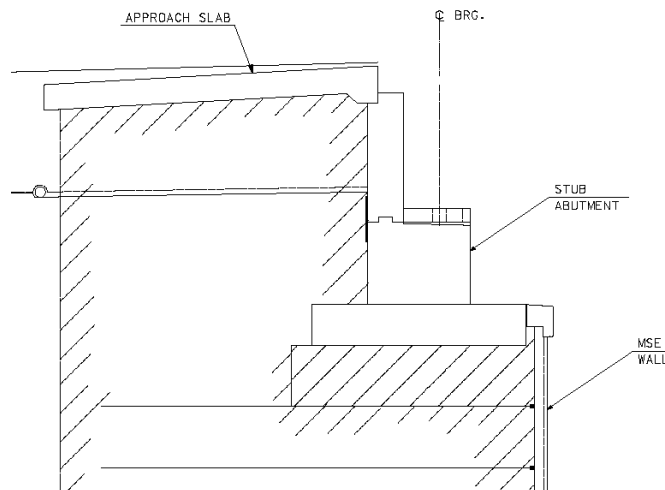
Fig 6.4.2-3

D. Stub Abutment

A stub abutment is constructed near the top an embankment fill or on a MSE wall structure. The stub abutment can be configured with a spread footing or a deep foundation. Stub abutments are generally easier and less costly to construct than a full height abutment. The stub abutment helps avoid many of the problems that cause rough approach pavements. It eliminates the difficulties of obtaining adequate compaction adjacent to the relatively high walls.

The parallel-to-abutment-centerline wingwalls or flared wingwalls, shall be used for grade separations when possible. This wing type is preferred because it increases flexibility in the abutment, simplifies compaction of fill, and improves stability.

If there is not enough room to place a berm in front of the stub abutment, then a MSE wall may be constructed to retain the embankment. See Figure 6.4.2-4, Section 6.5.6 [D](#), and [Appendix 6.4-B1](#) for details.



Stub Abutment

Fig 6.4.2-4

E. Special Designs

In addition to the standard abutment types described in the previous sections, many different styles and variations of those abutment types can also be designed. Such special abutment designs may be required due to special aesthetic requirements, unique soil conditions or unique structural reasons. Special designs of abutments require prior approval by the Design Chief.

6.4.3 Loads and Load Application for Abutment Design

When appropriate, abutment design shall include evaluation of settlement, lateral displacement, overall stability of the earth slope with the foundation unit, bearing capacity, sliding,

loss of contact with foundation soils, overturning, and structural capacity. Abutments shall be designed for extreme events such as vessel collisions, vehicle collisions, and seismic activity, along with changed conditions such as scour, as applicable.

Although the vertical and horizontal reactions from the superstructure represent concentrated loads, they are commonly assumed to be distributed over the entire length of the abutment wall or stem that support the reactions. That is, the sum of the reactions, either horizontal or vertical, is divided by the length of the wall to obtain a load per unit length to be used in both the stability analysis and the structural design. This procedure is sufficient for most design purposes.

Abutments shall be designed in accordance with the *AASHTO LRFD Specifications* and as noted in Chapter 4. Loads and load factors shall be determined in accordance with *AASHTO LRFD Section 3* and as outlined in Chapter 4 in this manual. Longitudinal forces for abutment design shall include any live load longitudinal forces developed through bearings such as braking forces, or others as specified in *AASHTO LRFD Section 3*.

A. Earth Load

Active earth pressure shall be applied as noted in *AASHTO 3.11.5* and Chapter 4, Section 4.3.3 from the finished grade (even if there is an approach slab) to the bottom of the footing or bottom of stem if integral abutment. See Figure [6.2.3-1](#) in this manual for a diagram of earth pressure loads.

Passive earth resistance shall not be applied for stability except for extreme limit states and integral and semi-integral abutment design and as noted in the Bridge Design Manual, Chapter 4, Section 4.3.3.

B. Live Load Surcharge

Apply live load surcharge as noted in *AASHTO LRFD 3.11.6* from finished grade to the bottom of the footing only when there is no approach slab.

C. Dead Load

In computing the weight of the approach slab, assume there is settlement under 2/3 of the approach slab and place one-half of the 2/3 weight of the slab applied on the abutment at the approach slab seat.

In computing the dead load of the superstructure, sum the dead load reactions for each beam and divide by the length of the abutment to obtain a load per unit length to be used in both the stability analysis and the structural design.

D. Dynamic Load Allowance

The dynamic load allowance shall be applied in accordance with *AASHTO 3.6.2* and Chapter 4, Section 4.3.5.

E. Live Load

Live load vertical reactions obtained directly from the superstructure design are based on maximum conditions assuming a “single beam analysis”. The girder vertical reactions are overly conservative in the design of substructure elements since they make no allowance for a realistic distribution of the live load across the roadway.

For abutment design, the live load reaction is calculated using a vehicle lane reaction. The vehicle lane reaction is determined by positioning the HL-93 lane loading and truck loading longitudinally on the structure to obtain the maximum reaction at the support location. The

vehicle lane reaction should be increased for the number of design traffic lanes and distributed over the entire length of the substructure stem to obtain a load per unit length. Multi-presence factors apply.

See Chapter 4, Section 4.3.5 for additional information.

F. Braking and Friction Forces

Vehicular braking force shall be applied as noted in *AASHTO LRFD 3.6.4* and Chapter 4, Section 4.3.6.

Friction force shall be applied as noted in *AASHTO LRFD 3.13* and Chapter 4, Section 4.3.8 of this manual. Substructures at expansion supports shall be designed for a force no less than the total frictional force that may develop at the expansion bearings.

Braking forces shall not be considered in the design of integral & semi-integral abutments since the load will be resisted by the passive soil acting on the abutment.

G. Bearing Translation

The longitudinal loads transmitted from the superstructures to the substructures shall be applied as noted in *AASHTO LRFD 3.12* and Chapter 4 of this manual.

H. Bearing Rotation

Rotations in the elastomeric bearing pads will cause bending moments that shall be transmitted to the substructure (*AASHTO LRFD 14.6.3.2*).

I. Transverse Horizontal Loads

The horizontal loads transmitted from the superstructure to the substructure and computed as noted in *AASHTO LRFD Section 3* and Chapter 4 of this manual shall be assumed to be resisted by the fixed supports (i.e., substructures where the superstructures are prevented from transverse translation by bearings or other means such as concrete restraints).

For loads which are applied over the length of the superstructure (such as wind on superstructure or wind on live load) the total transverse horizontal load shall be transmitted to the individual fixed support utilizing load distribution methods appropriate for the actual transverse continuity configuration.

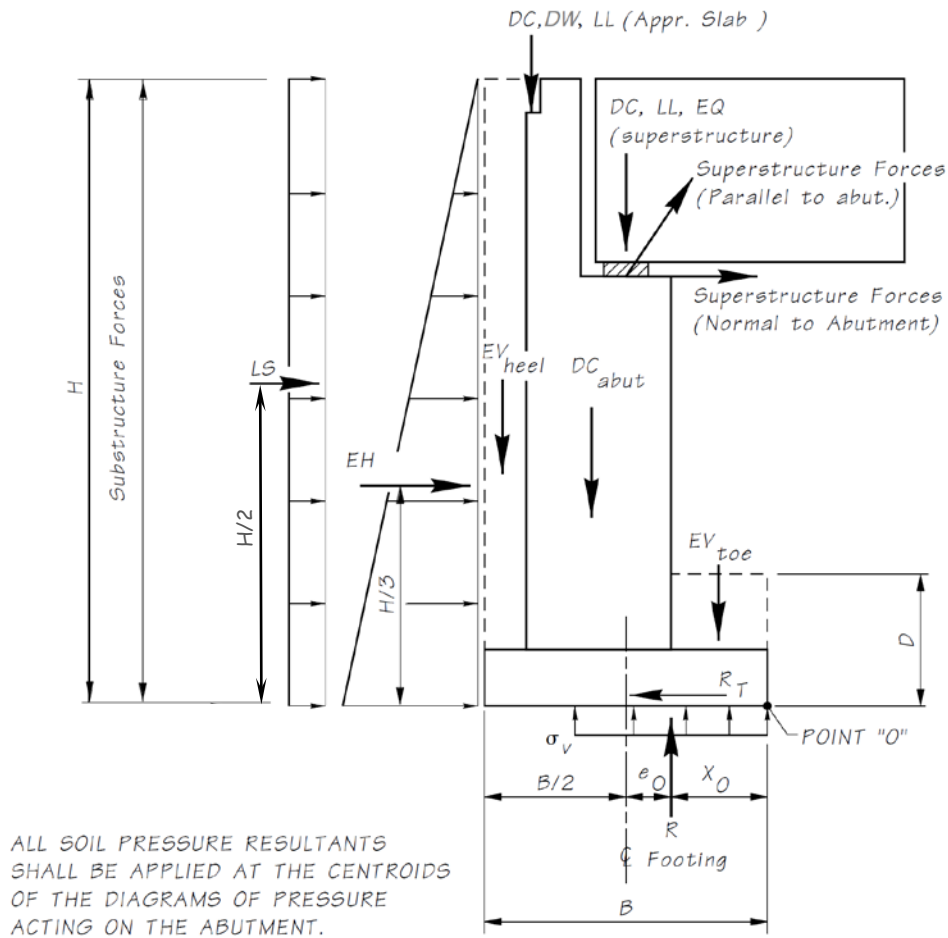
J. Other Loads

Abutments shall be designed for all applicable loading, in an addition to what is noted above, in accordance with the *AASHTO LRFD Section 3* and as noted in Chapter 4 in this manual.

6.4.4 Design/Analysis for Cantilever and Stub Abutments

For substructure design, foundation design at the Service Limit State includes settlement, lateral displacement and overall stability. See Section [6.2](#) and [6.3](#).

All components of the abutment must satisfy the appropriate strength and serviceability requirements. The three major parts of the abutment consist of the backwall, stem and footing. The design will be based on a foot-wide strip. See Figure [6.4.4-1](#) for an abutment force diagram.



**Abutment Force Diagram
(Bearing on Soil)**

Fig 6.4.4-1

A. Load Cases

As a minimum, the following load cases shall be evaluated:

- Construction Case 1:

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

Abutment has been constructed and backfilled, but the superstructure and approach panel are not in place. Each possible combination of maximum and minimum load factors shall be analyzed. Assume a live load surcharge for construction equipment. This load case shall also be analyzed with and without the abutment toe fill (slope protection). The toe fill may be removed due to any future construction or scour if located near a river.

- Construction Case 2:

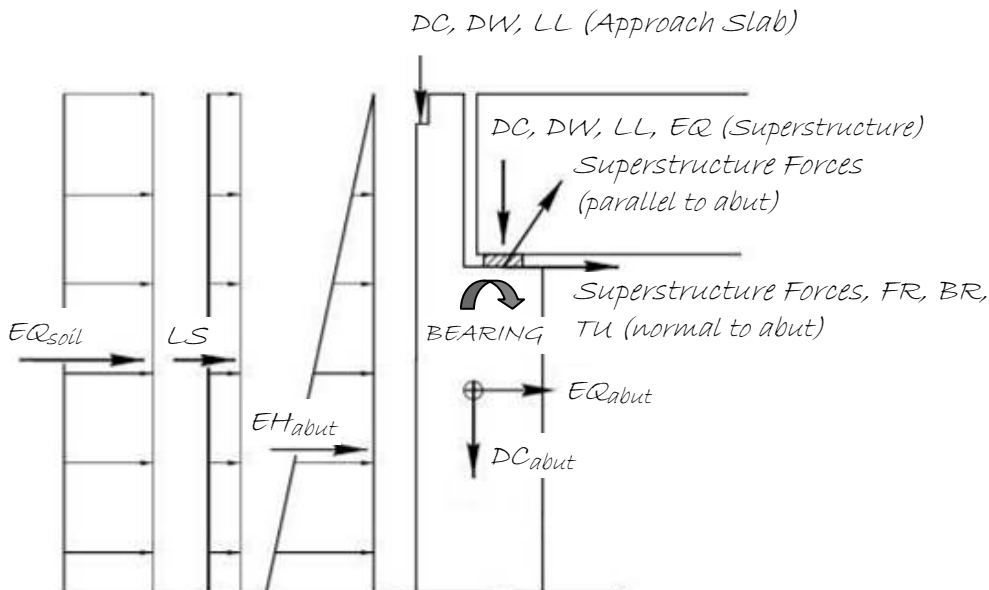
The abutment has been constructed and backfilled, the superstructure and approach panel are in place but the bridge is not open for live load traffic. Each possible combination of maximum and minimum load factors shall be analyzed. Assume a live load for construction equipment on the superstructure. This load case shall also be analyzed with and without the abutment toe fill (slope protection). The toe fill may be removed due to any future construction or scour if located near a river.

- Final Case 1:

Bridge is complete and live load traffic is on the bridge. Each possible combination of maximum and minimum load factors shall be analyzed. This load case shall also be analyzed with and without the abutment toe fill (slope protection). The toe fill may be removed due to any future construction or scour if located near a river

B. Stem Design

The combination of loads for the bottom of the abutment stem is similar to that of the backwall with the addition of the superstructure loads. The major loads on the stem are the horizontal earth pressure and the loads transmitted from the superstructure to the substructure through the bearings or pinned connection. See Figure 6.4.4-2 for an abutment stem force diagram.



Abutment Stem Force Diagram

Fig 6.4.4-2

6.4.5 Details for Cantilever and Stub Abutments

The following applies to the detailing of cantilever and stub abutments (See [Appendix 6.4-B1](#) for details):

A. Backfill and Drainage

The bridge designer shall determine where the drainage will occur around the bridge. Adequate drainage of fill behind structures is important to increase the longevity of retaining structures.

All abutments, wingwalls and retaining walls shall be backfilled with Granular Backfill (Bridge) in accordance with *NHDOT Standard Specifications for Road and Bridge Construction* to the limits as shown on the contract plans. Typically, the backfill limits extend 1-ft. (0.3-m) beyond the outline of the footing. Granular Backfill (Bridge) is a clean, free-draining, very dense, granular soil (See Chapter 4, Section 4.3.3 for the soil properties). Backfill for cantilever abutment structures should be placed at least to the level of the bridge seats prior to beam installation.

If a stub abutment is situated within an MSE reinforced zone, the backfill shall be the same material as placed for the MSE wall, and it shall be placed in lifts and compacted in the same way as the MSE wall backfill material.

Except for integral and semi-integral abutments, subsurface drainage shall be accomplished with the use of 4-in. (100-mm) diameter weepholes in all walls or abutments. The weepholes shall extend through the wall stems sloped at 12:1 and be located 1-ft. (0.3-m) above the footing or above normal high water, whichever is higher, at about 10-ft. (3-m) on center.

- When there is a sidewalk in front of any abutment or wall, an underdrain shall be used.
- When unusual drainage conditions exist, perforated drains shall be considered behind all walls and abutments.
- When there is slope paving in front of the abutment, the weepholes shall be located 1-ft. (0.3-m) above finished grade.
- Subsurface drainage for proprietary retaining walls shall conform to the special provision governing their design and construction.

B. Beam Seats

Because of the bridge deck cross-slopes or skewed abutments, it is necessary to provide beam seats of different elevations on the abutment. The tops of these beam seats are poured to precise elevations and are made level. To allow for drainage between the beam seats, provide wash sections sloped with a minimum 2-in. (50-mm) wash from the front face of the backwall and closed at the ends. This equates to a 7% slope for a 2.5-ft. (762-mm) wide abutment. If the abutment is wider than 2.5-ft. (762-mm), increase wash to achieve a 7% minimum slope.

- The width of the beam seat shall be such that it provides adequate room for the bearings and meets the seismic criteria of Chapter 5 of this manual. The minimum beam seat width shall be 2.0-ft. (0.6-m). Increase as required to support the bridge bearings with adequate cover to the anchor rods (e.g.; anchor bolts inside the top reinforcing hoop bars). Skew angle, keeper plates for seismic, future jacking needs, and adequate clear distance between the girder and the front face of the backwall needs to be considered in dimensioning the beam seat.
- The minimum beam seat height is 2-in. (50-mm). Since the tops of bearing seats are usually subjected to very large localized pressures under the bearings, reinforcement directly under

the bearings shall always be provided to prevent the formation of visible cracks or possible spalling of concrete. See Figure [6.6.4-2](#).

- The top of the abutment shall be stepped between the pedestals to provide a constant pedestal height. Do not detail the top of the abutment level with varying pedestal heights. The pedestal concrete has been breaking off and a higher pedestal increases this problem.
- All anchor bolts for bridge shoes shall be located inside the rebar cage.
- Place reinforcement so not to interfere with anchor rod locations.
- The bearing seats shall be wide enough to satisfy the requirements of *AASHTO LRFD 4.7.4.4*.

C. Abutment Stem and Backwall

- Top of abutment backwalls shall be a minimum 1.25-ft. (0.380-m) wide, excluding the approach slab seat.
- The approach slab seat width shall be a minimum 6-in. (150-mm) wide for fixed ends and for an expansion joint located in front of the backwall.
- The approach slab seat width shall be a minimum of 9-in. (228-mm) wide for an asphaltic plug expansion joint and 1.5-ft. (0.450-m) for an expansion joint located behind the backwall.
- The abutment stem shall be vertical with no batter. The cost of labor required to form a batter wall outweighs the cost of concrete.
- 4-in. (102-mm) diameter weepers shall be placed symmetrically in the abutment stem 10-ft. (3-m) apart and centered at 12-in. (300-mm) above the top of the footings. Weepers shall be sloped to drain with a 12:1 slope.
- For skew angles greater than 30°, detail a 3-in. (75-mm) minimum chamfer at acute corners.
- Concrete for the abutment stem shall have a concrete compressive strength, $f'_c = 3,000\text{-psi}$ (2.10-kg/mm²), Concrete Class A Above Footings.
- Concrete for the abutment backwall above the beam seat construction joint (new bridge) shall have a concrete compressive strength, $f'_c = 4,000\text{-psi}$ (2.8-kg/mm²), and meet the requirements of and be paid as the deck item Concrete Bridge Deck (QC/QA) as specified in *NHDOT Standard Specification Section 520.1.2.1.2 (d)*.
- Concrete for abutment backwall above the construction joint (bridge rehabilitation) shall have a concrete compressive strength, $f'_c = 4,000\text{-psi}$ (2.8-kg/mm²) and be paid as full-depth deck repairs and concrete curbs, Item 520.0201 Concrete Class AA, Above Footings. This item is used for the following reasons:
 - The “Above Footing” concrete items are items that require forming such as full-depth repairs, curbs and backwall work. The Concrete Class AA items that don’t have “Above Footings”, do not require forming of the concrete, such as partial-depth deck repair, Item 520.01, Concrete Class AA.
 - The item is a non-final pay item since the full-depth repair item is an estimate.
 - The concrete in rehabilitated bridges is not required to meet any QC/QA requirements.
- Reinforcing steel in the abutment backwall above beam seat construction joint (deck over backwall) shall be grade 60, black bars, except the approach slab dowel shall be epoxy coated. The top of the backwall is protected by the deck over it, so no additional protection is needed for the reinforcing bars.

- Reinforcing steel in the abutment backwall above construction joint between backwall and expansion joint blockout (expansion joint in front of the backwall) shall be grade 60, epoxy coated and include the hoop, horizontal and vertical bars that extend into the backwall. This portion of the backwall is exposed to road salt and the elements.
- Reinforcing steel in the pilasters (corners of abutments) shall be grade 60, epoxy coated. This includes all reinforcing bars from the top of the pilaster to the construction joint in the pilaster. The top of the pilaster is exposed to road salt and the elements.
- All approach slab dowels shall be grade 60, epoxy coated.
- In order to resist the formation of temperature and shrinkage cracks and to provide reinforcement for distribution of loads, all front and back faces of walls and abutments shall be provided with horizontal and vertical reinforcing in accordance with *AASHTO LRFD 5.10.8*. Minimum horizontal and vertical reinforcing of #5 (#16) bars spaced at 12-in.(300-mm) shall be provided.
- All abutment stem reinforcing shall be detailed and dimensioned from footing J-bars and shall not be less than #5 (#16) bars at 12-in. (300-mm) on center.
- Soil reinforcements (such as steel strips and bar mats used in mechanically stabilized earth (MSE) wall construction) can be used as attachments to abutment diaphragms or stem walls in an attempt to resist lateral loads applied to these components when part of an overall abutment MSE wall design.
- See [Appendix 6.4-B1](#) and Chapter 11, Section 11.6 and 11.8 for additional information regarding detailing.

D. Wall Joints

If vertical contraction joints are not provided in long abutments, nature usually creates them. To prevent uncontrolled cracking in the abutment stem or cracking at the abutment-wing joint, stem pours are limited to a maximum of 30-ft. (9-m) \pm measured horizontally from the center of the abutment wall to the corners (*AASHTO LRFD 11.6.1.6*). Use a vertical contraction joint if the abutment width exceeds 60-ft. (18-m).

Vertical expansion joints are required at a maximum of 90-ft. (27-m) \pm measured horizontally from the center of the abutment wall to the corners and at the abutment-wing joint (*AASHTO LRFD 11.6.1.6*).

Expansion or contraction joints shall not be provided in footings. Footings for abutments and walls shall be continuous including any steps provided.

Horizontal construction joints shall be provided between the footing and the stem, between the stem and backwall, between the backwall and backwall blockout for an expansion joint, and elsewhere as conditions warrant. In general, construction joints are keyed to hold the parts in line. Shear keys are provided in contraction, construction and expansion joints but not in abutment-wing joints.

- Reinforcing steel shall be extended through construction joints. No reinforcement shall pass through expansion and contraction joints.
- Water stops are used in expansion and contraction joints to prevent leakage and staining.
- A 2-ft. (0.6-m) wide barrier membrane shall be placed centered over the beam seat construction joint and centered over all vertical wall joints.
- The location of all joints shall be indicated on the contract drawings.

- See [Appendix 6.4-B3](#) for details of wall joints.

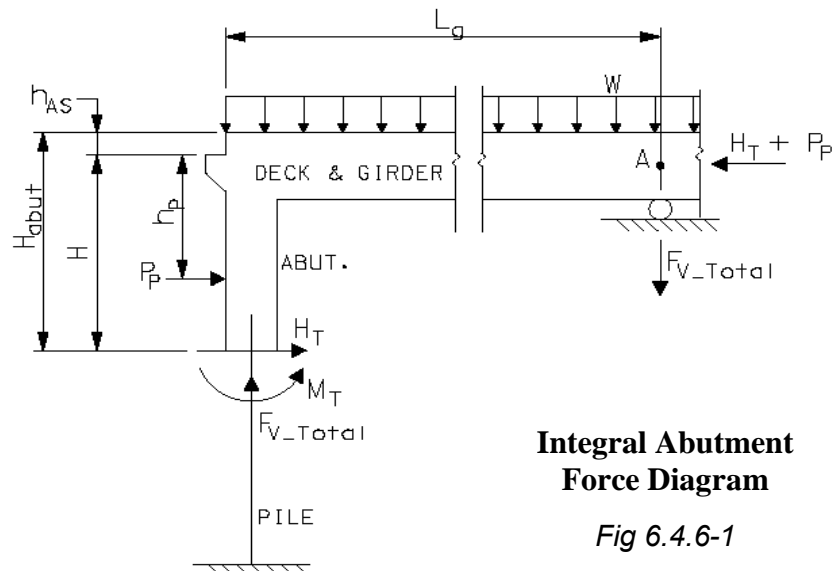
6.4.6 Design/Analysis for Integral Abutments

All components of the integral abutment must satisfy the appropriate strength and serviceability requirements. The three major parts of the abutment consist of the diaphragm, stem and pile. The design will be based on a one-foot strip. See Figure 6.4.6-1 for an integral abutment force diagram.

Typical integral abutment design is in accordance with *Vermont Agency of Transportation, Integral Abutment Bridge Design*.

Examples and information of integral abutment designs can be found from other states as noted:

- Vermont Agency of Transportation:
http://vtransengineering.vermont.gov/sites/aot_program_development/files/documents/structures/SEI-08-004-1.pdf
- Virginia Transportation Research Council, The Behavior of Integral Abutment Bridges:
http://www.virginiadot.org/vtrc/main/online_reports/pdf/00-cr3.pdf
- Mass DOT LRFD Bridge Manual Part I, Chapter 3.10:
<http://www.massdot.state.ma.us/highway/DoingBusinessWithUs/ManualsPublicationsForms/LRFDBridgeManual2013Edition/PartIDesignGuidelines.aspx>
- Indiana DOT, Integral Abutment Design Example:
[http://www.in.gov/dot/div/contracts/design/lrfd/14_Section%207.1_Integral%20Abutments\(E\).PDF](http://www.in.gov/dot/div/contracts/design/lrfd/14_Section%207.1_Integral%20Abutments(E).PDF)
- The 2005 – FHWA Conference: Integral Abutment and Jointless Bridges (IAJB 2005):
<http://www.cemr.wvu.edu/cfc/conference/Proceeding.pdf>



A. Load Cases

The design of the integral abutment shall consider the combined load effects at various stages of bridge construction and at the final stage. Refer to *Vermont Agency of Transportation, Integral Abutment Bridge Design Manual* for the load cases to be analyzed.

B. Lateral Earth Pressure

The intensity of lateral earth pressure is a function of the type of soil and amount of anticipated backfill movement relative to the wall height. Thus the lateral earth pressure distribution is dependent on the soil and pile interaction and is some value between the at-rest earth pressure and the full passive earth pressure. The approximate values of relative movements required to reach full passive pressure are provided in *AASHTO LRFD Table C3.11.1-1*. However, several other sources provide guidance with respect to the soil pressures developed when these movements are not realized and when the soil pressure is some value between the at rest pressure and the full passive pressure. One such reference is the *US Department of the Navy Design Manual – Foundations and Earth Structures, NAVFAC DM-7*.

C. Thermal Movements

Thermal movement is a major source of the loads on the abutment and abutment piles. The thermal movement shall be determined in accordance with Chapter 4, Section 4.3.7 of this manual.

D. Load Factors

A load factor, γ_{TU} of 1.0 (Service I Limit State) shall be applied when calculating the horizontal displacement due to temperature change, creep and shortening for integral bridges (steel substructures – piles) (*AASHTO 3.4.1 p. 3-12*).

The load factor for passive pressure γ_{EH} shall be taken as 1.50. The load factors for at rest and active earth pressure shall be as specified in *AASHTO LRFD Table 3.4.1-2*.

E. Pile to Superstructure Connection

The connection between the pile and the abutment shall be assumed to be rigid. Piles shall be embedded a minimum of 2.0-ft. (0.6-m) into abutment stem or as required to develop the pile plastic moment capacity. Designers shall verify that the 2.0-ft. (0.6-m) is adequate for the embedment. Additional embedment may be required depending on the pile size and direction.

The integral abutment shall be adequately designed and detailed to transfer all applied loads from the superstructure. The piles shall be adequately anchored into the abutment stem in case of any unanticipated uplift movements.

The connection of the bridge deck to the abutment diaphragm shall be reinforced to resist the moments caused by superstructure rotation under superimposed dead and live loads. The beneficial effects of end fixity shall not be used to reduce the design moments in the beams.

F. Pile Design

Integral abutments shall be supported on a single row of H-piles. The piles are oriented in either their weak or strong axis perpendicular to the longitudinal axis of the bridge, depending upon the requirements of the design, regardless of the bridge skew angle.

All piles shall be driven to a depth meeting the minimum pile penetration and design requirements of *AASHTO LRFD 10.7* and the recommendations of the Geotechnical Engineer.

The nominal pile structural resistance shall be determined in accordance with the requirements of Chapter 6, Section 6.3 of this manual.

One (1) pile per beam line at each abutment shall be used as a guideline.

In general, the effects of skew, curvature, thermal expansion of the superstructure, reveal, and grade are considered in the pile design and therefore, vertical as well as horizontal forces due to all applicable loads shall be applied to the piles. The piles are to be designed for both vertical loads and for bending. The interaction between the piles and the surrounding soil shall be considered.

The design of an axially loaded pile subject to lateral forces and/or lateral deformation involves a process which accounts for the soil-pile interaction, in that the pile deflection is dependent on the soil response and the soil response is a function of the pile deflection. The piles need sufficient soil embedment to resist the pile loads.

The analysis and design of piles is based on the principle that the pile will behave either elastically or inelastically. Once the plastic moment is reached and adequate pile ductility exists, a plastic hinge will form and no further increase in pile moment at the pile top will be achieved and there will be a redistribution of forces associated with this inelastic behavior. Therefore, as described below, the design of integral abutment piles may be based on a conventional elastic design approach or an inelastic design approach and shall be as follows:

Perform an iterative analysis by first determining the factored thermal movement and the factored pile axial load based on the calculated depth to fixity. Then analyze the pile using a computer program such as L-pile using the previously calculated loads assuming a fixed head condition with zero degrees of rotation of the pile cap. Compare the factored pile head moment from the program output to the pile plastic moment capacity and design the pile according to one of the following:

- Use a conventional elastic design method if the factored pile head moment from the above analysis is less than the plastic moment capacity of the pile: The design of the pile shall satisfy the *AASHTO LRFD Bridge Design Specification* combined axial compression and flexural interaction relationship (the factored axial load shall be the applied factored pile axial load and the pile factored design moment shall be the maximum factored bending moment as determined from the above analysis).
- Use an inelastic design approach if the factored pile head moment from the above analysis exceeds the plastic moment capacity of the pile: Perform an analysis by applying the pile head plastic moment capacity and the factored thermal movement. The design of the pile shall satisfy the combined axial compression and flexural interaction relationship. The factored axial load shall be the applied factored pile axial load and the factored bending moment shall be the maximum factored bending moment between the two points of zero moment closest to the abutment (from the above analysis).

The piles must have sufficient ductility at the pile head to accommodate the internal force redistribution resulting from the plastic hinge rotation.

In the above designs (except for short piles) three sections of the pile should be checked for determining the pile axial capacity. The first segment of the unbraced length is from the bottom of the stem to the first zero moment. The second segment of unbraced length is between the two zero moments. The third section to be checked is assuming a fully braced length below the second zero moment. For short piles with only one point of zero moment, the length shall be from the point of zero moment to the pile tip. In all design cases, the pile shall be considered pinned at both ends.

The theoretical depth of fixity is the depth at which the pile is firmly held by the soil (typically considered the second point of zero lateral deflection).

G. Substructure Design

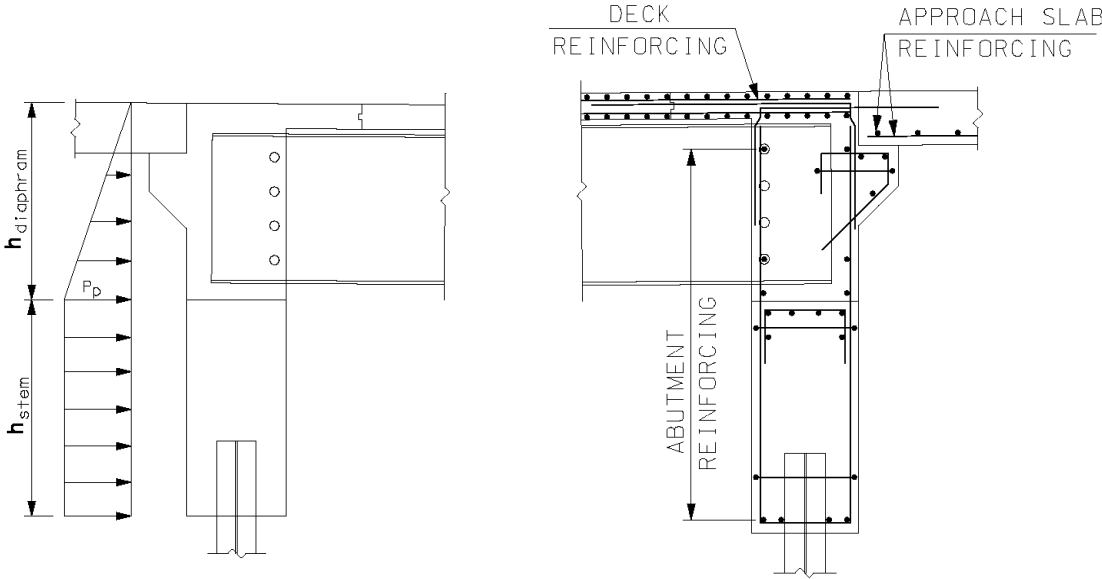
Refer to Figure 6.4.6-2, 3 & 4 for abutment design descriptions and reinforcement.

Design abutment stem top and bottom horizontal bars for vertical loads uniformly distributed over the abutment and the stem to be a continuous beam with piles as supports.

Design abutment stem back face vertical dowels for the moment due to passive soil pressure that develops when the bridge expands, combined with the moment due to pile rotation. Assume the abutment stem acts as a cantilever fixed at the bottom of the diaphragm and free at the bottom of the stem. Determine the passive pressure at the elevation of the bottom of the diaphragm and apply as a uniform pressure on the stem.

Design abutment diaphragm horizontal bars (between girders) for the moment due to passive soil pressure which results when the bridge expands and the moment due to pile rotation. For this case, consider the diaphragm to be a continuous beam with the superstructure girders as supports.

Design the abutment stem horizontal bars for the moment due to passive soil pressure which results when the bridge expands.

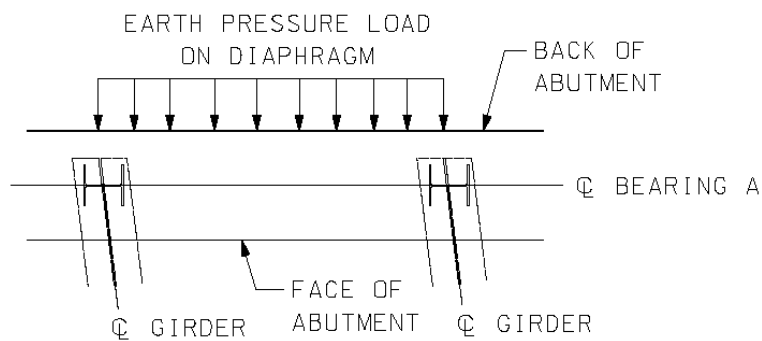


Integral Abutment Passive Pressure Diagram

Fig 6.4.6-2

Integral Abutment Reinforcement

Fig 6.4.6-3



**Integral Abutment Plan
Diaphragm Pressure
Diagram**

Fig 6.4.6-4

H. Concrete Deck Pour Sequence

The concrete deck pouring sequence shall be indicated in the contract drawings. It shall specify that the main portion of the deck be poured prior to the deck ends. This will permit all the dead load girder rotations to take place without any rotational forces being transferred to the piles. A construction joint shall be shown 4-ft. (1.2-m) from the abutment face. The end 4-ft. (1.2-m) of the deck shall be poured monolithic with the abutment cap.

I. Expansion joint, Approach Slab, Sleeper Slab

For abutment movements in excess of 0.25-in. (6.4-mm), movements shall be accommodated with an appropriately designed expansion joint at the free end of the approach slab where a sleeper slab shall be detailed to support the free end and to accommodate the expansion joint detail. For movements less than or equal to 0.25-in. (6.4-mm) a crack seal detail may be used at the end of the sleeper slab. Details of approach and sleeper slabs are shown in [Appendix 6.4-B2](#).

Approach slabs shall be designed to move with the abutment.

J. Wingwalls

In-line wingwalls (parallel and cantilevered from the abutment), not exceeding 10-ft. (3-m) in length (measured from the outside face of the coping), are the preferred arrangement. See [Figure 6.5.2-2](#) for wingwall configurations.

For monolithic wingwalls, design the horizontal steel at the intersection of the wingwall and the abutment to resist the cantilever forces induced by earth pressures acting behind the wingwall. For design purposes, the intersection of the wingwall and abutment shall be assumed to be a vertical line extended down from the outside of the coping. For in-line wingwalls (parallel with the abutment), the earth pressure is passive due to the expansion force in the girders. With flared wingwalls, forces comprised of passive pressure acting in a direction perpendicular to the abutment centerline and an active component acting in a direction perpendicular to the centerline of the roadway are present. Do not put piles or a footing under wingwalls that are monolithically attached to the abutment. Wingwalls requiring a length longer than 10-ft. (3-m) shall be split into two segments. The first segment shall be monolithically attached to the abutment and the second shall be designed as a freestanding cantilever wall and shall be isolated from the movement of the bridge.

K. Piers

Design of piers for integral abutment bridges is similar to the design of piers for a traditional bridge. The bearings at the pier are typically expansion bearings. Fixed bearings can be used when the expected amount of expansion in the spans adjacent to the pier is equal.

The following types of piers can be used with an integral abutment design:

- 1) Flexible Piers: piers rigidly connected to the superstructure. This pier type is assumed to provide vertical support only.
- 2) Isolated Rigid Piers: piers with a base fixed against rotation and translation and elastomeric bearings detailed with keeper plates/blocks that allow longitudinal movement of the superstructure but restrain transverse movements. Typically used with steel girder bridges that has expansion piers.
- 3) Semi-rigid Piers: piers with a fixed base connected to the superstructure with dowels and detailed with elastomeric bearings. Typically used with prestressed concrete girders with diaphragms between the girders. Dowels connect the diaphragm to the pier cap which forces the pier to move with the superstructure.

6.4.7 Details for Integral Abutments

The following applies to the detailing of integral abutments:

A. Backfill and Drainage

The bridge designer shall determine where the drainage will occur around the bridge. Adequate drainage of fill behind structures is important to increase the longevity of retaining structures.

A proper drainage system shall be provided to eliminate hydrostatic pressure and control erosion of the underside of the abutment embankment slope protection. A drainage system is of great importance when there is potential for a perched or high groundwater condition, when the bridge is located in a sag curve, when the bridge is located in a cut section with saturated subgrade, or when there is significant pavement water runoff to side slopes.

The embankment behind the integral abutments shall be positively drained with an adequately designed drainage system consisting of an underdrain system and free draining granular backfill. See [Appendix 6.4-B2](#) for details.

All abutments, wingwalls and retaining walls shall be backfilled with Granular Backfill (Bridge) in accordance with *NHDOT Standard Specifications for Road and Bridge Construction* to the limits as shown on the contract plans. Typically the backfill limits extend 1-ft. (0.3-m) in front of the abutment and wingwall stem, extend 5-ft. (1.5-m) behind the stem with a 1:1 slope to the bottom of the sleeper slab structural fill. Granular Backfill (Bridge) is a clean, free draining, very dense, granular soil (See Chapter 4, Section 4.3.3 for the soil properties). See [Appendix 6.4-B2](#) for details.

B. Utilities

In general, utilities with rigid pipes should not pass through integral abutments. If this cannot be avoided, then they must be properly detailed (appropriately sleeved) to accommodate the anticipated superstructure translational and rotational movements.

Flexible conduits for electrical or telephone utilities that are properly equipped with an expansion sleeve through the integral abutment are acceptable.

C. Detailing

Integral abutments shall include the following:

- A continuous corbel for support of the approach slab.
- A sleeper slab shall be used with all approach slabs.
- Reinforcing to sufficiently tie approach slabs to abutments.
- An underdrain system.
- A minimum of one (1) pile per beam line at each abutment.
- A reinforced abutment cap beam-diaphragm to distribute superstructure loads to piles and resist active and passive earth pressures.
- Sufficient shrinkage reinforcement in the deck slab above the abutment.
- Transverse reinforcing around each pile head.
- Continuity reinforcing to provide a rigid connection between abutment and superstructure.
- Grade 60 reinforcing, epoxy coated.
- Wings for retaining soil at the top of a shallow-slope embankment.
- A minimum of 3-ft. (0.9-m) from bottom of beam to top of berm to allow inspection access.
- A single row of compact section H-piles oriented with either their weak or strong axis parallel to the longitudinal axis of the bridge.
- Holes in steel girder webs for reinforcing steel according to design.
- Painted ends of girders to 10-ft. (3.0-m) beyond the front face of abutment.
- A crack seal joint in the pavement between the deck slab and approach slab.
- A roughened interior of the construction joint surface at the beam seat. Rake parallel with the face of the abutment to amplitude of ½ inch (13-mm).
- Beam anchorage shall include:
 - An anchor bolt on each side of the web, securing steel flanges of the girder with a double nut moment connection; Grade 50 (345), A449 or F1554 swedged with the top threaded to the construction joint, 2-in. (51-mm) in diameter and embedded a minimum of 18-in. (450-mm).
 - A 1-in. (25-mm) elastomeric pad, fully supporting the full width of the concrete bottom flange.
- See [Appendix 6.4-B2](#) for details.

6.4.8 Design/Analysis for Semi-Integral Abutments

Semi-Integral abutments shall be designed to resist all dead loads and live loads as well as all horizontal loads and movements. The minimum requirements for loads and their application, the applicable load factors, and the applicable load combinations shall all be in accordance with the requirements of the *AASHTO LRFD Section 3* and as modified in Chapter 4 of this manual.

The expansion and contraction movement of the superstructure shall be accommodated at the roadway end of the approach slab. The geometry of the approach slab and wingwalls must be

compatible with the freedom required for the semi-integral configuration (beams, deck, backwall and approach slab) to move longitudinally.

When phase construction is used with semi-integral abutments, the use of a closure placement between phases in the backwall shall be considered. The use of a closure placement can reduce the mismatch of the top of slab between phases caused by deflection from the superstructure.

Semi-integral abutment diaphragm vertical bars are designed to resist the passive pressure that develops when the bridge expands.

Semi-integral abutment diaphragm horizontal reinforcement can be designed using the Integral Abutment Design in Section [6.4.6](#), provided all of the criteria for the design guide are met. When using this guide for semi-integral abutments, the stem height requirement may be ignored.

6.4.9 Details for Semi-Integral Abutments

A. Backfill and Drainage

Semi-integral abutments backfill and drainage details are the same as required for integral abutments. See Section [6.4.7A](#).

B. Utilities

The detailing requirements for utilities passing through semi-integral abutments shall be the same as those for integral abutments. See Section [6.4.7B](#).

C. Expansion Joint, Approach Slab and Sleeper Slab

The expansion joint, approach slab, and sleeper slab details for semi-integral abutment bridges are the same as those for integral abutment bridges. See Section [6.4.6I](#).

D. Detailing

Semi-integral abutments shall include the following:

- A continuous corbel for support of the approach slab.
- A sleeper slab shall be used with all approach slabs.
- Reinforcing to sufficiently tie approach slabs to abutments.
- An underdrain system.
- A seat for the superstructure beam.
- Continuity reinforcement to provide a rigid connection between abutment and superstructure.
- Painted ends of girders to 10-ft. (3.0-m) beyond the front face of abutment.
- A crack seal joint in the pavement between the deck slab and approach slab.
- A minimum of 3-ft. (0.9-m) from bottom of beam to top of berm to allow inspection access.
- Designed bearings.

6.5 Retaining Walls

6.5.1 General

A retaining wall is a structure that provides lateral support for a mass of soil. A properly designed retaining wall will not fail by overturning, sliding, excessive settlement, or excessive bearing pressures or pile loads; and the structure itself possesses adequate strength to resist the applied earth and live loadings.

A retaining wall adjacent to a bridge abutment is commonly referred to in bridge plans as a wingwall.

Walls supporting a highway embankment are commonly referred to as retaining walls. These walls are used in a variety of applications including right-of-way restrictions, protection of existing structures that must remain in place, grade separations, new highway embankment construction, roadway widening, stabilization of slopes, protection of environmentally sensitive areas, staging, and temporary support including excavation or underwater construction support, etc.

Several types of retaining wall systems are available to retain earth and meet specific project requirements. Many of these wall systems are proprietary. The wall selection criteria and design policies presented in this chapter are to ensure consistency of standards and applications used throughout NHDOT projects.

6.5.2 Abutment Wingwalls

A. Design

- Wingwalls are designed as retaining walls. Earth loads and surcharge loads are applied to wingwalls similar to how they are applied to the stem of a retaining wall. Loading and design criteria shall be in accordance with *AASHTO LRFD*, and Chapters 4 and 6 of this manual.
- Wingwalls without footings that are poured monolithically with the abutment are subjected to bending moment, shear force and torsion. The primary force is the bending moment.
- Wingwalls with a footing shall be separated from the abutment with a joint. They shall be designed as unrestrained, which means that they are free to rotate at the top in an active state of earth pressure. This applies even when the wingwall footing is continuous with the abutment footing.
- The design of U-back wingwalls (parallel to the roadway) may be subject to frost pressure where ice could be trapped and result in a build-up of pressure. If applicable, apply a frost pressure loading of 0.7-kip/linear ft. (10.2-kN/m) at the top of the wingwall. This loading condition applies for the design of stem reinforcing only, not for wall stability or footing design. Do not apply frost pressure simultaneously with a live load surcharge loading.
- Passive earth pressure resistance in front of a wingwall footing or stem may not be dependable due to potential for erosion, scour, or future excavation. *Do not* include passive earth pressure in stability calculations, unless approved by the Design Chief of the Bureau of Bridge Design.
- The design section for long wingwalls with a variable height and no butterfly section shall be taken at the higher third point of the wall.
- If a U-back wingwall has a butterfly section, the wall shall be designed using the maximum wall height with butterfly effects distributed over the wall length. The maximum moment for the horizontal reinforcement occurs at the bottom corner of the butterfly wing due to torsion.

Thus, the horizontal reinforcement and reinforcement that follows the slope of the bottom of the butterfly needs adequate development into wall.

- If a flared wingwall has a variable height with a butterfly section, the wall design section shall be the higher third point of the wall with butterfly effects distributed over the wall length.
- The designer shall ensure that the backfill behind a wingwall is adequately drained using weepers. If the backfill cannot be drained adequately the designer shall add full hydrostatic pressure (WA) to the lateral earth pressure.
- See Figure [6.5.2-2](#) for wingwall configurations.

B. Railings/Barriers on Top of Wingwalls

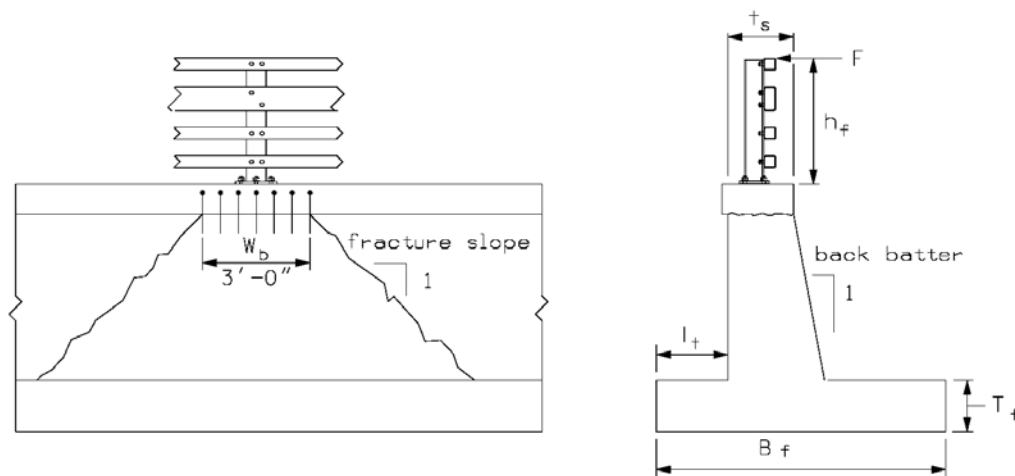
Concrete wingwalls (retaining walls) with railing/barrier mounted on top shall be designed for local wall stability (overturning, sliding and bearing pressures) and structural capacity.

Per NHDOT Bureau of Bridge Design, the vehicular collision load (Extreme Event II Limit State) of 54-kips (TL-4) or the appropriate load for the location, shall be applied *only* to the stem structural capacity design (stem reinforcing, including reinforcing J-bars extending into the footing) along with other applicable loads.

For designing the wall stem reinforcing, the vehicular collision load shall be distributed a distance (W_b) at the rail post. W_b shall be the total distance of the coping hoop reinforcing required at each post (7-#5 bars at 6-in. = 3-ft.) which has been crash tested and proven to distribute the load. The vehicular collision load shall be distributed over 3-ft. at the top of the wall and then distributed down to the footing at a 1:1 slope and shall have a load factor of 1.0. The loading is not applied to the footing design. See Figure 6.5.2-1.

The horizontal earth pressure shall act concurrently with the vehicular collision load because the horizontal earth pressure has already induced a strain in the reinforcing bars. The total strain in the rebar shall include the vehicular collision load along with the horizontal earth pressure, and other applicable loads.

See Section 6.5.6 [C](#) for barriers placed on top of MSE wall systems.



Wingwall with Railing/Barrier

Fig 6.5.2-1

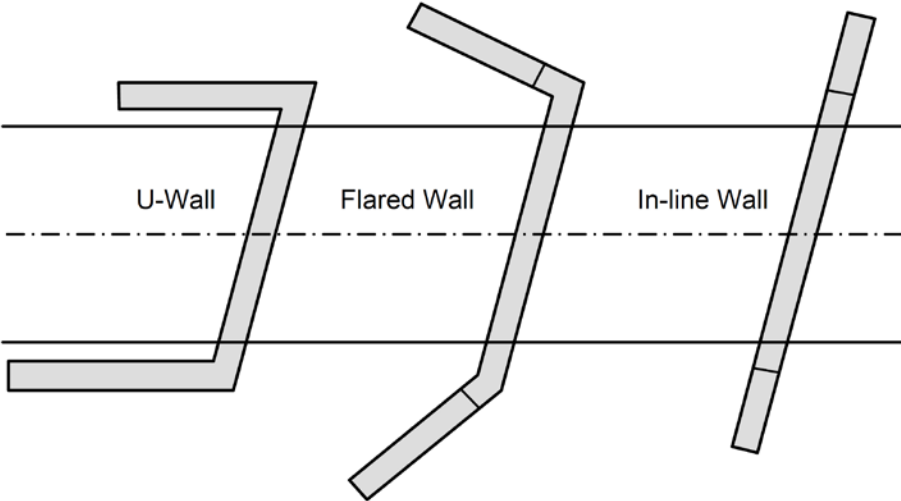
A. Detailing

- See [Appendix 6.5 – B1](#) for wingwall details and Figure [6.5.2-2](#) for wingwall configurations.
- The back face of wingwalls shall be detailed to be either vertical or a 1:12 batter.
- The minimum thickness at top of wall shall be 1'-3" (380-mm). If a traffic railing is placed on top of the wall, the minimum thickness shall be 1'-10" (560-mm) with a 2'-0" (610-mm) wide cap.
- Wingwall lengths shall be measured in 6-in. (150-mm) increments. This allows for the footing J-bar reinforcing (spaced at 6-in. [150-mm] increments with 3-in. [75-mm] cover) to be evenly spaced in the wingwalls.
- Flared wingwalls shall be the first option considered when laying out wingwall geometry. The flared wingwall gives the shortest length, lowest initial cost, and few conflicts with guardrail posts, especially if a MSE wall system is used.
- U-back wingwalls (parallel to the roadway) shall be considered for the following conditions:
 - To avoid conflicts with cofferdams or provide ease of cofferdam construction.
 - To avoid impacts to the existing bridge or approach roadway.
 - To avoid construction of the wingwall in construction phasing.
- Weepholes shall be used in all wingwalls. When unusual drainage conditions exist including wall constructed in a cut section, perforated drains behind the heel of the wall shall be considered.
- In order to resist the formation of temperature and shrinkage cracks and to provide reinforcement for distribution of loads, all faces of wingwalls shall be provided with a minimum horizontal and vertical reinforcing of #5 (#16) spaced at 12-in. (300-mm).
- Vertical contraction joints shall be spaced no more than 30-ft. (9-m)± of wall length. Reinforcing steel does not extend through the joint. See [Appendix 6.4-B3](#) for detail.
- Vertical expansion joints shall be spaced not more than 90-ft. (27-m)± of wall length. Reinforcing steel does not extend through the joint. See [Appendix 6.4-B3](#) for detail.
- The location of vertical and horizontal joints, detailed with appropriate shear keys, shall be indicated on the contract drawing.
- A seat for the granite approach curb shall be formed at the ends of U-back wingwalls.
- Wingwalls greater than 20-ft. (6-m) in length can include a butterfly (cantilever portion) at the end of the wingwall. The maximum butterfly length shall be 10-ft. (3-m) and shall not be longer than one-third the total wingwall length.
- A butterfly section shall be considered for the following reasons:
 - To avoid conflicts with adjacent structures, temporary sheeting or utilities.
 - To avoid undermining of adjacent footings.
 - To minimize bedrock excavation.
- The wingwall length shall be sufficient so the sloping soil will intersect the berm elevation in front of the abutment and not spill onto the bridge seat. For river crossings, the typical slope is 1 vertical: 2 horizontal. A maximum slope of 1:1.5 can be used if necessary.
- The end of a flared wingwall is located where the shoulder break from the roadway meets the underbridge embankment slope. The intersection shall occur at the rear corner of the wingwall. The elevation of the top of the wingwall shall be 3-in. (75-mm) higher than this intersection and stated on the plans. See Chapter 2, Figure 2.4.1-2.

- Curved wingwalls shall be avoided whenever possible. If it is absolutely necessary to provide a curved wingwall, it is best to place a widened footing on a chord and only curve the wall. Curved wingwalls shall never be battered since the forming is extremely difficult.
- Wingwall foundations shall match the abutment foundation requirements (e.g., a pile supported abutment will always have pile-supported wingwalls) except for integral abutments.

B. Slope Protection

Bank protection for wingwalls at river crossings shall be provided as noted in Chapter 2, Section 2.7.7.C Channel Protection.



Wingwall Configurations

Fig 6.5.2-2

6.5.3 Retaining Wall Types

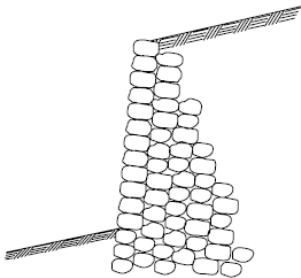
The key considerations in deciding which wall is feasible are the amount of excavation or shoring required and the overall wall height. The site geometric constraints must be well-defined to determine these elements.

Consider the following items when determining the type of retaining wall:

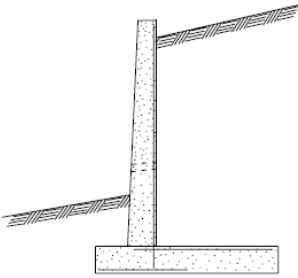
- Functional classification
- Highway geometry
- Design Clear Zone requirements
- Amount of excavation required

- Traffic characteristics
- Constructability and traffic control
- Impact to adjacent environmentally sensitive areas
- Impact to adjacent structures
- Potential added lanes
- Length and height of wall
- Material to be retained
- Foundation support, external stability, and potential for differential settlement
- Groundwater and frost protection depth
- Earthquake loads
- Right of way costs
- Need for construction easements
- Risk
- Overall cost
- Maintenance concerns
- Utilities
- Visual appearance

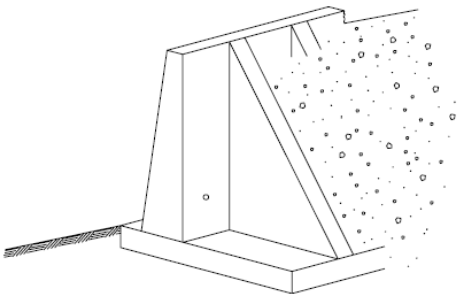
Retaining walls are generally classified as gravity, semigravity, nongravity cantilever, or anchored. Some systems are more suitable as permanent installations, while others are better suited as temporary walls; some are more applicable for rural areas, while others are more suited for suburban areas. The selection of the most appropriate system will thus depend on the specific project requirements. See Figures 6.5.3-[1](#), [2](#), & [3](#) for Wall Types and [Appendix 6.5-A1](#) for Wall Systems Selection Tables.



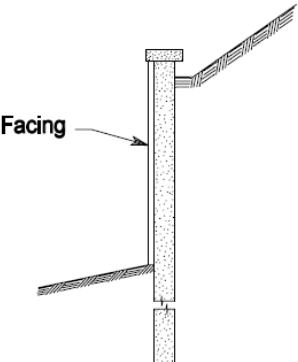
Mortar Rubble Masonry Rigid Gravity Wall



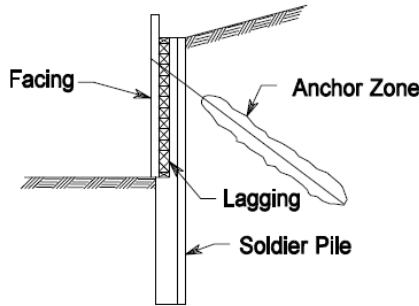
Reinforced Concrete Cantilever Semigravity Wall



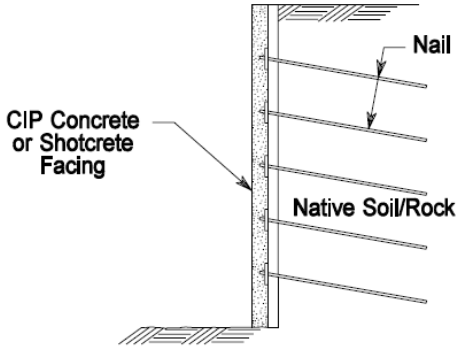
Reinforced Concrete Counterfort Semigravity Wall



Slurry or Cylinder Pile Nongravity Cantilever Wall



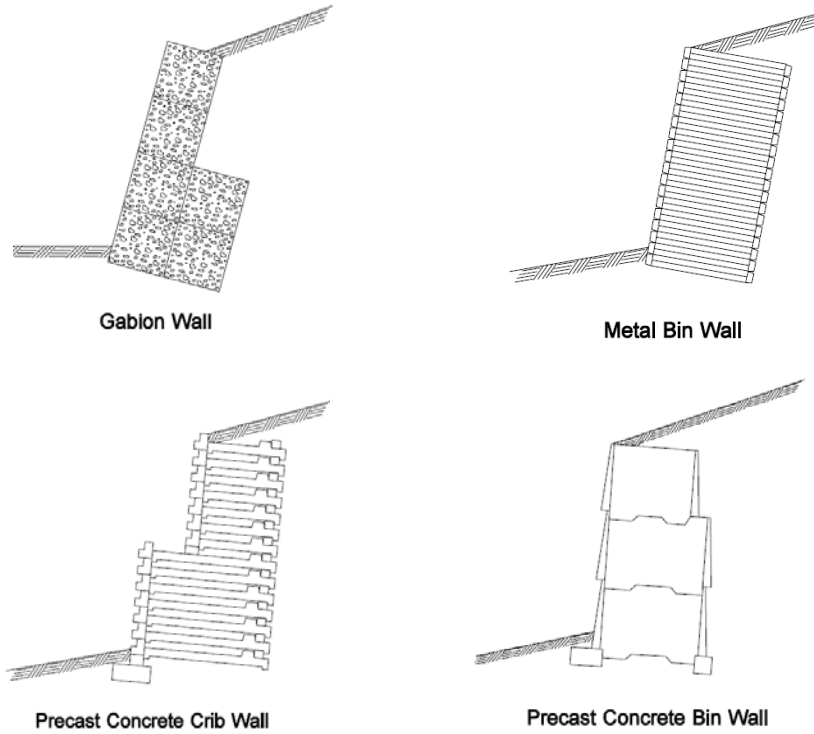
Soldier Pile Tieback Wall



Soil Nail Wall in Cut

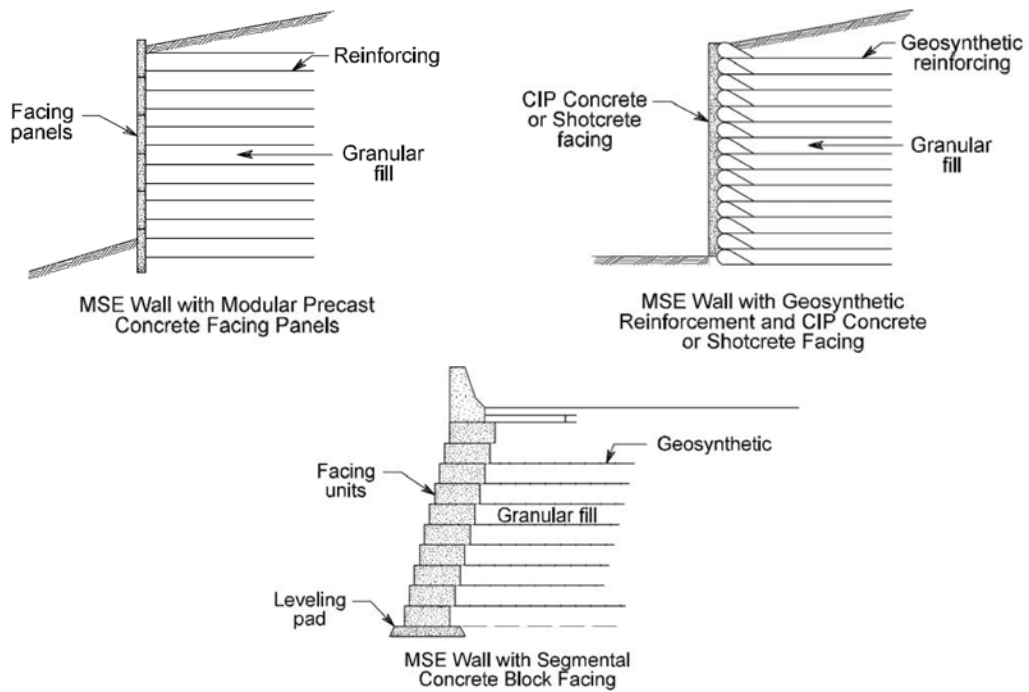
Rigid Gravity, Semigravity, Cantilever, Nongravity Cantilever, and Anchored Wall Types

Fig 6.5.3-1



Prefabricated Modular Gravity Wall Types

Fig 6.5.3-2



Mechanical Stabilized Earth Gravity Wall Types

Fig 6.5.3-3

A. Permanent or Temporary

Permanent retaining walls shall be designed for a minimum service life of 75 years, achieve an aesthetically pleasing appearance, and be maintenance free throughout their design life (*AASHTO 11.5.1*).

Temporary wall systems generally have a service life of 3 years or the project duration since they are removed upon completion of the project. MSE walls and wrapped face geosynthetic walls are well suited for support of temporary fills because they can be easily constructed and are inexpensive. Sheet pile and soldier pile lagging wall systems are well suited for cut situations where excavation room is limited.

B. Fill or Cut Walls

Retaining walls can also be classified by the method of construction; however, this is not necessarily the type of earthwork at the location. Fill wall refers to bottom-up construction. Cut wall refers to top-down construction. Due to the construction technique and base width required, some wall types are best suited for cut situations, whereas others are best suited for fill situations. For example, anchored walls and soil nail walls have soil reinforcements drilled into the in-situ soil/rock and are therefore generally used in cut situations. MSE walls and reinforced slopes, however, are constructed by placing soil reinforcement between layers of fill from the bottom up and are therefore best suited to fill situations. Furthermore, the base width of MSE and cantilever walls is typically on the order of 70% of the wall height, which requires considerable excavation in a cut situation.

The classification of each wall system according to its construction method is shown in [Appendix 6.5-A1](#).

C. Proprietary or Non-Proprietary

A proprietary retaining wall is a wall system in which the system itself or some portion thereof is patented. The internal stability of prefabricated proprietary retaining walls is typically analyzed and designed by the vendor. Proprietary retaining walls include MSE, precast concrete modular walls, and bin or crib walls.

Proprietary retaining wall systems require pre-approval by NHDOT and special provisions. The Bureau of Bridge Design maintains a list of the proprietary retaining wall systems that are pre-approved. NHDOT does not pre-approve the manufacturer, but specific wall systems by a given manufacturer.

Pre-approval of a proprietary retaining wall system and its addition to the Pre-Approved Proprietary Retaining Wall List does not constitute a blanket approval of the wall system on all projects. For projects that require a retaining wall, the Bureaus of Bridge Design, Highway Design and Materials and Research will evaluate the project wall criteria and prepare a special provision that lists the wall systems from the Pre-Approved List that are qualified specifically for use on the project. The criteria for acceptance for inclusion on the Department's Pre-Approved Proprietary Retaining Wall System List are noted in the *NHDOT Proprietary Retaining Wall System Pre-Approval Process* document. The *Pre-Approved Proprietary Retaining Wall System List* ([Appendix 6.5-A2](#)) and the pre-approval process document are located on the Bridge Design Document Webpage at <http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/documents.htm>

A non-proprietary retaining wall is fully designed and detailed by the designer and shown on the contract plans in detail. Proprietary elements as well as non-proprietary elements may be included in a non-proprietary retaining wall. Non-proprietary retaining walls include cast-in-place cantilever walls, rock walls, soil nail walls and non-gravity walls.

6.5.4 General Design Concepts

The design of a retaining wall consists of the following principal activities:

- Develop wall/slope geometry
- Provide adequate subsurface investigation
- Evaluate loads and pressures that will act on the structure
- Design the structure to withstand the loads and pressures
- Design the structure to meet aesthetic requirements
- Ensure wall/slope constructability
- Coordinate with other design elements

The structure and adjacent soil mass also needs to be stable as a system, and the anticipated wall settlement needs to be within acceptable limits.

A. Design Considerations

Retaining wall systems shall be designed in accordance with *AASHTO LRFD* and Chapter 4 and 6 of this manual. The retaining walls shall be designed to address all applicable loads, limit states (strength, service, and extreme event), frost protection, scour, bearing capacity, settlement, stability, drainage and seismic loads.

B. Aesthetics

Retaining walls can have a pleasing appearance that is compatible with the surrounding terrain and other structures in the vicinity. To the extent possible within functional requirements and cost-effectiveness criteria, this aesthetic goal is to be met for all visible permanent retaining walls.

Aesthetic requirements include consideration of the wall face material, top profile, terminals, and surface finish (texture, color, and pattern). Avoid short sections of retaining wall or steep slopes where possible. See Chapter 2, Section 2.6 and Appendix 2.6-A1 for additional information regarding aesthetics.

Approval by the Design Chef is required on all retaining wall aesthetics, including finishes, materials, and configuration.

6.5.5 Cast-In-Place Concrete Cantilever Walls

A cast-in-place, reinforced concrete cantilever wall is a semi-gravity wall. Semigravity walls rely more on structural resistance through cantilevering action of the wall stem. Generally, the backfill for a semigravity wall rests on part of the wall footing. The backfill, in combination with the weight of the wall and footing, provides the dead weight for resistance of the forces induced by the retained soil. Typically, cantilevered walls are limited to heights less than 30-ft. (9-m). See [Appendix 6.5-B2](#) for CIP concrete cantilever wall details.

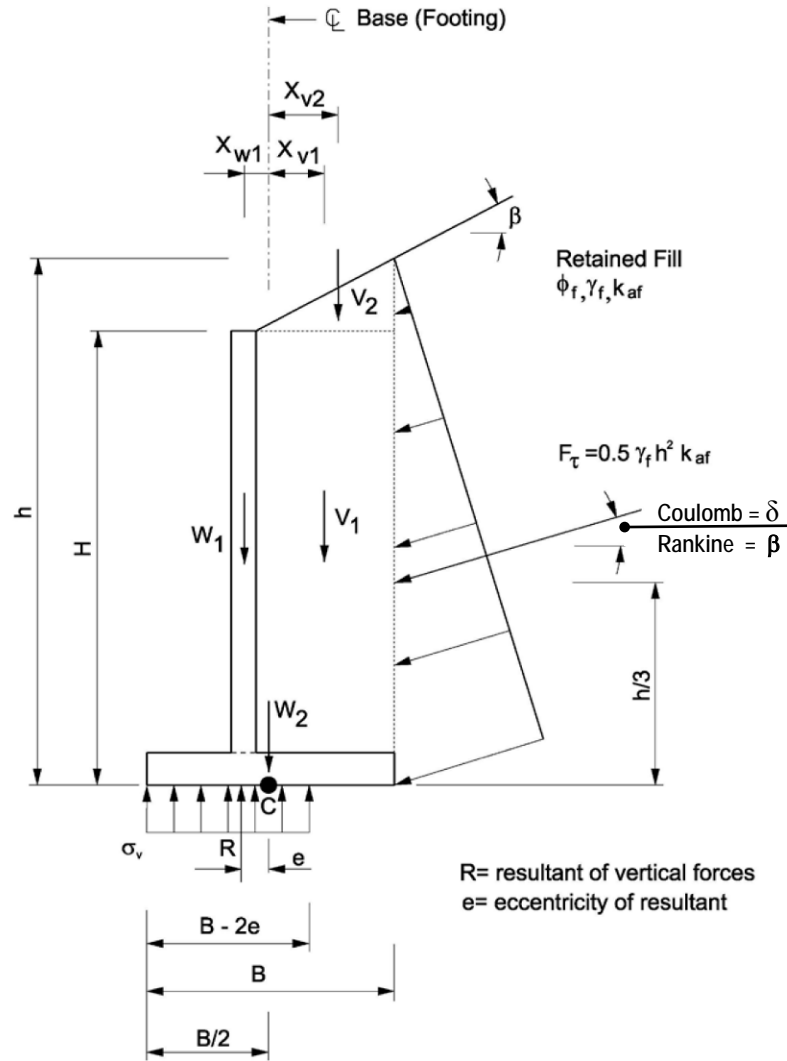
A. Design

- Earth loads and surcharge loads are applied to CIP cantilever walls similarly to the way they are applied to the stem of an abutment. Loading and design criteria shall be in accordance with *AASHTO LRFD Sections 3 and 11*, and Chapter 4 and 6 of this manual.
- Passive earth pressure resistance in front of a CIP cantilever wall footing or stem is not dependable due to potential for erosion, scour, or future excavation. *Do not* include passive earth pressure in stability calculations, unless approved by the Design Chief of the Bureau of Bridge Design.

- The designer shall ensure that the backfill behind a CIP cantilever wall is adequately drained using weepers. If the backfill cannot be drained adequately the designer shall add full hydrostatic pressure (WA) to the lateral earth pressure.
- CIP Cantilever walls with railing/barrier attached to the wall, shall be designed as noted in Section 6.5.2 [B](#).
- As per *AASHTO LRFD Section 11*, an external stability check shall be performed to evaluate bearing resistance, eccentricity, and sliding along with an overall stability check. The stem shall be designed as a cantilever supported at the footing. Both the footing and stem shall be designed with adequate flexure in accordance with *AASHTO LRFD Section 11* and this manual.
- See Figure 6.5.5-[1](#), [2](#) & [3](#) or cantilever wall loading diagrams and bearing stress criteria as noted in *AASHTO LRFD Section 11*. The vertical component of the earth pressure load due to a sloping backfill shall be included in evaluating bearing resistance, eccentricity, sliding and overturning.

B. Details

- See [Appendix 6.5 – B2](#) for CIP cantilever wall details.
- The back face of cantilever wall shall be detailed to be vertical or with a 1:12 batter.
- The minimum thickness at top of the wall shall be 1'-3" (380-mm). If a traffic railing is placed on top of the wall, the minimum thickness at the top of the wall shall be 1'-10" (560-mm) with a 2'-0" (610-mm) wide cap.
- The wall lengths shall be measured in 6-in. (150-mm) increments. This allows for the footing J-bar reinforcing (spaced at 6-in. [150-mm] increments with 3-in. [75-mm] cover) to be spaced evenly in the cantilever wall.
- Weepholes shall be used in all cantilever walls. When unusual drainage conditions exist, perforated drains behind the walls shall be considered.
- In order to resist the formation of temperature and shrinkage cracks and to provide reinforcement for distribution of loads, all faces of CIP cantilever walls shall be provided with a minimum horizontal and vertical reinforcing of #5 (#16) spaced at 12-in. (300-mm).
- Vertical contraction joints shall be spaced no more than 30-ft. (9-m)± of wall length. Reinforcing steel does not extend through the joint. See [Appendix 6.4-B3](#) for details.
- Vertical expansion joints shall be spaced not more than 90-ft. (27-m)± of wall length. Reinforcing steel does not extend through the joint. See [Appendix 6.4-B3](#) for details.
- The location of vertical and horizontal joints, detailed with appropriate shear keys, shall be indicated on the contract drawing.
- Bank protection for walls along a river shall be provided as noted in Chapter 2, Section 2.7.7.C Channel Protection.

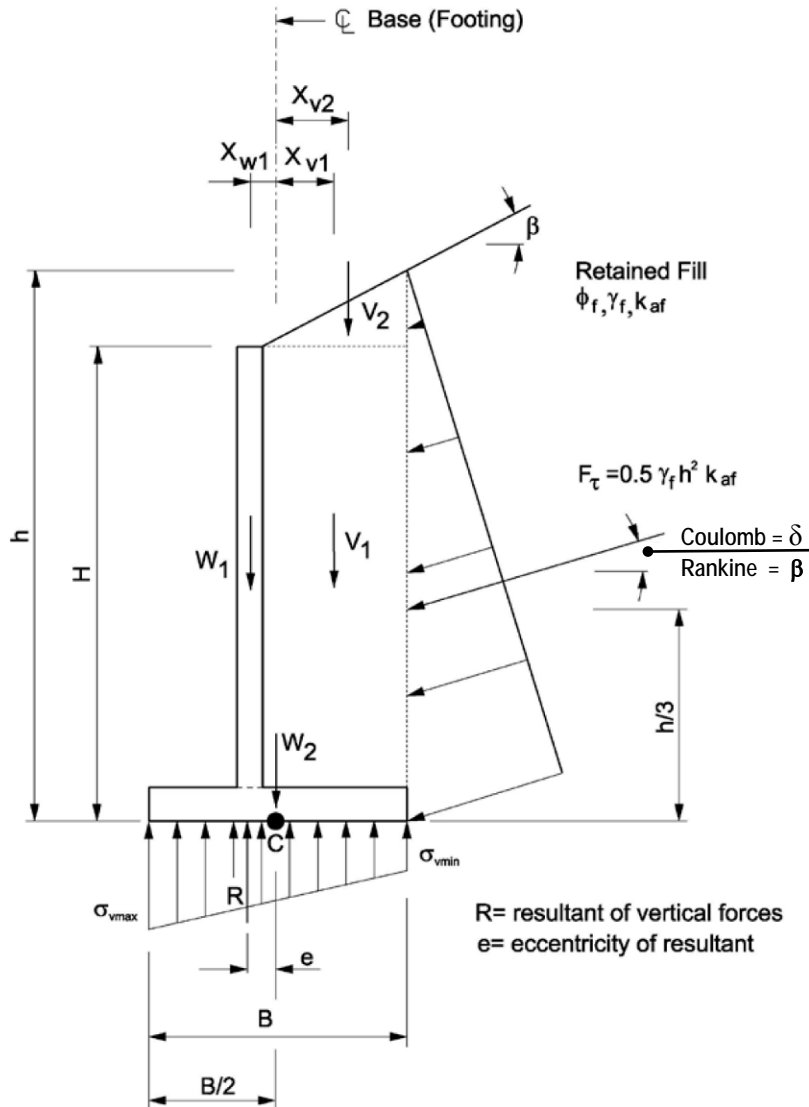


Summing Moments about Point C (Rankine):

$$e = \frac{(F_T \cos \beta)h/3 - (F_T \sin \beta)B/2 - V_1 X_{v1} - V_2 X_{v2} + W_1 X_{w1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin \beta}$$

Loading Diagram and Bearing Stress Criteria for CIP Cantilever Wall on Soil

Fig 6.5.5-1



If $e > B/6$, σ_{vmin} will drop to zero, and as "e" increases, the portion of the heel of the footing which has zero vertical stress increases.

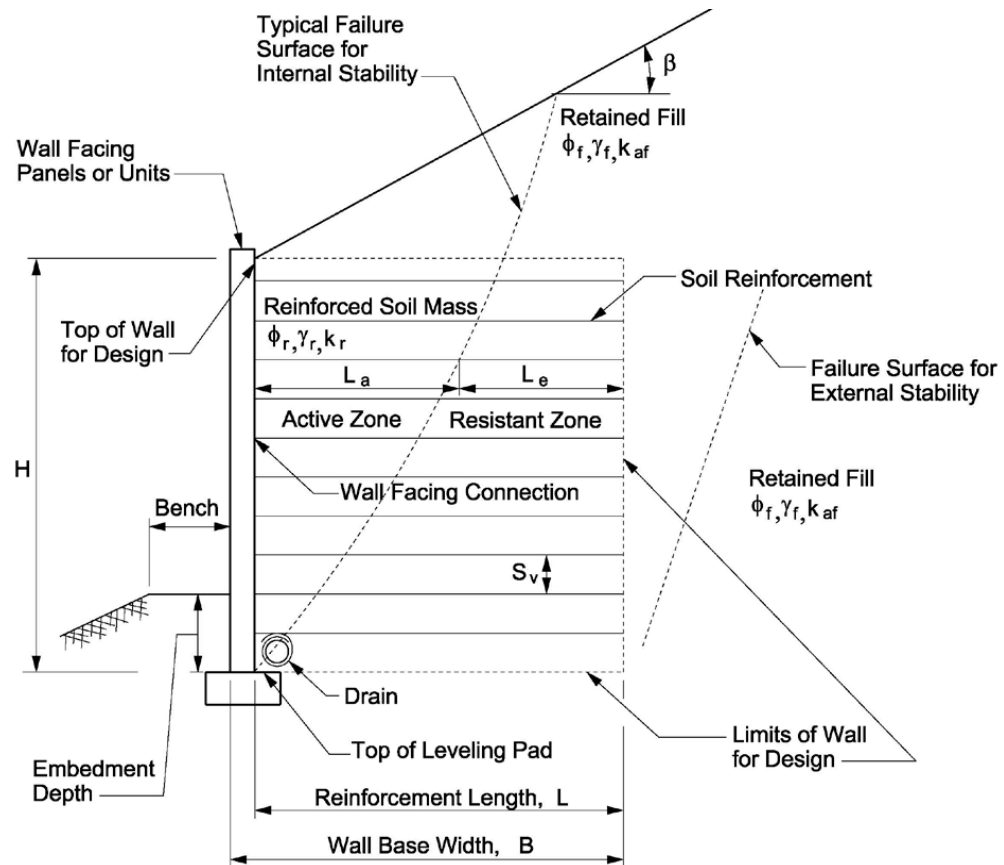
Summing Moments about Point C (Rankine):

$$e = \frac{(F_T \cos\beta)h/3 - (F_T \sin\beta)B/2 - V_1 X_{v1} - V_2 X_{v2} + W_1 X_{w1}}{V_1 + V_2 + W_1 + W_2 + F_T \sin\beta}$$

Loading Diagram and Bearing Stress Criteria for CIP Cantilever Wall on Rock

Fig 6.5.5-2

- MSE walls are proprietary so the wall vendor performs the internal stability design. The Bureaus of Bridge Design, Construction, and Materials and Research will evaluate the project wall criteria and prepare a special provision that lists the wall systems from the Pre-Approved List that are qualified specifically for use on the project along with the design loads and requirements. Once the contract is bid, the Contractor chooses one of the vendors listed on the special provision. The vendor then submits the final design of the wall system for review and approval for the specific project per Section 105.02 of the *NHDOT Standard Specifications* and the Special Provision, Item 592.1- Mechanically Stabilized Earth Retaining Wall.
- The criteria for acceptance for inclusion on the Department's Pre-Approved Proprietary Retaining Wall System List are noted in the NHDOT Proprietary Retaining Wall System Pre-Approval Process document. The Pre-Approved Proprietary Retaining Wall System List and their limitations ([Appendix 6.5-A2](#)) and the pre-approval process document are located on the Bridge Design Document Webpage at:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/documents.htm>
- Special Provision, Item 209.5 – Granular Backfill for MSE Walls notes the requirements for the high quality backfill material including the electrochemical requirements. This granular backfill material is specified for its durability, good drainage, constructability, and good soil reinforcement interaction. The reinforced soil, Granular Backfill for MSE Walls, shall assume a soil friction angle of 34 degrees and a soil unit weight of 125-pcf (2002-kg/m³).
- The design life of a permanent MSE structure based on corrosion shall be 125 years for all reinforcement located above the impervious membrane, and 100 years for all reinforcement located below the impervious membrane.
- The minimum reinforcement length (L) is approximately 70 percent of the MSE wall height H, as measured from the top of the leveling footing (See [Figure 6.5.6-1](#)). The reinforcement length shall be uniform throughout the entire height of the wall and satisfy all external and internal stability considerations. External loads, such as abutment footings or surcharges, will increase the minimum reinforcement length.
- The minimum embedment depth to the top of the leveling pad shall be 4.5-ft. See [Appendix 6.5-B2](#) for MSE retaining wall details.
- The facing element for a permanent MSE wall is an element of the vendor system. It is typically a wet precast concrete panel with an Ashlar Stone finish. See Chapter 2, Section 2.6 and [Appendix 2.6-A1](#) for information regarding aesthetics. All panel finishes shall be approved by the Design Chief. See [Appendix 6.5-A3](#) for pictures of retaining wall types and facing.



MSE Wall Element Dimensions for Design

Fig 6.5.6-1

B. Restrictions

MSE retaining walls shall not be used under the following conditions:

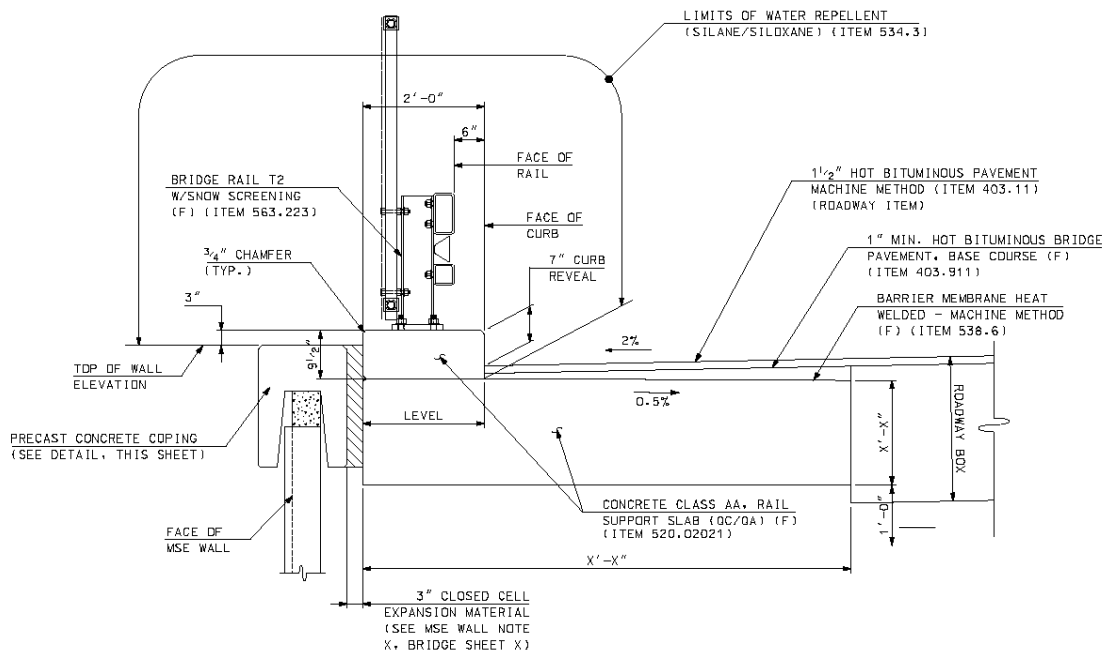
- Along shorelines where there is a scour or erosion potential that may undermine the reinforced fill zone, facing or the supporting footing.
- Reinforcements exposed to surface or ground water contaminated by acid mine drainage, industrial pollutants, or other aggressive environmental conditions.
- Where utilities, such as a sewer or water main, are constructed within the reinforced backfill zone since the utilities would not be accessible for replacement or maintenance and a breakage or rupture of utility line would have a detrimental effect on the stability of the wall.

C. Traffic Railing/Barrier at Top of MSE Wall

If steel bridge railing or concrete barrier is placed on top of a MSE retaining wall, the traffic impact loading must be transferred from the railing/barrier into a reinforced concrete moment

slab that is located just below the roadway pavement. The concrete slab is designed to keep vehicle impact forces from being transferred onto the MSE wall. The MSE retaining wall system shall be designed in accordance with *AASHTO LRFD 11.10.10.2*.

- Per NHDOT Bureau of Bridge Design, the equivalent static load of 10-kips (TL-4) shall be used for the elastic design for sizing the moment slab (analyze the stability of the moment slab: sliding and overturning analysis).
- Per NHDOT Bureau of Bridge Design, the dynamic load of 54-kips (TL-4) shall be used for the ultimate strength analysis for the structural capacity design (reinforcing design).
- In accordance with *NCHRP Report 663*, the 54-kips load level comes from measurements made on an instrumented barrier during impact and, therefore is a dynamic load used for structural capacity of the barrier, coping and moment slab. The 54-kip used as a static load results in an excessively conservative moment slab size.
(http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_663.pdf)
- The upper layers of soil reinforcement of the MSE retaining wall shall have sufficient tensile capacity to resist a concentrated horizontal load of γP_H where $P_H = 10$ -kips (44-kN) distributed over a barrier length of 5.0-ft. (1.5-m) per *AASHTO LRFD 11.10.10.2*
- See *NCHRP Report 663* for an example of designing a moment slab on a MSE wall located at: http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_rpt_663.pdf
- See Figure 6.5.6-2 and [Appendix 6.5-B2](#) for a typical moment slab detail.

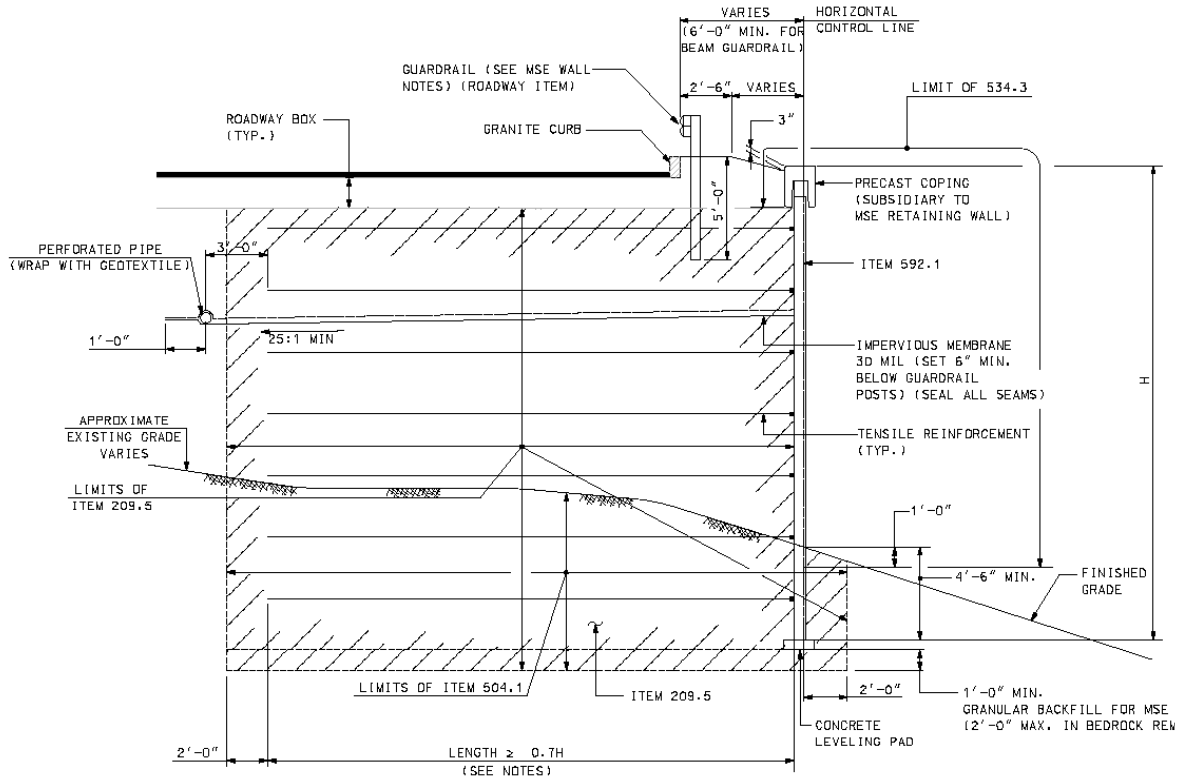


Typical Bridge Railing Moment Slab Detail

Fig 6.5.6-2

Beam guardrail posts located within the wall reinforcement zone of a MSE retaining wall shall have a minimum offset. The offset is 6-ft. (1.8-m) measured from the face of the wall to the face of the guardrail. This offset may be reduced if a stiffer barrier that has a lower deflection is used (e.g., precast single slope concrete barrier, modified thrie-beam).

- *Roadside Design Guide, 4th Ed., 2011* states the steel post beam guard rail has a dynamic deflection of 3.9-ft. (1.2-m) with a MASH TL-3 crash test. The single slope concrete barrier has a dynamic deflection of 6-in. (150-mm). The modified thrie-beam barrier has a dynamic deflection of 2.0-ft (0.6-m) for TL-3 NHCRP 350 crash test.
- The beam guardrail has a total thickness of 17-in. (432-mm) (includes w-beam, blockout and post). The MSE wall precast facing has a total thickness of 5.5-in. (140-mm). If a 6-ft. (1.8-m) offset is used, a distance of 4'-1 1/2" (1.26-m) remains between the back of the wall face and the back of the post for deflection.
- *AASHTO LRFD 11.10.10.2* states to use a 3-ft. (0.9-m) offset from the face of the wall if a flexible post and beam barrier is used. This dimension is to provide a distance to distribute the vehicular impact load onto the reinforcements without damage to the wall precast face, not for deflection of the barrier. The 6-ft. (1.8-m) offset will govern for beam guardrail.
- Guardrail posts shall be installed (minimum embedment depth of 5-ft. [1.5-m]) through the soil reinforcement in a manner that prevents ripping, damage and distortion of the soil reinforcement. In addition, the soil reinforcement shall be designed to account for the reduced cross-section resulting from the guardrail post holes.
- As noted on the plans, the Contractor shall take all necessary precautions when laying out the tensile reinforcement to prevent conflicts with beam guardrail posts, drainage structures, and impervious membrane.
- See Figure [6.5.6-3](#) and [Appendix 6.5-B2](#) for MSE wall details.



Typical MSE Retaining Wall Detail

Fig 6.5.6-3

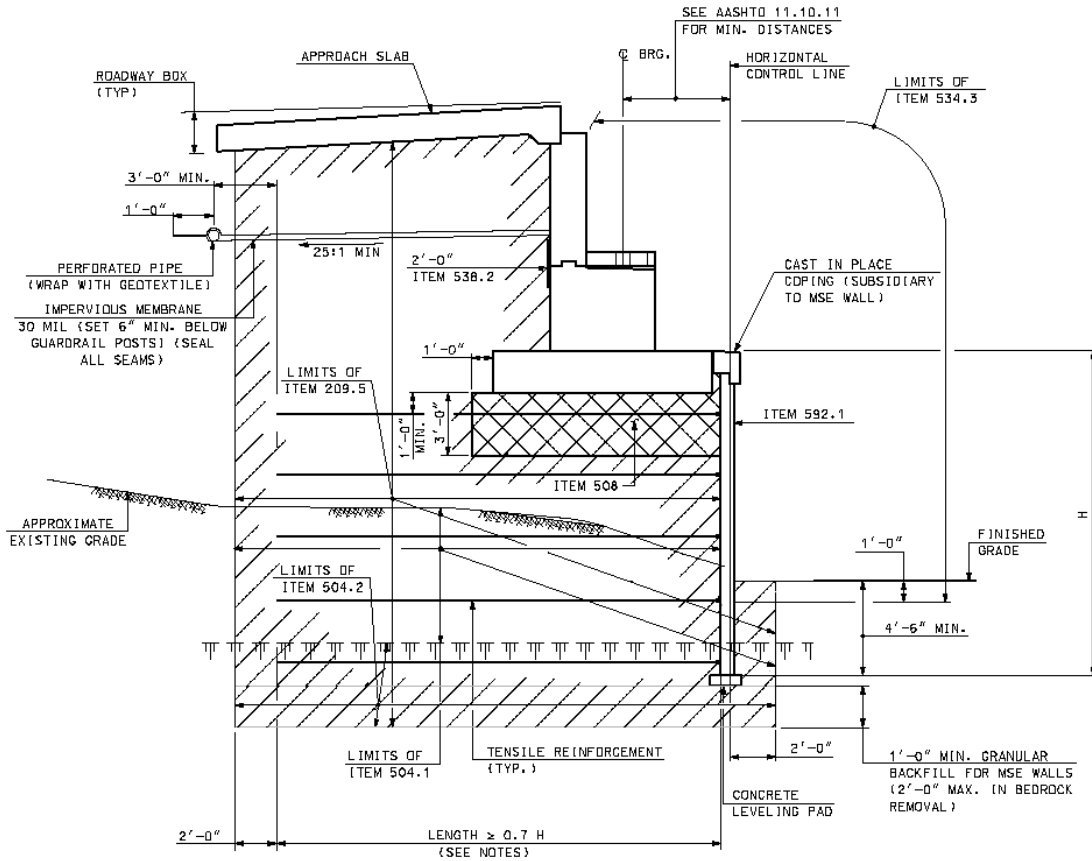
D. Abutment on Top of MSE Wall

Using MSE retaining walls to support bridge abutments can provide simplification in the design and construction of bridge abutments and faster construction of the bridge. It can also result in construction cost savings due to elimination of tall abutments.

The designer should consider the height of the MSE retaining wall as well as the sensitivity of the proposed structure to differential settlement or differential movement. MSE retaining walls are not rigid structures, and are designed to deflect slightly vertically and laterally under load. The risk of settlement and differential movement typically increases with the height of the wall.

- MSE retaining walls supporting abutments shall be designed in accordance with *AASHTO LRFD 11.10.11*.
- MSE walls directly supporting spread footing bridge abutments shall be 30-ft. (9-m) or less in total height, which includes the retained soil height up to the bottom of the embedded spread footing. This is due to the potential of larger vertical and lateral movements with taller MSE walls. However, the height restriction can be waived in areas with very competent bearing conditions, such as those with very dense soils or bedrock close to the surface.

- MSE retaining walls supporting bridge abutments shall be special designed wall systems. Only preapproved precast concrete panel faced MSE walls with inextensible tensile reinforcement may be used for permanent bridge installations. The preapproved systems are listed on the Pre-Approved Proprietary Retaining Wall System List, [Appendix 6.5-A2](#).
- Abutment spread footings, or the ends of the superstructure flat slab bearing directly on the surface of the MSE wall, should be designed for bearing service loads not to exceed 4-ksf (192-kPa) to limit the vertical movement to less than 0.5-in (13-mm) within the reinforced soil mass. For the strength limit state, factored bearing pressure shall not exceed a factored bearing resistance of 7-ksf (335-kPa) in accordance with *AASHTO LRFD C11.10.11*.
- The minimum distance from the centerline of the bearing on the abutment to the outer edge of the wall facing shall be 3.5-ft. (1.1-m).
- The minimum distance between the back face of the panel and the footing shall be 6.0-in. (152-mm).
- To provide bridge inspection access, provide a minimum of 3-ft. (0.9-m) from the bottom of the bridge superstructure to top of the MSE wall for I-girder type bridges, and 5-ft. 1.5-m) minimum for slab and box girder type bridges.
- The abutment footing shall be placed on a minimum 3-ft. (0.9-m) bed thickness of non-frost susceptible compacted coarse aggregate, Item 508, Structural Fill, clean stone.
- MSE walls supporting a bridge abutment shall not be used on water crossings unless approved by the Geotechnical Engineer.
- To prevent adverse stress concentrations at the reinforcement connections, the minimum vertical clearance between the bottom of the bridge support spread footing and the top level of reinforcement should be 1-ft. (0.3-m).
- The seismic design forces acting on the MSE wall should also include any seismic forces transferred from the bridge through bearing supports which do not slide freely (e.g., elastomeric bearings).
- See Figure [6.5.6-4](#) and [Appendix 6.5-B2](#) for MSE wall details.



Typical Abutment on a MSE Retaining Wall Detail

Fig 6.5.6-4

E. Contract Plans

A final design for the proprietary wall options is not included in the contract plans. Rather, the proprietary wall supplier will design and submit shop drawings after the contract is awarded. The contract plans for a proprietary retaining wall system shall include the following:

- Site plans showing survey layout, boring locations, contours, wall outline, drainage and utilities
- General wall plan showing wall lengths and expansion joints
- Elevation view showing top and bottom wall elevations and panel facing finish
- Final ground line in front of and behind the wall
- Typical cross-section
- Generic details for the desired appurtenances and drainage requirements
- Design loads, materials and specifications

- Stations and offsets relative to the survey centerline on the face of the wall for the beginning and ending points, working points, and all such offsets for turning-point locations where the wall forms an angle.
- MSE wall notes and quantities
Sample MSE wall notes are located on the NHDOT Bridge Design Website, Sample Plans, Sample Project Notes:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/sampleplans/index.htm>
- Pay quantities for the MSE wall are measured by the square foot and are determined by the height and length of the wall as shown on the contract plans.
- MSE wall backfill quantity is measured by the cubic yard and estimated by the approximate wall strip length shown on the plans that the designer has assumed plus 2-ft. (0.6-m) to pay limit line.
- Designers need to check for any conflict between the MSE wall soil reinforcing and sloping approach slabs off the bridge. If the MSE wall acts as a U-back wing, provisions need to be made to address any conflict with the soil reinforcement. A maximum soil reinforcement elevation could be shown on the plans or details provided to anchor the soil reinforcement to the side of the approach slab.
- If the top of wall profile is 3.5:1 or steeper, either the precast coping needs to be anchored to prevent sliding, or the coping shall be cast-in-place. The precast wall coping detail shall include a note to anchor the coping as needed to prevent sliding. For coping details, see NHDOT Bridge Details located at
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Beginning and ending locations should be checked to determine where the final grading elevations are equal both in front of and behind the wall, whereby the wall is no longer required.

A special design is required if the wall will be supporting structure foundations, sound walls, signs or sign bridges, luminaires, or other types of surcharge loads. Any structure above the wall shall be shown on the contract plans with any special loading noted.

6.5.7 Precast Concrete Modular Walls

Precast concrete modular walls are proprietary wall systems. Each proprietary system has its own unique method of locking the block units together to resist the horizontal shear forces that develop. NHDOT pre-approved concrete modular walls are prefabricated wet-cast concrete blocks that are stacked vertically or slightly battered, functioning as an externally stabilized gravity wall. The concrete modular walls are either solid concrete or have openings filled with granular material such as crushed stone or gravel. Concrete modular wall systems have a variety of face textures and colors for aesthetic appearances. The shape of the blocks usually allows the walls to be built along a curve, either concave or convex.

Loading and design criteria shall be in accordance with *AASHTO LRFD Sections 3 and 11*, and Chapters 4 and 6 of this manual.

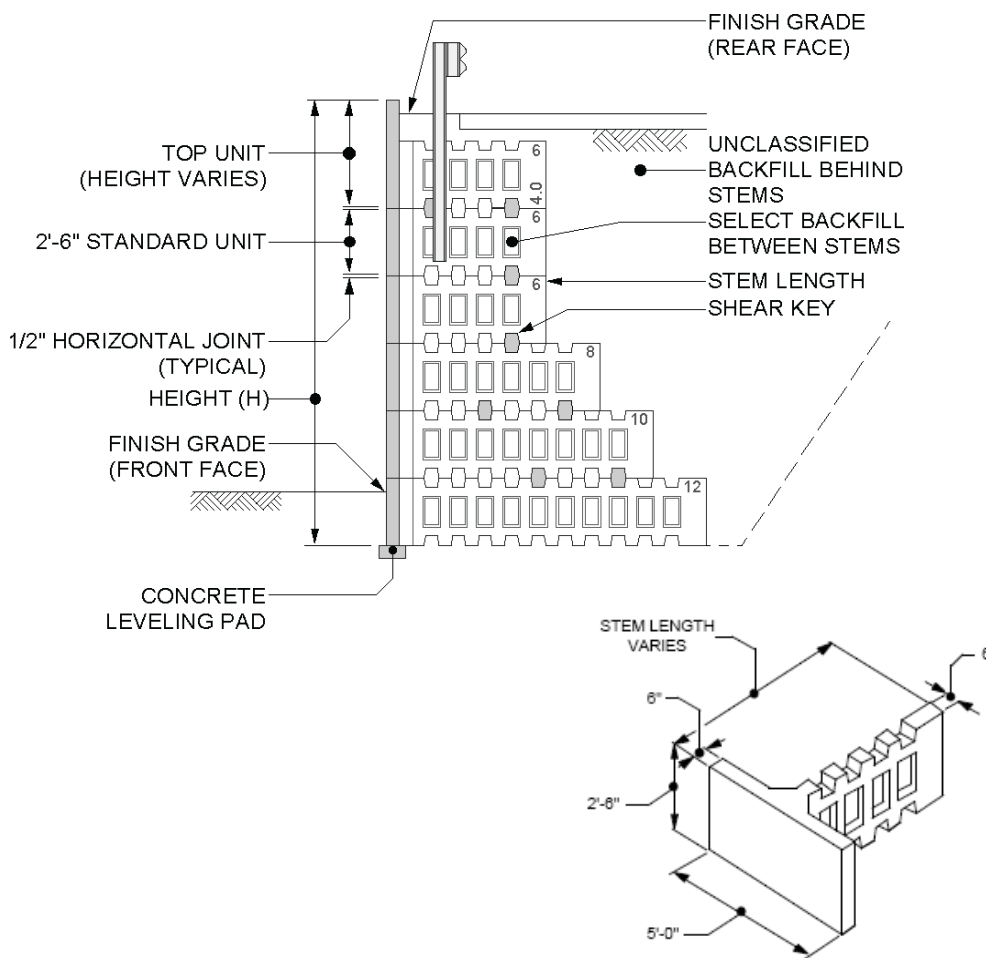
Precast concrete modular walls are proprietary so the wall vendor performs the internal stability design. The Bureaus of Bridge Design, Construction, and Materials and Research will evaluate the project wall criteria and prepare a special provision that lists the wall systems from the Pre-Approved List that are qualified specifically for use on the project along with the design loads and requirements. Once the contract is bid, the Contractor chooses one of the vendors listed on the

special provision. The vendor then submits the final design of the wall system for review and approval for the specific project per Section 105.02 of the NHDOT Standard Specification and the Special Provision, Item 592.31- Precast Concrete Modular Wall.

The criteria for acceptance for inclusion on the Department’s Pre-Approved Proprietary Retaining Wall System List are noted in the NHDOT Proprietary Retaining Wall System Pre-Approval Process document. The Pre-Approved Proprietary Retaining Wall System List, their limitations (Appendix 6.5-A2), and the pre-approval process document are located on the Bridge Design Document Webpage at:

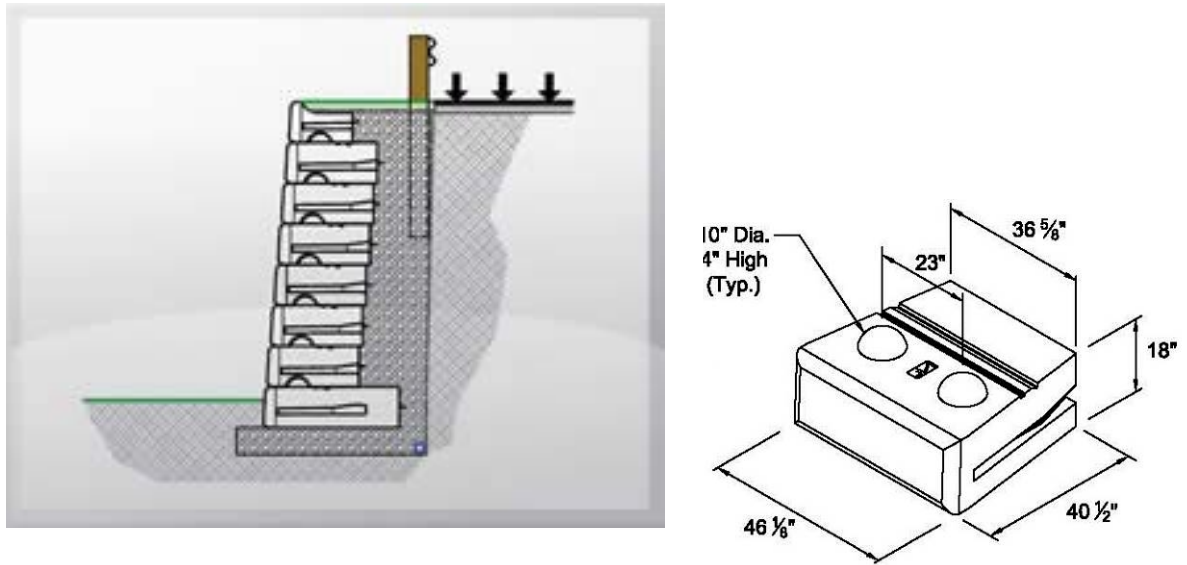
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/documents.htm>

See Figures 6.5.7-1, 2, & 3 for the pre-approved precast concrete modular wall systems and Appendix 6.5-A3 for pictures of the walls.



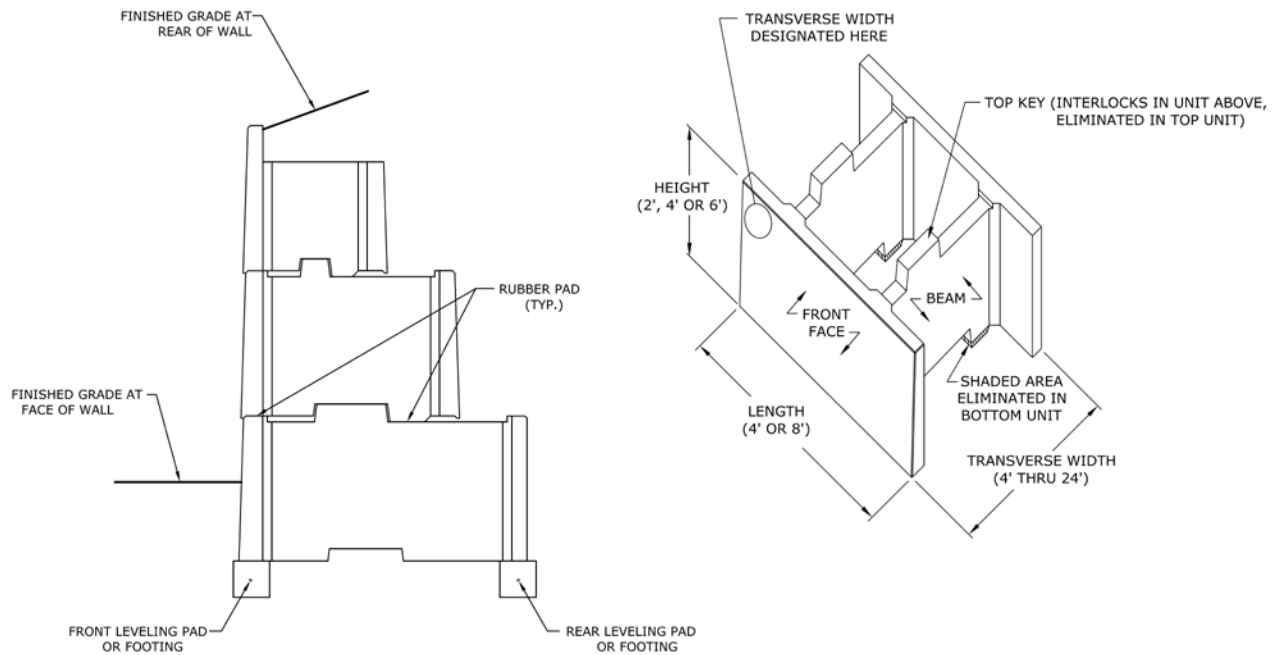
T-Wall Precast Concrete Modular Wall Detail

Fig 6.5.7-1



Redi-Rock Precast Concrete Modular Wall Detail

Fig 6.5.7-2



Double Wal II Precast Concrete Modular Wall Detail

Fig 6.5.7-3

6.5.8 Sheet Pile Walls

Sheet piling typically consists of interlocking steel sheets that are driven into the ground to form a continuous sheet pile wall. Sheets can also be constructed of vinyl, aluminum, concrete, or wood; however, steel sheet piling is most commonly used due to its ability to withstand driving stresses and be removed and reused for other walls. Sheet piling is typically installed by driving with a vibratory pile-driving hammer. For sheet piling in cut applications, the piling is installed first, and then the soil in front of the wall is excavated or dredged to the design elevation. There are two general types of sheet pile walls: cantilever, and anchored/braced. Both types of sheet pile walls are commonly used by NHDOT for both temporary and permanent conditions.

Sheet piling is a common temporary shoring system (cofferdam) in cut applications. The following are some of the advantages and disadvantages of sheet piling as a temporary shoring system:

Advantages:

- Provides a cutoff for groundwater flow
- Can be installed without lowering the groundwater table
- Can be used for irregularly-shaped excavations
- Can be easily removed
- Can be anchored or braced for deep excavations or with large surcharge loading

Disadvantages:

- Installation by vibrating or driving (thus, in areas where vibration-sensitive structures or soils are present, sheet piling may not be appropriate)
- Installation of the sheets is difficult in very dense or stony soils

Temporary sheet piling (cofferdam) is designed by the Contractor and submitted to the Department for Documentation.

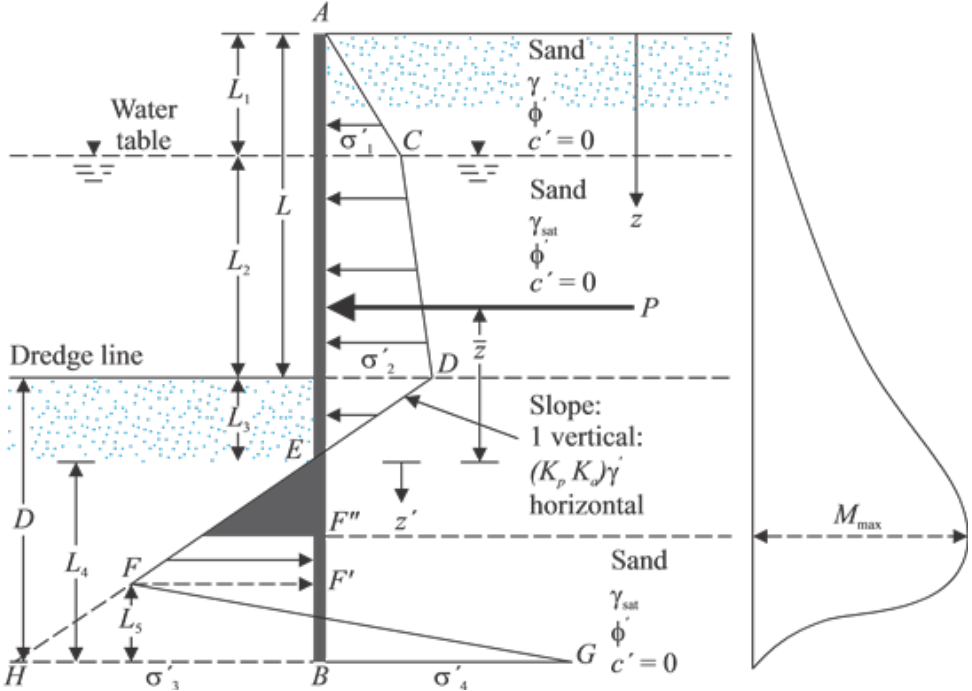
A. Cantilever Sheet Pile Wall Design

- Sheet piles resist lateral earth pressure through both the passive resistance in front of the wall and the flexural resistance of the sheet pile.
- Most cantilever sheet pile walls can be designed to an exposed height of approximately 15-ft. (4.6-m). Over 15-ft. (4.6-m) exposed height, the walls may require tie-backs or deadman-type anchors.
- The sheet piling shall be designed to resist lateral stresses due to soil and groundwater, both for temporary (i.e., due to dewatering) and permanent ground water levels, as well as any temporary and permanent surcharges located above the wall. If there is potential for a difference in ground water head between the back and front of the wall, the embedment depth of the wall, and the amount of dewatering behind the wall, shall be established to prevent piping and boiling of the soil in front of the wall.
- The steel section used shall be designed for the anticipated corrosion loss during the design life of the wall. The ground anchors for temporary walls do not need special corrosion protection if the wall design life is three years or less, though the anchor bar or steel strand section shall be designed for the anticipated corrosion loss that could occur during the wall design life. Permanent easements are required if the ground anchors, extend outside the right of way/property boundary.
- Sheet piling should not be used in dense soil or soil with cobbles or boulders. It also should not be used in soils or near adjacent structures that are sensitive to vibration.

- Sheet pile retaining walls (temporary and permanent) shall be designed in accordance with *AASHTO LRFD Sections 3 and 11*, and Chapters 4 and 6 of this manual.
- The designer shall check that the standard shapes meeting the required properties are readily available from domestic suppliers.
- Z-shape sheet piles are generally recommended because they offer the most efficiency.
- NZ sheet piling shall be specified for permanent installations. Other sheet pile sections may be used with the approval of the Design Chief. The NZ section is an economical, domestically produced sheet pile section and has largely replaced the PZ section in common use. PZ sheet piling is less economical, but is still commonly used as temporary shoring; because of its durability, it can be removed and reused on multiple projects.
- Soil parameters and recommendations will be provided by the Geotechnical Engineer.
- All loads must be taken into consideration, including earth pressure, water pressure, equipment operating adjacent to the wall, and highway and railroad loads. Since sheet pile walls can trap water, a head imbalance may need to be taken into account due to effects of waves, tidal action, storm surges or heavy rainfall. Where applicable, weepholes may be cut in the sheet piles to help relieve the head imbalance.
- The sheet pile wall design procedure is as follows:
 - 1) Determine the required depth of sheet pile embedment (D) (*AASHTO LRFD 11.8.4 and C11.8.4.1*) with the appropriate load factors (*AASHTO LRFD Table 3.4.1-1 and 3.4.1-2*) and resistance factors (*AASHTO LRFD 11.6.2.3*). Use a load factor of 1.1 for determining embedment depth and a load factor of 1.50 for flexural design.
 - 2) Determine the maximum bending moments and stresses in the sheet pile section (*AASHTO LRFD 11.8.5*).
 - 3) Design anchorage systems if required.
- See Figure [6.5.8-1](#) for an earth pressure diagram of a cantilever sheet pile wall.

B. Cantilever Sheet Pile Wall Details

- For aesthetics, precast panels or cast-in-place concrete facing is typically installed on permanent steel sheet pile walls. Many different surface finishes can be obtained using form liners as shown in Chapter 2, Appendix 2.6-A1.
- Where an aesthetic concrete facing is not required or is not feasible, the steel piles can be coated with a durable paint system.
- Designers should specify the proposed sheet pile section, required section modulus, steel grade and embedment depths on the plans along with the geotechnical and construction notes.
- See [Appendix 6.5-B2](#) for cantilever sheet pile wall details and [Appendix 6.5-A3](#) for pictures of sheet pile facing.



**Cantilever Sheet Pile Wall Earth Pressure Diagram
(Granular Soils)**

Fig 6.5.8-1

6.6 Piers

6.6.1 General

Piers provide a load path between the superstructure and the foundation. Piers shall be designed to resist vertical and horizontal loads from the superstructure and loads acting directly on it (e.g., wind loads, ice loads, water pressure, and vehicle impact). The entire bridge system (abutments and piers) needs to be analyzed to determine the magnitude of the superstructure loads applied to the pier, taking into account the bearing types and relative stiffness of all the piers.

The initial bridge span length is determined in the TS&L plan stage. Piers normally are spaced to meet the geometric and aesthetic requirements of the site and to give maximum economy for the total structure.

6.6.2 Pier Types and Considerations

The type of pier used is dependent on the location and bridge geometry. When determining the pier layout, cost savings can be gained by keeping the cap size and column size the same for all piers to enable the reuse of forms.

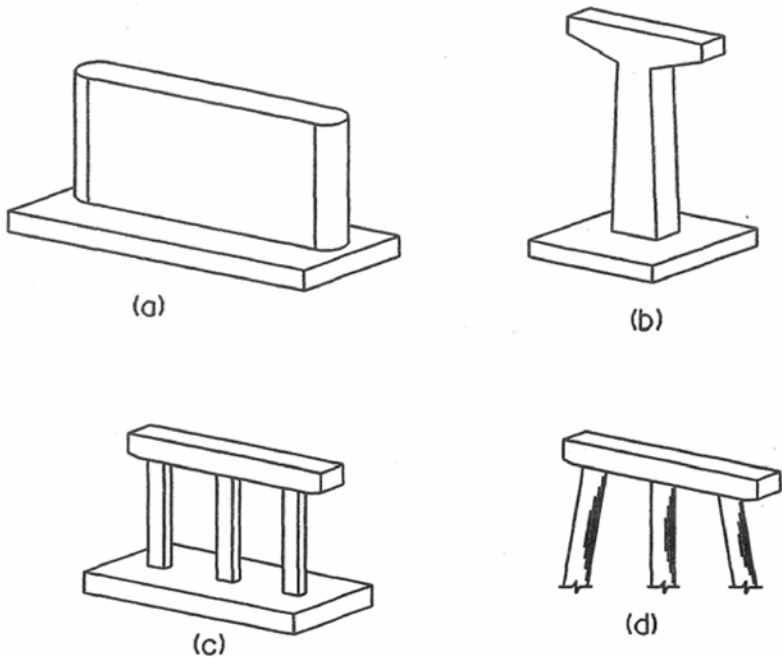
To maintain an aesthetically-pleasing appearance, long spans may be justified for tall piers. Difficult and expensive foundation conditions will also justify longer spans. Span lengths may change in the design stage if substantial structural improvement and/or cost savings can be realized. The designer should discuss the possibilities of span lengths or skew with the supervisor as soon as possible.

Pier column spacing should minimize column dead load moments. Multiple columns are better suited for handling lateral loads due to wind and/or earthquake. The designer may alter column size or spacing for structural reasons or change from a single-column pier to a multicolumn pier.

See Figure [6.6.2-1](#) and [Appendix 6.6-A1](#) for typical NHDOT pier types.

In selecting a pier type and configuration, the following factors shall be considered:

- Subsurface conditions
- Span lengths
- Feature crossed
- Environmental conditions
- Bridge width
- Bearing type
- Skew
- Unsupported pier height
- Vertical and horizontal clearances
- Floodplain and scour sensitive regions (if applicable)
- Economy
- Roadway, railway or waterway conditions
- Aesthetics
- Seismic response



Pier Types

(a) Wall Pier (b) Hammerhead (c) Multi-column (d) Pile Bent

Fig 6.6.2-1

A. Wall Pier

Wall piers are primarily used for river or stream crossings; divided highways with narrow medians; and short, wide bridges where short columns would create high stress due to shrinkage.

Wall piers are a solid mass of reinforced concrete that can be designed to resist forces from floating ice, debris, and vehicle, vessel, or rail collision.

B. Hammerhead Pier

Hammerhead piers have a single large column with a cap overhanging each side of the column supporting the superstructure. For tall piers, a hammerhead pier becomes more economical than a wall pier due to the reduced amount of materials. Like a wall pier, a hammerhead pier can be designed to resist forces in a stream, as well as vehicle, vessel, or rail collision.

C. Multi-Column Pier

A multi-column pier consists of three or more reinforced concrete columns with a continuous pier cap. Non-redundant pier columns shall not be used. Two-column piers vulnerable to collision damage may cause catastrophic collapse if one of the columns is destroyed. The columns can be either rectangular or circular. This pier type is typically used for grade separation structures. Do not use in streams where debris may lodge between columns.

D. Pile Bent Pier

Pile bent piers consists of multiple piles driven into the ground in a straight line, with a continuous reinforced concrete cap. The piling can be H-steel piling or round piling filled with concrete. Pile bents can also be constructed with a drilled shafts/column system. Steel-cap pile piers should be avoided since this is a fracture critical design that historically has shown weld and steel member cracking. This type of pier is inexpensive since it has no foundation or columns to form and cast.

Pile bent piers can be located in tidal rivers, environmentally sensitive areas, scourable areas, and grade-separations; however, consideration must be given to corrosion protection.

E. Aesthetics

Rustication grooves, form panels, and decorative finishes may visually improve the appearance of piers. The architectural treatment used on the pier shall match the treatment used on the abutments. Designers shall consider the effect of architectural treatment on clearances and position of reinforcement. Approval by the Design Chef is required on all pier aesthetics, including finishes, materials, and configuration. See Chapter 2, Section 2.6 and Appendix 2.6-A1 for additional information regarding aesthetics.

6.6.3 Loads and Load Application/Design and Analysis

Piers shall be designed in accordance with both the *AASHTO LRFD Specifications* and as noted in Chapter 4. Loads and load factors shall be determined in accordance with *AASHTO LRFD Chapter 3* and as outlined in Chapter 4 of this manual.

Determining pier design forces requires an overall understanding of the behavior of the bridge structure (e.g., load paths and the assumptions associated with the different pier types, bearings and superstructure layout). The designer shall determine all the loads acting on the superstructure and substructure and then apply the applicable combination of loads to the pier at the appropriate limit states.

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

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

For pier design, transverse forces applied above the bridge seat will create overturning moments that can be transmitted through upward and downward forces on bearings across the pier cap. Longitudinal forces applied above the bearings usually do not transmit moments through bearings so those forces are transmitted directly through the bearing to the bridge seat.

The following factors shall be considered when applying forces to the piers:

- Location of pier
- Symmetry of spans
- Variation in pier heights and cross sections
- Skew of piers

- Foundation fixity
- Type of bearing (fixed or expansion)

A. Bearing Type

A friction-acting bearing, such as a steel sliding expansion bearing, cannot transmit more longitudinal force than the friction force that causes the bearing to move. Any amount of force above the friction force must therefore be distributed to fixed bearings in the bridge.

A fixed bearing is intended to fully transmit all longitudinal and transverse forces from the superstructure to the pier.

An anchored (vulcanized) elastomeric expansion bearing will transmit to the pier the force required to produce the maximum amount of shear deflection for which the bearing is designed.

B. Pier Skew

If a pier has a skew, the designer will need to take the longitudinal and transverse forces for each applicable horizontal superstructure load (BR, CE, WS superstructure, WL, TU, and FR) and compute parallel and perpendicular components that shall be applied to the pier. In addition, if the pier is not aligned with stream flow, the parallel and perpendicular components for WA and IC will need to be computed.

C. Pier Elements

If pier caps are supported on multiple columns, the columns shall be spaced to balance the dead load moments in the cap. The pier cap ends shall be perpendicular to the centerline of the cap to provide uniform development lengths of the reinforcing.

Caps can be designed as continuous beams which are pin supported at the columns.

The overhangs of hammerhead piers may need to be investigated for the bracket and corbel effect.

AASHTO LRFD Equation 5.7.4.2-3 provides the minimum area of longitudinal reinforcement for column piers. For wall piers, the equation computes much more reinforcing than what is needed for flexure. NHDOT Bridge Design policy is to determine the minimum area of longitudinal reinforcement for wall piers as follows:

- 1) Determine the amount of concrete required for the vertical load:
 $(A_s / (f_y / f'_c)) = A_g$
- 2) Use this reduced A_g in *AASHTO LRFD Equation 5.7.4.2-3*.

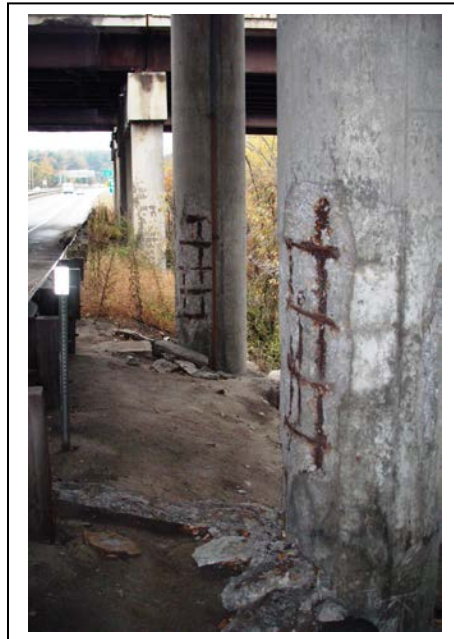
D. Scour

Piers located in a river shall include provisions to adequately protect them from undermining caused by scouring of the riverbed. Methods to account for scour to be considered in the design are: 1) increasing the depth for spread footing or use of a foundation seal, 2) designing the deep foundation elements for an unsupported length [the exposed pile length should be the vertical distance from the bottom of the footing (foundation seal if present) to the calculated scour depth], 3) placing adequately sized riprap, and 4) driving sheeting left in place. See Chapter 2, Section 2.7.7 for additional scour information.

6.6.4 Details for Piers

The following applies to the detailing of piers:

- The minimum width of any pier component shall be 3.5-ft. (1.1-m) to allow access for workers during construction for cage inspection and vibration of concrete, unless approved otherwise by the Design Chief.
- For multi-column piers, do not design a phase to be supported on less than two (2) columns.
- Wall piers shall be tapered 1H:12V both ends. The sides shall be tapered 1H:48V when the pier is in the waterway or ice path.
- Wall piers shall be oriented parallel to the direction of water flow with steel nose armor (\angle 8x8x3/4) installed on the upstream end.
- Corrosion inhibitor and epoxy coated reinforcing steel shall be used for all pier components that are located in the “splash zone”. The passing traffic and/or snow plows splash salt-laden snow against the pier, which eventually causes corrosion of the reinforcing steel and spalling of the concrete. See Figure 6.6.4-1.



Pier Splash Zone

Fig 6.6.4-1

- Item 536.11, Epoxy Coating for Concrete shall only be used on the top of piers which have an expansion joint above. Bridge Maintenance feels the product works well on piers and adds additional protection.
- The pier top dimensions shall be sufficient to support the bridge bearings with adequate cover to the anchor rods (e.g.; anchor bolts inside the top reinforcing hoop bars). Skew angle, intermediate expansion joints, minimum support length, and keeper plates for seismic restraint need to be considered in dimensioning the pier cap/top.
- Cantilevered pier caps may have the bottom surface of the cantilever sloped upward from the column toward the end of the cap.

- For pile bent piers, the piles may be exposed steel H-piles, encased steel H-piles, concrete-filled steel pipes, or concrete columns combined with drilled shafts.
- Wall piers may have a horizontal tie spacing up to 48-in. (1220-mm) in locations not specifically required to have a closer spacing (*AASHTO LRFD Fig. C5.10.6.3-1*). Vertical tie spacing shall be per *AASHTO LRFD 5.10.6.3*.
- Reinforcement in pier caps shall be detailed so as not to interfere with bearing anchor rods.
- Multiple reinforcement layers should be considered to alleviate congestion.
- Use 90-degree hook with an 8-in. (203-mm) radius to anchor the dowel bars in the footing/column connection. Show the lap splice length for bent dowels and check development length of hooked end of dowel bar at footing/column interface.
- See [Appendix 6.6-B1](#) for pier details.

A. Pedestals:

- To allow for drainage between the pedestals, provide wash sections sloped with a minimum 2-in. (50-mm) wash from the center to the outside on both sides of the pier.
- The top of the pier shall be stepped between the pedestals to provide a constant pedestal height. Do not detail the top of the pier level with varying pedestal heights. The pedestal concrete has been breaking off and a higher pedestal would increase this problem. See [Appendix 6.6-B1](#) for details.
- The width of the pedestal shall be equal to the top width of the pier. The pedestal shall provide adequate room for the bearings and meet the seismic criteria of Chapter 5 of this manual. Increase pedestal size as required for anchor rod embedment, skewed beams, future jacking, and seismic design requirements.
- The minimum beam seat height is 2-in. (50-mm). Since the tops of bearing seats are usually subjected to very large localized pressures under the bearings, reinforcement directly under the bearings shall always be provided to prevent the formation of visible cracks or possible spalling of concrete. See [Figure 6.6.4-2](#) and [3](#).
- The bearing seats shall be wide enough to satisfy the requirements of *AASHTO LRFD 4.7.4.4*.

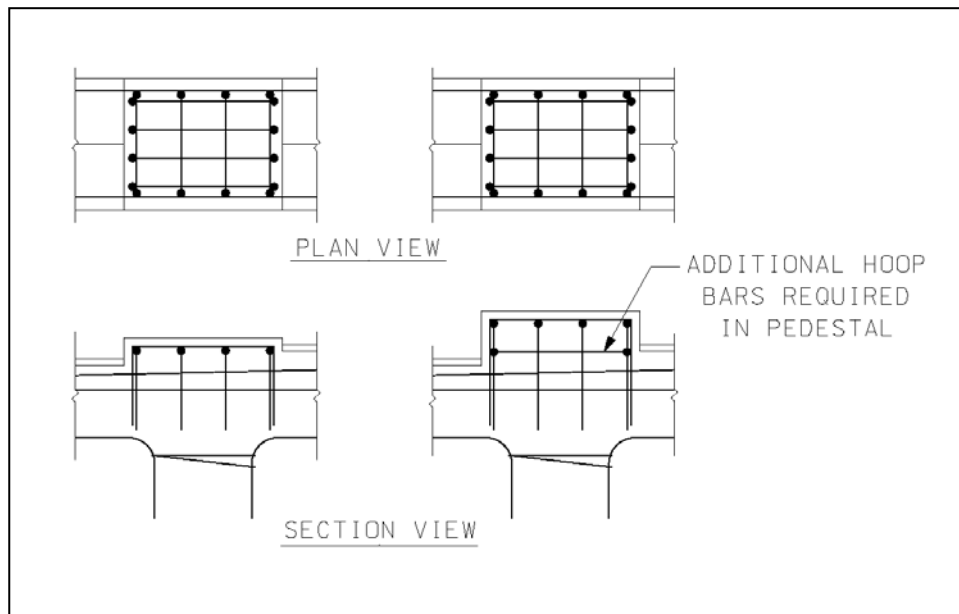
B. Anchor Rods:

- All anchor rods for bridge shoes shall be located *inside* the rebar cage.
- Drilling holes for anchor rods will not be permitted in concrete pier caps. Anchor rods shall be installed before the concrete is placed by setting accurately with a template.
- Place reinforcement so as not to interfere with anchor rod locations.



Pedestal Failure

Fig 6.6.4-2



Pedestal Reinforcement

Fig 6.6.4-3

6.6.5 Vehicular Collision Pier Protection

AASHTO LRFD 3.6.5 contains provisions for vehicular collision forces (CT) on structures that cross over roadways that routinely carry trucks and have design speeds of 50-mph (80-kph) or higher. If a bridge pier, meeting *AASHTO LRFD 3.6.5* provisions and located within 30-ft. (9-m) of the edge of a roadway (NHDOT Bridge Design defines “edge of roadway” to be “edge of travel lane”), does not have sufficient strength to resist the vehicular collision force, nor meets the criteria for exemption as stated in Chapter 4, Section 4.3.14, Vehicular Collision Force, the bridge pier shall be protected in accordance with *AASHTO LRFD* Section 2.3.2.2.1 and by one of the following:

- A permanent single slope 54-in. (1372-mm) tall concrete barrier designed for TL-5 loading, when the distance from the back of the barrier to the face of the pier is less than or equal to 10-ft. (3-m).
- A permanent single slope 42-in. (1067-mm) tall concrete barrier designed for TL-5 loading, when the distance from the back of the barrier to the face of the pier is greater than 10-ft. (3-m). (NHDOT barrier height is 45-in. [1143-mm])
- An embankment.

A. Single Slope Concrete Barrier 54-in. (1372-mm) Tall

1) Double-Faced Barrier

- If there is enough room for the single slope double-faced barrier, the road side toe of the barrier shall be placed a minimum of 3-ft. (0.9-m) from the face of the pier.
 - See the Bridge Design Detail Sheet, Pier Protection Type I and Type II (54-in. Single Slope Concrete Barrier) and [Appendix 6.6-B2](#).
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/detailsheets/index.htm>.

2) Single-Faced Barrier

- If there is not enough room for a double-faced barrier section, then a single-faced barrier may be used. The pier side face of the single-faced barrier shall be placed a minimum of 9-in. (229-mm) from the pier.
 - See Bridge Detail Sheet, Pier Protection Type I and Type II (54-in. Single Slope Concrete Barrier, Single-Faced) and [Appendix 6.6-B2](#).
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/detailsheets/index.htm>

B. Single Slope Concrete Barrier 45-in. (1143-mm) Tall

- The pier side toe of the 45-in. (1143-mm) tall single slope double-faced barrier, the pier side toe of the barrier shall be placed a minimum of 10-ft. (3-m) from the substructure and embedded 3-in. (76 -mm).
- See Highway Standard Plan, Single Slope Barrier at:
<http://www.nh.gov/dot/org/projectdevelopment/highwaydesign/standardplans/index.htm>

The 45-in. (1143-mm) and the 54-in. (1372-mm) single slope concrete barrier details are in compliance with requirements per updated NCHRP Report 350 for test No. 5-11 (MASH TL-5). Documentation is included in the FHWA Acceptance Letter B64 (http://safety.fhwa.dot.gov/roadway_dept/policy_guide/road_hardware/barriers/pdf/b64.pdf) and NCHRP 157 Vol. 1 (http://onlinepubs.trb.org/onlinepubs/nchrp/nchrp_w157.pdf).

The intended purpose of these barriers is to shield a pier from traffic, primarily large trucks and tractor trailers, so as to reduce the separate but related potential for damage to the pier and collapse of the bridge that might result from a truck collision with a pier.

- For new structures, consider planned widenings or future realignments of lower roadways when establishing limits of setback distances, clear zones, or horizontal clearance limits.
- When considering the pier protection options, the designer shall include aesthetics, maintenance, and cost as they apply to the bridge pier.
- The bridge designer shall work with the roadway design engineer to determine if a median barrier is provided as part of the highway design and the method of transition from the pier protection barrier to the highway median barrier.
- Consider overall safety at a given location, including vehicle and pedestrian traffic, when selecting the appropriate type of pier protection to be used. Consider the effect that tall barriers might have on sight distances, particularly near intersections, and the end treatments that will be required for these taller barriers.
- Select the pier protection barrier terminal treatment for design speeds greater than or equal to 50-mph (80-kph) from the following options:
 - Terminate outside the clear zone of any approaching traffic.
 - Terminate within a shielded location.
 - Terminate by the use of a crash cushion system; or,
 - Terminate in conjunction with a suitably designed transition to another barrier.
- In accordance with the *Roadside Design Guide, 4th Ed., 2011, Section 5.5.2*, the pier protection barrier shall be extended 10-ft. (3-m) in advance of the pier. Beyond that point, the barrier height shall be vertically transitioned on a 10:1 slope to the height of the adjoining barrier. The barrier shall continue with a flare-rate in accordance with the *Roadside Design Guide* for the length of need required, or transition to another barrier.
- The above noted offsets of the barrier to the pier take into account the working widths and zone of intrusion (ZOI) width.
 - The working width is the lateral distance from the front face/toe of barrier to the greatest of vehicle extent, barrier deflection, or barrier width.
 - The zone of intrusion refers to the maximum distance a vehicle extends behind the top front face of the barrier.
 - The *Roadside Design Guide, 4th Ed., 2011, Section 5.5.2*, shows the zone of intrusion for TL-4 barriers. (See Figure [6.6.5-1](#))
 - The zone of intrusion for TL-5 loading and a 54-in. (1372-mm) single-slope barrier is 18-in. (457-mm) for a truck cab and 45-in. (1143-mm) for a cargo box, as presented in a paper prepared by Stephen F. Hobbs of McElhanney Engineering Services Ltd. for the 2010 Annual Conference of the Transportation Association of Canada. (See Table [6.6.5-1](#)).
 - The 54-in. (1372-mm) single-faced single slope concrete barrier is placed a minimum of 9-in. (229-mm) from the back of the barrier to the face of the pier. The top of the barrier is 9 ¾-in. wide (248-mm). This provides a setback of 18 ¾-in. (476-mm), which is greater than the zone of intrusion for a truck cab, TL-5 loading.

- The NHDOT pier protection placed at the minimum offsets to the pier does not provide the required intrusion zone for a truck box. However, an impact from a cargo box would not cause major damage to the pier nor the loss of a span. Also, the majority of barrier impacts are from pick-up trucks or smaller vehicles.

Table 1 Zone of Intrusion with Concrete Barrier

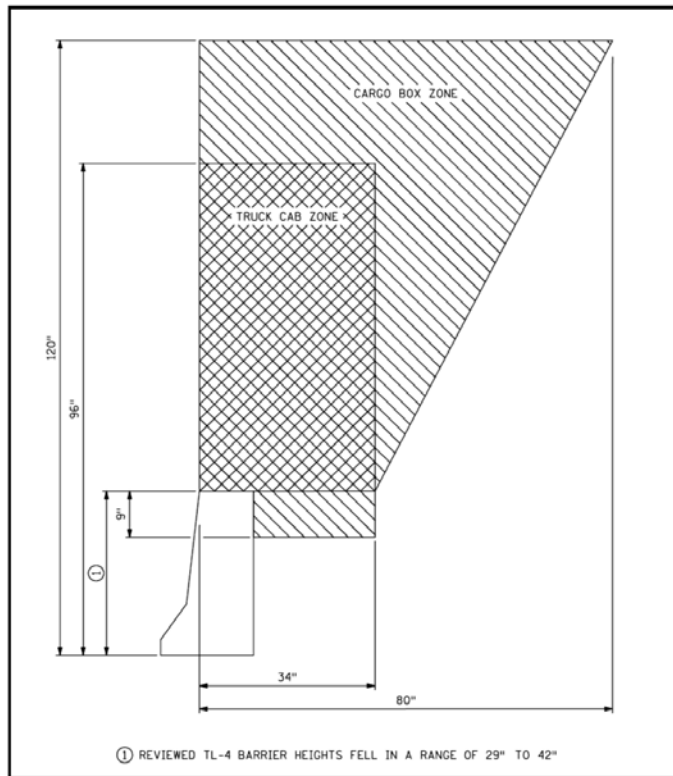
Test Level	Height of Concrete Barrier (mm)	Cab Width of Intrusion (mm), (i)	Truck Box Width of Intrusion (mm), (i)
TL-1	508	550	no box
TL-1	810	178 (1)	no box
TL-2	508	711 (1)	no box
TL-2	690	457 (1)	no box
TL-2	810	305 (1)	no box
TL-2	1070	178 (1)	no box
TL-3	690	762 (1)	no box
TL-3	810	610 (1)(7)	no box
TL-3	1070	254 (1) (interpolated)	no box
TL-3	1420	100 (interpolated)	no box
TL-4	810	864 (1)(7)	2032 (1)(7)
TL-4	1070	610 (1)	610 (1)
TL-4	1420	230 (interpolated)	457 (interpolated)
TL-5	810	TL-5 not achievable with 810 high barrier	
TL-5	1070	1220 (1)	2200 (1)
TL-5	1370	457 (1)	1140 (1)
TL-6	2286	0 (1)	100 (1)

Note:

- (i) Intrusion widths shown are for safety shaped (F-shaped or New Jersey) or constant sloped (California or Texas) concrete barrier. There is an inverse correlation between barrier height and intrusion width. For a specific result for another type of barrier and crash test level, refer to the appendix of reference (1). Alternately review video or high speed photos of the specific device crash tests to measure or estimate widths needed.
- (1) "Guidelines for Attachments to Bridge Rails and Median Barriers", Keller, Sicking, Fallor, Polivka & Rohde, February 26, 2003
- (7) Charles Boyd, Florida Department of Transportation, Attachments to Traffic Railings, Session 23, FICE/FDOT Design Conference 2006, Designing for More Than Bridges and Roads.

Zone of Intrusion with Concrete Barrier
(Zone of Intrusion and Concrete Barrier Measure
by Stephen F. Hobbs)⁸

Table 6.6.5-1



Intrusion Zones for TL-4 Barriers
(Roadside Design Guide, 4th Ed., 2011)

Figure 6.6.5-1

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6.7 Approach Slabs

6.7.1 General

Bridge approaches typically experience two types of settlement, global and local. Global settlement is consolidation of the deeper natural foundation soils. Local settlement is mainly compression of fill materials directly beneath the approach pavement due to construction. The combination of global and local settlements adjacent to the bridge end causes the characteristic “bump” in the pavement at the bridge. The approach slab significantly reduces local settlement and provides a transition to the long-term roadway differential settlements.

Bridge approach slabs are required for all new and widened bridges unless directed otherwise by the Design Chief (e.g., short span bridges don't necessarily require approach slabs).

6.7.2 Design Criteria

The standard bridge approach slab design is based on the following criteria:

- Designed as a slab in accordance with *AASHTO LRFD 4.6.2.3 (slab type bridges)*.
- The effective span length (unsupported length) is assumed to be 2/3 the length of the slab.
- Bottom reinforcing cover = 3-in. (75-mm)
- Concrete compressive strength, $f'_c = 4,000$ psi (2.8-kg/mm²)
- Reinforcing steel yield strength, $f_y = 60$ ksi (42-kg/mm²)
- The approach slab shall extend far enough from the abutment to intersect the failure slope for backfill material (60° from the bottom of the back of footing). Otherwise, the effects of live load surcharge shall be included in the soil pressure diagram of the abutment.
- Approach slab lengths shall be 10-ft. (3-m), 15-ft. (4.6-m), 20-ft. (6-m), 25-ft. (7.6-m) or 30-ft. (9-m) measured perpendicular to the abutment.
- Longitudinal reinforcement shall run parallel with the centerline of the roadway; transverse reinforcement shall run and measured parallel with the centerline of abutment.

6.7.3 Details for Approach Slabs

- There is no top mat of reinforcing steel. The approach slab concrete is fiber reinforced (Item 544.7, Synthetic Fiber Reinforcement). The bottom reinforcing mat shall be epoxy coated. Choose bottom reinforcing size and spacing from table on [Appendix 6.7-B1](#).
- Approach slabs are typically 15-in. (380-mm) thick and shall be sloped to the percent grade as noted in table on [Appendix 6.7-B1](#).
- No haunch is required for approach slabs at expansion ends when the expansion joint is located in front of the backwall.
- A 2-ft. (0.6-m) deep haunch is required for approach slabs at fixed ends and when the expansion joint is located behind the backwall.
- Approach slabs shall extend to within 6-in. (150-mm) of the approach curb or u-back wingwall. The opening between the approach slab and curb shall be filled with the same concrete mix as the approach slab to a 6-in. (150-mm) depth.

- Any drainage structures shall be located on the plan and the approach slab modified to accommodate the structures.
- As noted under the Approach Slab Notes on the plans, 1-ft. (0.3-m) x 2-ft. (0.6-m) blocks shall be formed on approach slab seats to support approach curbs, paid for under Item 520.0302 Concrete Class AA, Approach Slabs (QC/QA). If the bridge has u-back wings, a seat for the approach curb shall be formed at the end of the wing.
- If the bridge has phased construction, the approach slab will also have phased construction, which shall be clearly delineated on the plans.
- If the bridge has phased construction and a skew greater than 20°, the mechanical connectors will not be able to splice to the transverse reinforcement since the mechanical connectors are set perpendicular to the centerline of the roadway and the transverse reinforcement is parallel with the centerline of bearing. The designer shall specify on the approach slab plans one of the following options:
 - 1) The transverse bars can be placed normal to the centerline of construction and slab phase construction joint. This creates different length bars for the triangular area of the approach slab; or
 - 2) Place the transverse bars parallel with the centerline of bearing and tie the mechanical connectors to the longitudinal bars.
- Designers need to check for any conflict between the MSE wall reinforcing and approach slabs. If the MSE wall acts as a U-back wing, provisions need to be made to address any conflict with the tensile reinforcement. In this situation, either show a maximum tensile reinforcement elevation on the plans or provide a way for the soil reinforcement to anchor to the side of the approach slab.
- For approach slabs at the expansion end with the expansion joint located behind the backwall, a 3-in. (75-mm) diameter split PVC, SCH 40 drain pipe shall be placed on the approach slab seat at 6-ft. (1.8-m) on center prior to placement of the approach slab. See expansion joint details in Chapter 7, Section 4.
- See [Appendix 6.7-B1](#) for approach slab details and tables.

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Bridge Design Manual

Chapter 6 – Appendix A

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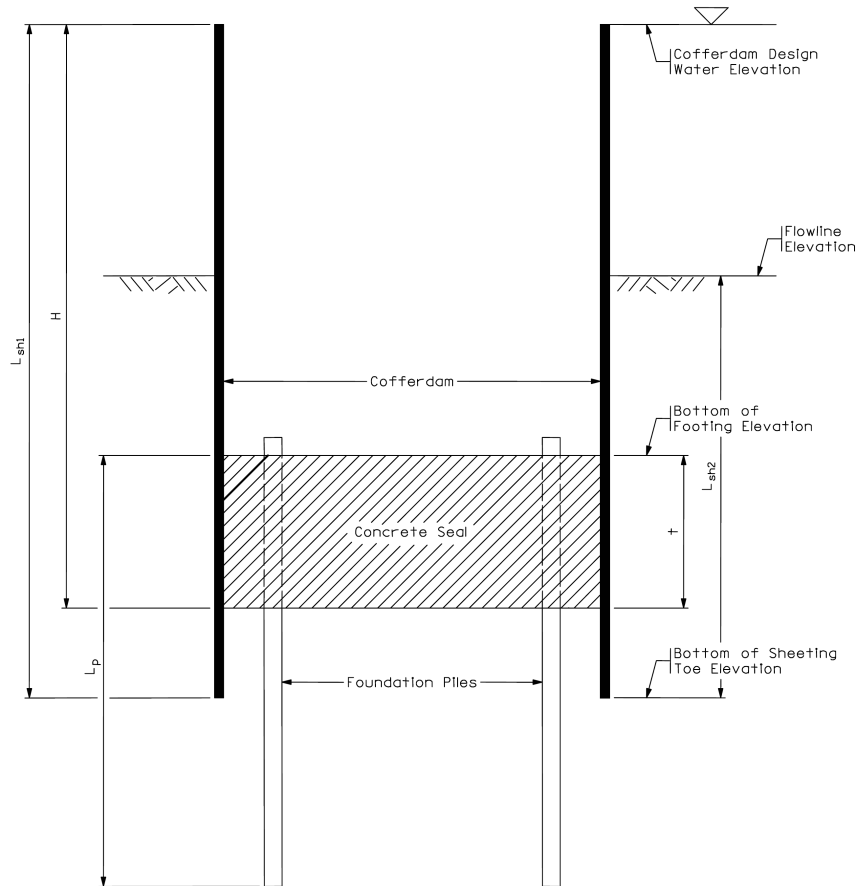
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TREMIE SEAL DESIGN EXAMPLE

Reference:

Minnesota Department of Transportation, *LRFD Bridge Design Manual 10.4.2*,

Retrieved from <http://www.dot.state.mn.us/bridge/lrfd.html>



- 1) Determine preliminary dimensions:

A rule of thumb for preliminary seal thickness is $0.25H$ for pile footings. The minimum allowed seal thickness is 4.0-ft. In plan, the minimum length and width of the seal is 2.0-ft larger than the footing on all sides, but it must also be large enough to avoid interference between sheet piling and battered piles.

- 2) Determine hydrostatic buoyancy force, P_b , due to hydrostatic pressure developed at the bottom of the seal:

$$P_b = H \cdot A \cdot \gamma_w$$

where H = hydrostatic head, ft.

A = plan area of cofferdam minus area of piles, ft^2

γ_w = unit weight of water, 0.0624 kips/ft^3

3) Determine resistance due to seal weight, R_{sc} :

$$R_{sc} = A \cdot t \cdot \gamma_c$$

where t = thickness of seal, ft.

γ_c = unit weight of concrete, 0.145 kips/ft³

4) Determine sheet pile resistance, R_{sh} . This will be the smaller of:

- sheet pile weight, P_{sh} + soil friction on sheet pile, $P_{sh \text{ soil}}$ *or* bond between sheet piling and seal, $P_{sh \text{ seal}}$

$$P_{sh} = L_{sh1} \cdot p_{cof} \cdot \omega_{sh}$$

where L_{sh1} = length of sheet piling in feet, normally based on sheet piling embedment of approximately H/3

p_{cof} = nominal perimeter of cofferdam, ft.

ω_{sh} = weight per square foot of sheet piling, normally assume 0.022 kips/ft²

$$P_{sh \text{ soil}} = L_{sh2} \cdot p_{cof} \cdot f_{sh \text{ soil}}$$

where L_{sh2} = length of sheet piling below flowline in feet, normally based on sheet piling embedment of approximately H/3 (choose conservative value for flowline elevation to account for scour or reduce by 5.0-ft.)

$f_{sh \text{ soil}}$ = friction of sheet piling with soil, normally assume 0.150 kips/ft²

$$P_{sh \text{ seal}} = (t - 2) \cdot p_{cof} \cdot f_{sh \text{ seal}}$$

where $f_{sh \text{ seal}}$ = bond of sheet piling to soil, normally assume 1.0 kips/ft²

5) Determine foundation piling resistance R_{pile} . This will be the smaller of:

- foundation pile weight, P_p + piling pullout resistance, $P_{p \text{ pull}}$ *or* the bond between foundation piling and seal $P_{pile \text{ seal}}$

The piling pullout resistance, $P_{p \text{ pull}}$ is the smaller of:

- soil friction on all individual piles $P_{pile \text{ soil}}$ *or*
- soil friction on pile group, P_{grp} + weight of soil in pile group, P_{soil}

$$P_p = N \cdot [\omega_p \cdot L_p - (H + L_p - t) \cdot \gamma_w \cdot A_p]$$

where N = number of piles

ω_p = non-buoyant weight per foot of an unfilled pile, kips/ft

L_p = estimated pile length

A_p = end bearing area of pile, ft²

$$P_{pile \text{ soil}} = N \cdot A_{p \text{ surf}} \cdot f_{pile \text{ soil}} \cdot (L_p - t)$$

where $A_{p \text{ surf}}$ = surface area of pile per unit length, ft²
(for H-piles, take $A_{p \text{ surf}} = 2 \cdot (\text{depth} + \text{width})$)

$f_{pile \text{ soil}}$ = friction between piles and soil, normally assume 0.150 kips/ft.

$$P_{\text{grp}} = (L_p - t) \cdot f_{\text{pile soil}} \cdot p_{\text{grp}}$$

where p_{grp} = perimeter of pile group, ft.

$$P_{\text{soil}} = (L_p - t) \cdot A_s \cdot \gamma_{\text{sb}}$$

where A_s = area of soil engaged by pile group, which is the group perimeter area defined by the outside piles minus the area of the piles, ft² (use perimeter at top of pile group)

γ_{sb} = buoyant unit weight of soil, 0.040 kips/ft³

$$P_{\text{pile seal}} = t \cdot N \cdot A_{\text{p surf}} \cdot f_{\text{pile seal}}$$

where $A_{\text{p surf}}$ = surface area of pile per unit length, ft²
(for H-piles, take $A_{\text{p surf}} = 2 \cdot \text{depth} \cdot \text{width}$)

$f_{\text{pile seal}}$ = friction between piles and seal, normally assume 1.0 kips/ft²

- 6) Determine factor of safety, FS, and revise design as needed. The minimum required factor of safety is 1.1:

$$FS = (R_{\text{sc}} + R_{\text{sh}} + R_{\text{pile}}) / P_b$$

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Selection of Wall System in a Fill-Section							
Wall Type	Perm.	Temp.	Max Height (ft)	Required ROW ¹	Lateral Movements	Advantages	Disadvantages
Concrete Gravity	X		25	0.5 - 0.7H ³	1/500	<ul style="list-style-type: none"> • durable • less select backfill as compared to MSE walls • concrete can meet aesthetic requirements 	<ul style="list-style-type: none"> • deep foundation support may be necessary • relatively long construction time
Concrete Cantilever	X		35	0.4 - 0.7H ³	1/500	<ul style="list-style-type: none"> • durable • less select backfill as compared to MSE walls • concrete can meet aesthetic requirements 	<ul style="list-style-type: none"> • deep foundation support may be necessary • relatively long construction time
Concrete Counterfort	X		60	0.4 - 0.7H ³	1/500	<ul style="list-style-type: none"> • durable • less select backfill as compared to MSE walls • concrete can meet aesthetic requirements 	<ul style="list-style-type: none"> • deep foundation support may be necessary • relatively long construction time
Concrete Crib	X		40	0.5 - 0.7H	1/300	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment • rapid construction 	<ul style="list-style-type: none"> • difficult to make height adjustments in field
Metal Bin	X		40	0.5 - 0.7H	1/300	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment • rapid construction 	<ul style="list-style-type: none"> • difficult to make height adjustments in field • subject to corrosion in aggressive environment
Gabion	X		25	0.5 - 0.7H	1/50	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment 	<ul style="list-style-type: none"> • need adequate source of stone • construction of wall requires significant labor • susceptible to damage from plows or vandalism
MSE (precast facing)	X		65	0.7 - 1.0H	1/100	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment • flexibility in choice of facing 	<ul style="list-style-type: none"> • requires use of select backfill • subject to corrosion in aggressive environment
MSE (modular block facing)	X		50	0.7 - 1.0H	1/200	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment • flexibility in choice of facing • blocks are easily handled 	<ul style="list-style-type: none"> • requires use of select backfill • potential for freeze-thaw issues • positive reinforcement connection to block is difficult to achieve
MSE (geotextile/geogrid/welded wire facing)	X	X	50	0.7 - 1.0H	1/60	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment • flexibility in choice of facing • speed of construction 	<ul style="list-style-type: none"> • facing may not be aesthetically pleasing • geosynthetic reinforcement is subject to degradation in some environments
Reinforced Soil Slope	X	X	100	0.5 - 1.0H	1/60	<ul style="list-style-type: none"> • does not require skilled labor or specialized equipment • flexibility in choice of facing • vegetation provides ultraviolet light protection to geosynthetic reinforcement 	<ul style="list-style-type: none"> • facing may not be aesthetically pleasing • geosynthetic reinforcement is subject to degradation in some environments • vegetated soil face requires significant maintenance

1. ROW requirements expressed as distance (as a fraction of wall height) behind wall face where fill placement is generally req'd for flat backfill conditions, except where noted.
 2. Ratio of the difference in vertical settlement between two points along the wall to the horizontal distance points.
 3. ROW requirements given is the typical wall base width as a fraction of wall height.

Selection of Wall System in a Cut-Section									
Wall Type	Perm.	Temp.	Max Height (ft)	Required ROW ¹	Lateral Movements	Water Tightness	Advantages	Disadvantages	
Sheet Piles	X	X	15	none	large	fair	<ul style="list-style-type: none"> rapid construction readily available 	<ul style="list-style-type: none"> difficult to construct in hard ground or through obstructions 	
Soldier Piles/ Lagging	X	X	15	none	medium	poor	<ul style="list-style-type: none"> rapid construction soldier piles can be drilled or driven 	<ul style="list-style-type: none"> difficult to maintain vertical tolerance in hard ground potential for ground loss at excavated face 	
Slurry (diaphragm)	X	X	80 ²	none ⁴	small	good	<ul style="list-style-type: none"> can be constructed in all soil types or on weathered rock watertight wide range of wall stiffness 	<ul style="list-style-type: none"> requires specialty contractor significant spoil for disposal requires specialized equipment 	
Tangent Pile	X	X	30 80 ²	none ⁴	small	fair	<ul style="list-style-type: none"> adaptable to irregular layout can control wall stiffness 	<ul style="list-style-type: none"> difficult to maintain vertical tolerance in hard ground significant spoil for disposal requires specialized equipment 	
Secant Pile	X	X	30 80 ²	none ⁴	small	fair	<ul style="list-style-type: none"> adaptable to irregular layout can control wall stiffness 	<ul style="list-style-type: none"> significant spoil for disposal requires specialized equipment 	
Soil Mixed	X	X	80 ²	none ⁴	small	fair	<ul style="list-style-type: none"> adaptable to irregular layout 	<ul style="list-style-type: none"> requires specialized equipment relatively small bending capacity 	
Anchored	X	X	65 ³	0.6H+anchor bond length	small-medium	N/A	<ul style="list-style-type: none"> can resist large horizontal pressure 	<ul style="list-style-type: none"> requires skilled labor and specialized equipment anchors may require permanent easements 	
Soil-nailed	X	X	40	0.6H - 1.0H	small-medium	N/A	<ul style="list-style-type: none"> rapid construction adaptable to irregular wall 	<ul style="list-style-type: none"> nails may require permanent easements difficult to construct and design below water 	
Micropile	X		N/A	varies	N/A	N/A	<ul style="list-style-type: none"> does not require excavation 	<ul style="list-style-type: none"> requires specialty contractor 	

1. ROW requirements expressed as distance (as a fraction of wall height) behind wall face where fill placement is generally req'd for flat backfill conditions, except where noted.

2. Max height given for wall with anchors

3. For soldier piles and lagging walls only

4. ROW required if wall includes anchors

<p style="text-align: center;">NHDOT Pre-Approved Proprietary Wall Systems* (Revised 6/26/2015)</p>					
<p style="text-align: center;">Wet Cast MSE Retaining Walls</p>					
<p>Wall Company/ Technology</p>	<p>Company System Name</p>	<p>System Description</p>	<p>Qualified Applications</p>	<p>Limitations/Concerns</p>	
<p>The Reinforced Earth Company</p>	<p>Reinforced/ Retained Earth Wall</p>	<p>Precast concrete 5'x5' facing panels with steel strip tensile reinforcement</p>	<ul style="list-style-type: none"> • Retaining walls • Wingwalls • Abutments 		
<p>Tensor</p>	<p>Ares Wall</p>	<p>Precast concrete 5'x5' or 5'x9' facing panels with Tensor HDPE geogrid tensile reinforcement</p>	<ul style="list-style-type: none"> • Retaining walls only 	<ul style="list-style-type: none"> • Retaining wall heights < 30-ft. • All designs need to address the long-term stability of the HDPE geogrid in regards to creep and elongation. 	
<p>Big R Bridge Corp.</p>	<p>Vist-A-Wall MSE Wall</p>	<p>Precast concrete 5'x5' facing panels with welded wire mesh tensile reinforcement</p>	<ul style="list-style-type: none"> • Retaining walls • Wingwalls • Abutments 		

*** All systems are subject to additional review and approval by NHDOT for each specific project application.**

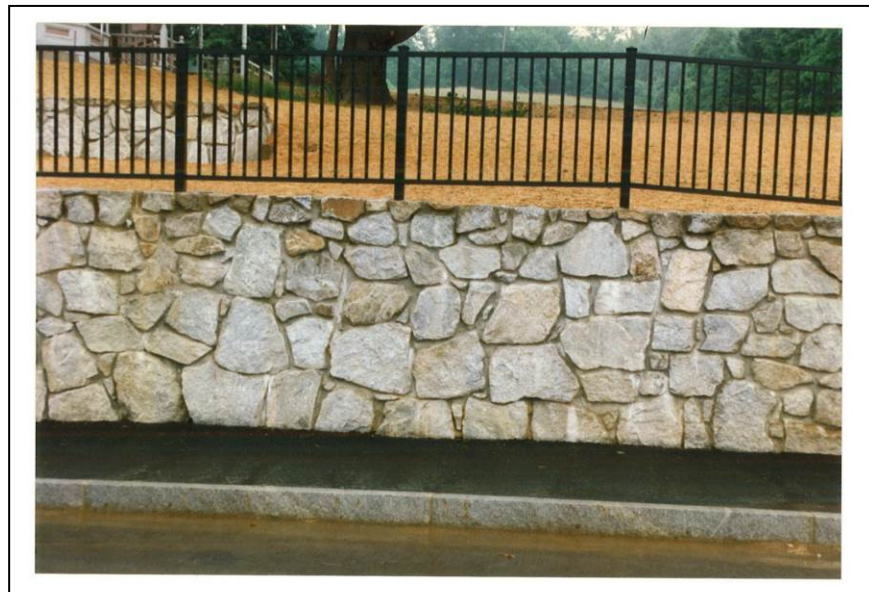
<p style="text-align: center;">NHDOT Pre-Approved Proprietary Wall Systems* (Revised 3/3/2015)</p>					
<p style="text-align: center;">Precast Concrete Modular Walls</p>					
<p>Wall Company/ Technology</p>	<p>Company System Name</p>	<p>System Description</p>	<p>Qualified Applications</p>	<p>Limitations/Concerns</p>	
<p>The Neel Company</p>	<p>T-Wall</p>	<p>T-shaped precast concrete module with soil infill and 2.5'x5' facing panel</p>	<ul style="list-style-type: none"> Retaining walls only 	<ul style="list-style-type: none"> Retaining wall heights < 40-ft. 	
<p>Double Wal</p>	<p>Double Wal II</p>	<p>Four sided precast concrete module with soil infill and 4'x8' facing panel</p>	<ul style="list-style-type: none"> Retaining walls only 	<ul style="list-style-type: none"> Retaining wall heights < 30-ft. 	
<p>Redi-Rock</p>	<p>Gravity wall</p>	<p>Full width, large precast concrete blocks that act as a gravity wall</p>	<ul style="list-style-type: none"> Retaining walls only 	<ul style="list-style-type: none"> Use for wall heights up to 9-ft. Blocks are 46" wide x 18" high, with typical depths of 28", 42" and 60" 	

**All systems are subject to additional review and approval by NHDOT for each specific project application.*

Permanent Retaining Walls



BARTLETT
(Stonewall)



EPSOM
(Stonewall)

Permanent Retaining Walls



GILSUM

(Concrete Cantilever Retaining Wall with Stone Facing)



DURHAM

(Concrete Cantilever Retaining Wall with Stone Facing)

Permanent Retaining Walls – MSE Walls



Permanent Retaining Walls – MSE Walls



Permanent Retaining Walls



ALBANY
(Soil Nail Wall with Timber Facing)

Permanent Retaining Walls



BARTLETT
(Sheet Pile Wall with Stone Facing)



I-93, EXIT 2
(T- Wall)

Temporary Retaining Walls



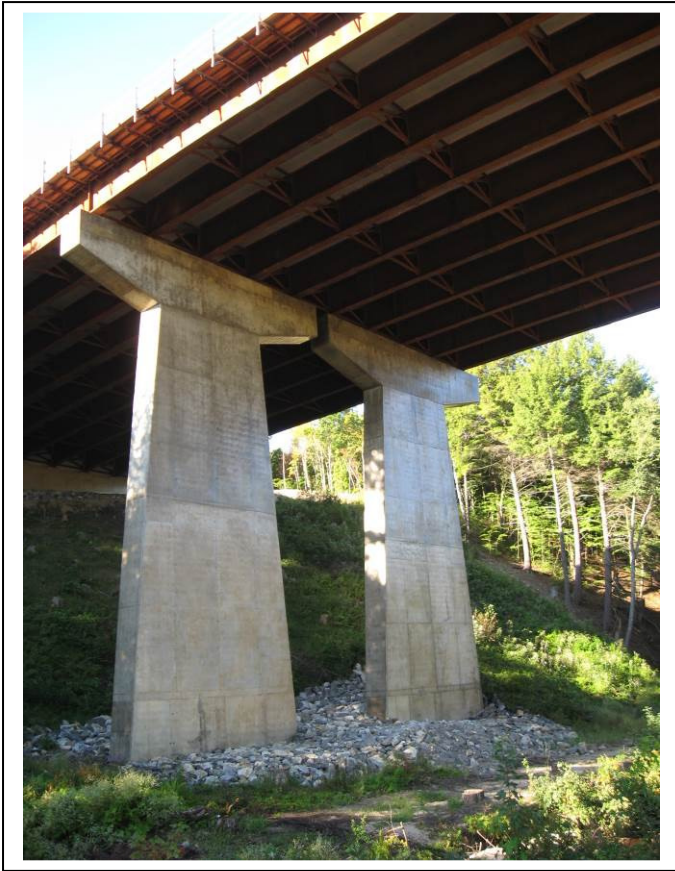
(Soldier Pile with
Timber Lagging)



(Steel Sheetting -
Cofferdams)

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PIER TYPES



Hammerhead Pier



PIER TYPES



(Phase 1 construction, three-column pier when completed)

Multi-column Pier



PIER TYPES



Wall Pier



PIER TYPES



Wall Pier



PIER TYPES



Pile Bent Pier



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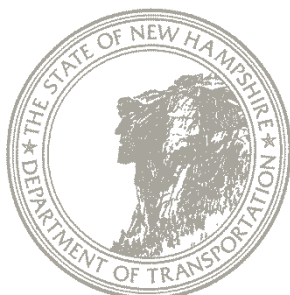


Bridge Design Manual

Chapter 6 – Appendix B

January 2015 – v 2.0

(Revised August 2019)



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SPREAD FOOTING DETAILS

The spread footing details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Spread Footing

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ABUTMENT DETAILS

The abutment details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Abutment Details

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INTEGRAL & SEMI-INTEGRAL ABUTMENT DETAILS

The integral abutment details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Integral & Semi-integral abutment details

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WALL JOINTS

The wall joint details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Wall Joints

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WINGWALL DETAILS

The wingwall details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Wingwall Details

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RETAINING WALL DETAILS

The retaining wall details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Retaining Wall Details

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PIER DETAILS

The pier details can be found at NHDOT Bridge Design Bridge Details web page:

<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Pier Details

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PIER PROTECTION

The pier protection details can be found at NHDOT Bridge Design Detail Sheets web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/detail sheets/index.htm>

Scroll down to: Vehicular Collision Pier Protection

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APPROACH SLAB DETAILS

The approach slab details can be found at NHDOT Bridge Design Bridge Details web page:
<http://www.nh.gov/dot/org/projectdevelopment/bridgedesign/bridgedetails/index.htm>

Scroll down to: Substructure/Approach Slab Details

APPROACH SLAB REINFORCING			
APPROACH SLAB LENGTH *	APPROACH SLAB THICKNESS	SKEW	BOTTOM REINFORCING (spaced perpendicular to CL construction)
10-ft.	15-in.	0° - 45°	#7 sp. @ 12-in.
15-ft.	15-in.	0° - 45°	#8 sp. @ 12-in.
20-ft.	15-in.	0° - 25°	#6 sp. @ 6-in.
		26° - 45°	#7 sp. @ 6-in.
25-ft.	15-in.	0° - 25°	#7 sp. @ 6-in.
		26° - 45°	#8 sp. @ 6-in.
30-ft.	15-in.	0° - 25°	#8 sp. @ 6-in.
		26° - 45°	#9 sp. @ 6-in.

*Measured perpendicular to abutment.

Approach Slab Reinforcing

Table 6.7.3-1

Approach Slab % Slope Table on the next page.

APPROACH SLAB % SLOPE		
ROADWAY SLOPE	DOWNHILL APPROACH SLAB SLOPE	UPHILL APPROACH SLAB SLOPE
0%	-5%	-5%
0% < SLOPE ≤ 1%	-6%	-5%
1% < SLOPE ≤ 2%	-7%	-5%
2% < SLOPE ≤ 3%	-8%	-5%
3% < SLOPE ≤ 4%	-9%	-5%
4% < SLOPE ≤ 5%	-10%	-5%
5% < SLOPE ≤ 6%	-11%	-5%
6% < SLOPE ≤ 7%	-12%	-5%
7% < SLOPE ≤ 8%	-13%	-5%
8% < SLOPE ≤ 9%	-14%	-5%
9% < SLOPE ≤ 10%	-15%	-5%

Approach Slab % Slope

Table 6.7.3-2

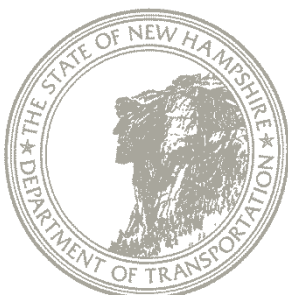


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Chapter 6 – Appendix C

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(Revised October 2023)



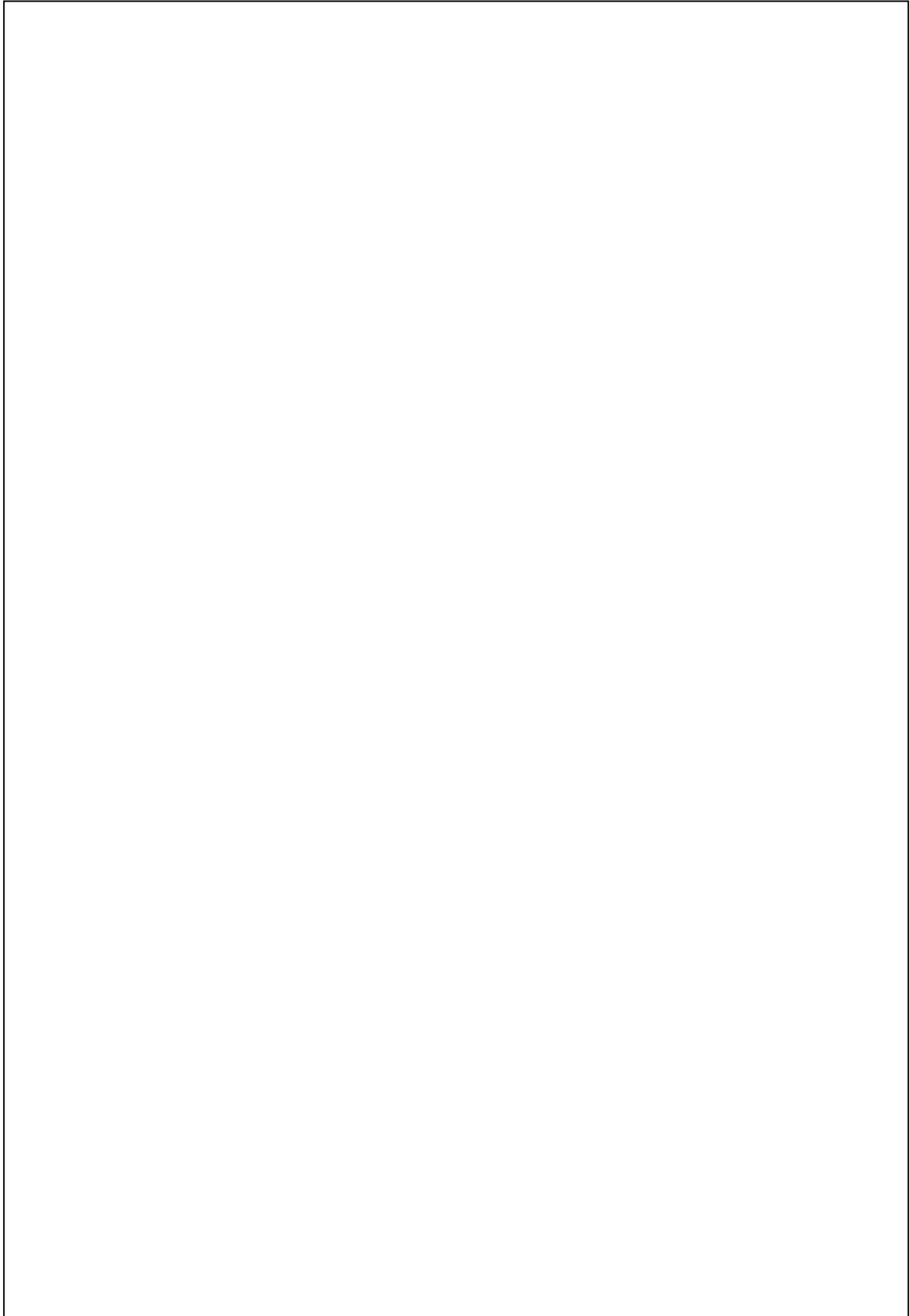
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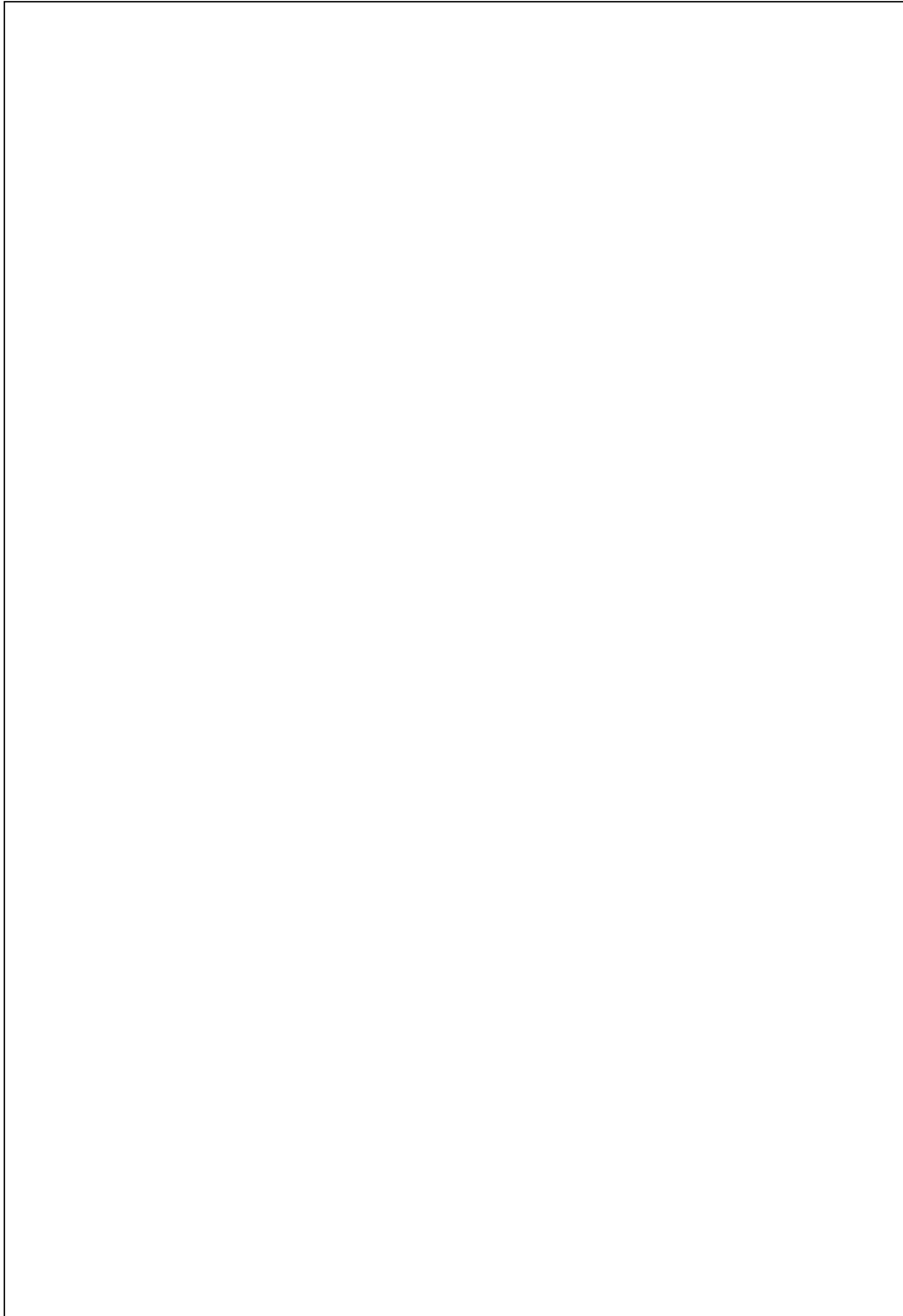
REINFORCING TENSION DEVELOPMENT AND
SPLICE LENGTHS

See AASHTO LRFD Bridge Design Specifications

REINFORCING TENSION DEVELOPMENT AND
SPlice LENGTHS

See AASHTO LRFD Bridge Design Specifications





TENSION DEVELOPMENT LENGTHS OF
90° AND 180° STANDARD HOOKS

See AASHTO LRFD Bridge Design Specifications

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REINFORCING BAR PROPERTIES

Standard Reinforcing Bar Properties					
Bar Size	Area (in ²)	Weight (lb/ft)	Nominal Diameter (in)	Outside Diameter (in)	Standard Mill Length (ft)
#3	0.11	0.376	0.375	0.42	40
#4	0.20	0.668	0.500	0.56	40
#5	0.31	1.043	0.625	0.70	60
#6	0.44	1.502	0.750	0.83	60
#7	0.60	2.044	0.875	0.96	60
#8	0.79	2.670	1.000	1.10	72
#9	1.00	3.400	1.128	1.24	72
#10	1.27	4.303	1.270	1.40	72
#11	1.56	5.313	1.410	1.55	90
#14	2.25	7.65	1.693	1.86	90
#18	4.00	13.60	2.257	2.48	90

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**PRESTRESSING STRAND PROPERTIES
AND DEVELOPMENT LENGTHS**

**AASHTO M203 Grade 270 Uncoated Prestressing Strands
Properties and Development Length**

Strand Diameter	Weight (lbs/ft)	Nominal Diameter (in)	Area (in ²)	Transfer Length (in)	Development Length k = 1.0 (ft)	Development Length k = 1.6 (ft)
3/8	0.290	0.375	0.085	22.5	4.77	7.63
7/16	0.390	0.438	0.115	26.3	5.57	8.92
1/2	0.520	0.500	0.153	30.0	6.36	10.18
1/2 S	0.568	0.520	0.167	31.2	6.62	10.58
9/16	0.651	0.563	0.192	33.8	7.16	11.46
0.60	0.740	0.600	0.217	36.0	7.63	12.21
0.62	0.788	0.620	0.231	37.2	7.89	12.62
0.70	1.000	0.700	0.294	42.0	8.91	14.25

Assumptions per AASHTO LRFD 5.11.2.4:

normal weight concrete

$f_{ps} = f_{pu} = 270 \text{ ksi}$

$f'_{ci} \leq 10 \text{ ksi}, f'_c \leq 15 \text{ ksi}$

transfer length = $60d_b$ (AASHTO 5.11.4.1)

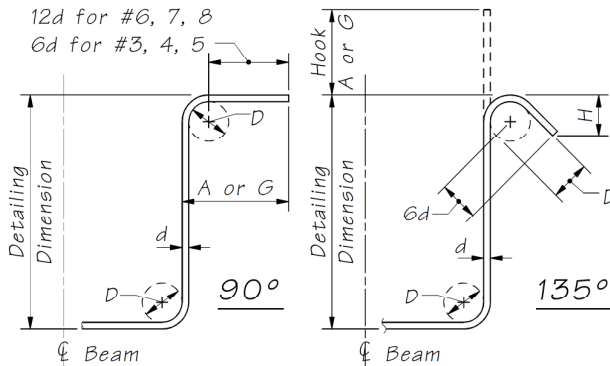
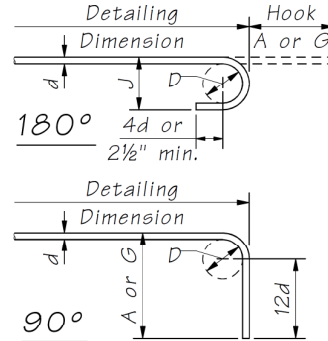
$f_{pe} = (270 \text{ ksi} \times 0.80) - 40 \text{ ksi} = 176 \text{ ksi}$ (AASHTO Table 5.9.3-1)

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STANDARD HOOKS

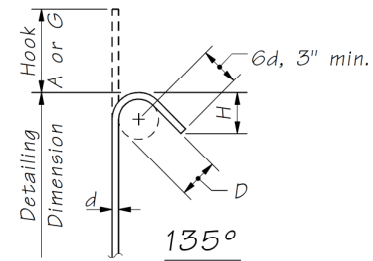
RECOMMENDED END HOOKS
All Grades
D = Finished bend diameter

Bar Size	D	180° HOOKS		90° HOOKS
		A or G	J	A or G
#3	2¼"	5'	3'	6"
#4	3"	6'	4'	8"
#5	3¾"	7'	5'	10"
#6	4½"	8'	6'	1'-0"
#7	5¼"	10"	7'	1'-2"
#8	6"	11"	8"	1'-4"
#9	9½"	1'-3"	11¾"	1'-7"
#10	10¾"	1'-5"	1'-1¼"	1'-10"
#11	1'-0"	1'-7"	1'-2¾"	2'-0"
#14	1'-6¼"	2'-3"	1'-9¾"	2'-7"
#18	2'-0"	3'-0"	2'-4½"	3'-5"



STIRRUP AND TIE HOOK DIMENSIONS
All Grades

Bar Size	D	90° HOOKS			135° HOOKS	
		Hook A or G	Hook A or G	H	Hook A or G	H Approx.
#3	1½"	4"	4"	2½"		
#4	2"	4½"	4½"	3"		
#5	2½"	6"	5½"	3¾"		
#6	4½"	1'-0"	8"	4½"		
#7	5¼"	1'-2"	9"	5¼"		
#8	6"	1'-4"	10½"	6"		



135° SEISMIC STIRRUP/TIE
HOOK DIMENSIONS
All Grades

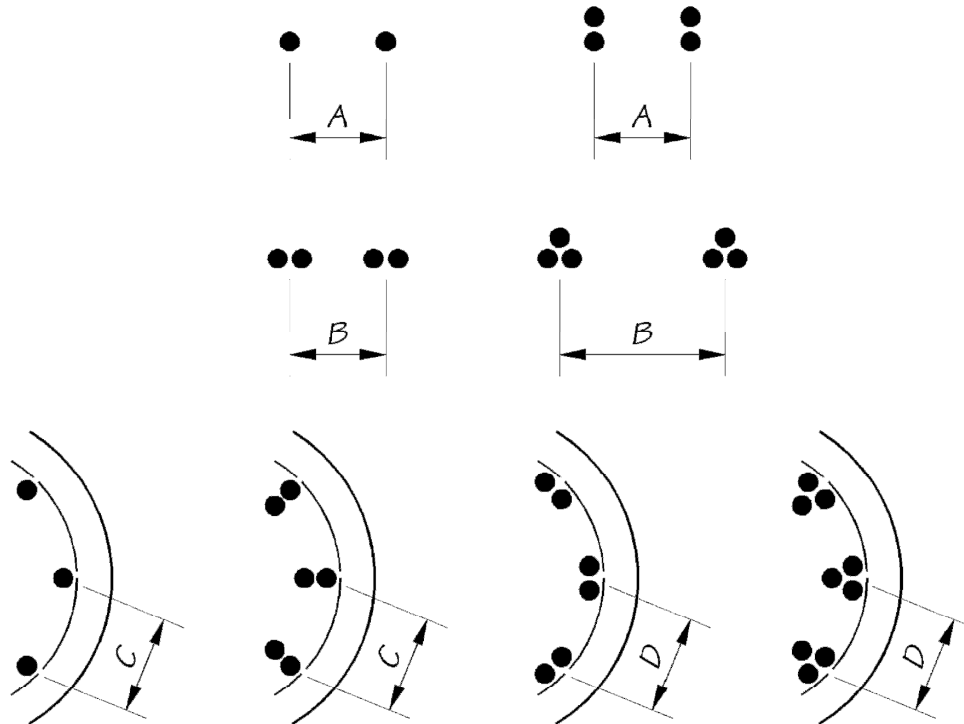
Bar Size	D	135° HOOKS	
		Hook A or G	H Approx.
#3	1½"	4¼"	3"
#4	2"	4½"	3"
#5	2½"	5½"	3¾"
#6	4½"	8"	4½"
#7	5¼"	9"	5¼"
#8	6"	10½"	6"

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Minimum Reinforcing Clearance & Spacing for Beams and Columns

PREFERRED MINIMUM CLEARANCE AND SPACING FOR BEAMS AND COLUMNS.
(DISTANCES IN INCHES)

Bar Size	A	B	C	D
#4	3¼	4	-	-
5	3½	4¼	-	-
6	3¾	4½	-	-
7	4	4¾	-	-
8	4¼	5	4½	5½
9	4½	5¼	4¾	6
10	4¾	5½	5	6¼
11	5	6	5⅓	6½
14	5½	7	5½	7¼
18	6	8½	6¾	9



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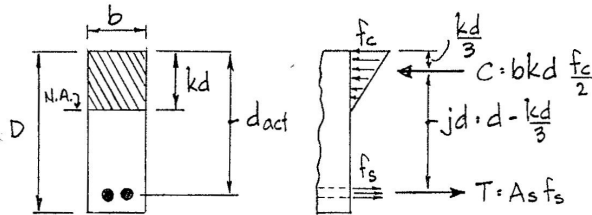
WORKING STRESS DESIGN
(Rectangular Reinforced Concrete Beam)

CONCRETE (PSI)	f'c	3,000	3,000	4,000
REBAR	GRADE	40	60	60
PROPERTIES (PSI)	fc	1,200	1,200	1,600
	(PSI) fy	40,000	60,000	60,000
	(PSI) fs	20,000	24,000	24,000
	n	9	9	8
	k	.351	.310	.348
	j	.883	.897	.884
	(KSI) K	.186	.167	.246
EFFECTIVE DEPTH*	dreq'd	$\sqrt{5.38 M_T}$	$\sqrt{5.99 M_T}$	$\sqrt{4.06 M_T}$
AREA STEEL REQ'D	Asreq'd	$\frac{.680 M_T}{d_{act}}$	$\frac{.557 M_T}{d_{act}}$	$\frac{.566 M_T}{d_{act}}$

* FOR A ONE-FOOT SECTION UNITS: $M_T \equiv$ KIP-FT. $d \equiv$ INCHES $A_s \equiv$ SQ. IN.

DERIVATION:

GIVEN: $E_s = 29,000,000$ psi
 $E_c = 57,000 \sqrt{f'_c}$ psi
 $n = E_s / E_c$
 $r = f_s / f_c$
 $k = n / (n + r)$
 $j = 1 - k/3$
 $K = 1/2 f_c j k$



CONCRETE BEAM FORCE DIAGRAM

FORMULA (dreq'd) FROM COMPRESSION FORCE:

$$M_T = Cjd = bkd \frac{f_c}{2} jd = bd^2 \frac{f_c}{2} jk = bd^2 K, \text{ where } K = \frac{f_c}{2} jk$$

$$d_{req'd} = \sqrt{\frac{2M_T}{bjkf_c}} = \sqrt{\frac{M_T}{bK}}, \text{ FOR } b=1.0' \left[\text{e.g. } d_{req'd} = \sqrt{\frac{M_T}{(1.0)(.246)}} = \sqrt{4.06 M_T} \right]$$

FORMULA (Asreq'd) FROM TENSION FORCE:

$$M_T = Tjd = A_s \cdot f_s \cdot j \cdot d$$

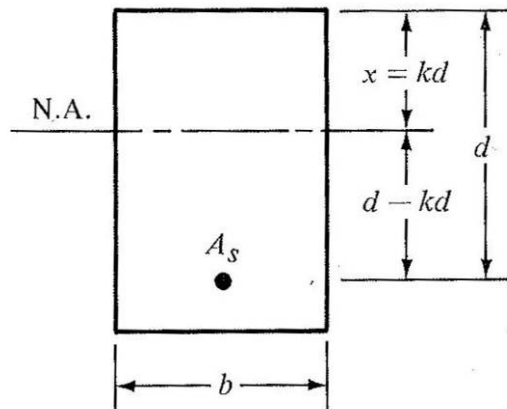
$$A_{sreq'd} = \frac{M_T}{f_s \cdot j \cdot d} \left[\text{e.g. } A_{sreq'd} = \frac{(12 \frac{in}{ft}) M_T}{(24)(.884)d} = \frac{.566 M_T}{d_{act}} \right]$$

- REFERENCES: (1) AASHTO (2) DESIGN OF CONCRETE STRUCTURES, WINTER
 (3) FOUNDATION ANALYSIS AND DESIGN, BOWLES

WORKING STRESS DESIGN (Rectangular Reinforced Concrete Beam)

Reference:

McCormac, Jack C., *Design of Reinforced Concrete*, Harper and Row Publishers, 1978.



Taking moments about the neutral axis

$$bkd\left(\frac{kd}{2}\right) = nA_s(d - kd)$$

Letting p = percentage of steel = A_s/bd ; thus $A_s = pbd$:

$$\frac{bk^2d^2}{2} = n\rho bd^2 - n\rho bd^2k$$

$$k^2 = 2\rho n - 2\rho nk$$

$$k^2 + 2\rho nk = 2\rho n$$

$$(k + \rho n)(k + \rho n) = 2\rho n + (\rho n)^2$$

$$k + \rho n = \sqrt{2\rho n + (\rho n)^2}$$

$$k = \sqrt{2\rho n + (\rho n)^2} - \rho n \quad (2.1)$$

The internal forces (C = total compression and T = total tension) shown in Figure 2.15 are now considered. C is located at the center of gravity (c.g.) of the compression stress triangle that is a distance $kd/3$ from the top of the beam, and T is located at the center of gravity of the steel bars. The distance between C and T is shown as jd .

Solving for the value of j

$$jd = d - \frac{kd}{3}$$

$$j = 1 - \frac{k}{3}$$

The moment of the couple Cjd or Tjd must equal the external moment M , and from these expressions values for f_s and f_c can be obtained.

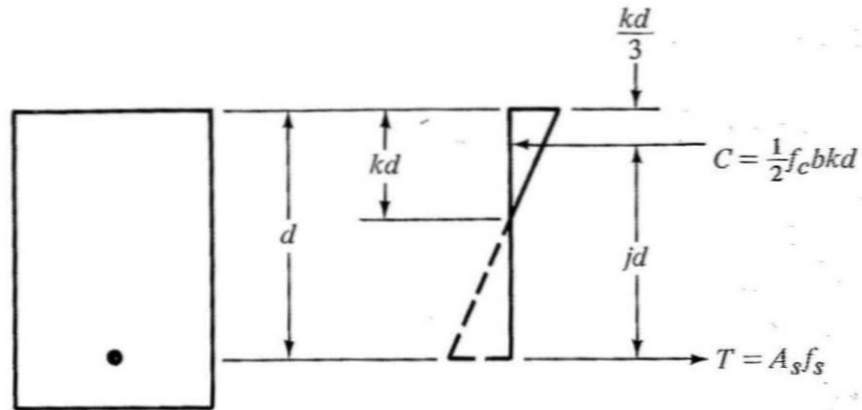


Figure 2.15

For the steel

$$Tjd = M$$

$$A_s f_s jd = M$$

$$f_s = \frac{M}{A_s jd}$$

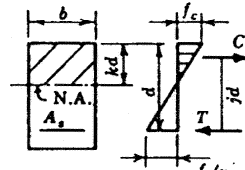
For the concrete

$$Cjd = M$$

$$\frac{f_c}{2} b k d j d = M$$

$$f_c = \frac{2M}{b d^2 k j}$$

COEFFICIENTS (K, k, j, p) FOR RECTANGULAR SECTIONS

f' _c and n	f _c	K	k	j	p	K	k	j	p				
		f _s = 16,000 a = 1.13				f _s = 18,000 a = 1.29							
2500	875.	137.	.356	.881	.0097	128.	.329	.890	.0080	$p = \frac{A_s}{bd}$ $k = \frac{1}{1 + f_s/nf_c} \quad j = 1 - \frac{1}{3}k$ $p^* = \frac{f_c}{2f_s} \times k \quad K = \frac{f_c}{2}kj$ $a = \frac{f_s}{12,000} \times (\text{av. } j\text{-value})$ for use in $A_s = \frac{M}{ad} \quad \text{or} \quad A_s = \frac{NE}{adi}$			
	1000.	169.	.387	.871	.0121	158.	.359	.880	.0100				
	1125.	201.	.415	.862	.0146	190.	.387	.871	.0121				
	1250.	235.	.441	.853	.0172	222.	.412	.863	.0143				
3000	1050.	173.	.376	.875	.0124	162.	.349	.884	.0102				
	1200.	212.	.408	.864	.0153	199.	.380	.873	.0127				
	1350.	252.	.437	.854	.0184	238.	.408	.864	.0153				
	1500.	294.	.463	.846	.0217	278.	.434	.855	.0181				
4000	1400.	249.	.412	.863	.0180	234.	.384	.872	.0149				
	1600.	303.	.444	.852	.0222	286.	.416	.861	.0185				
	1800.	359.	.474	.842	.0266	341.	.444	.852	.0222				
	2000.	417.	.500	.833	.0313	397.	.471	.843	.0261				
5000	1750.	327.	.437	.854	.0239	309.	.408	.864	.0199				
	2000.	397.	.470	.843	.0294	376.	.441	.853	.0245				
	2250.	468.	.500	.833	.0351	446.	.470	.843	.0294				
	2500.	542.	.526	.825	.0411	518.	.497	.835	.0345				
	3000.	694.	.571	.810	.0535	666.	.542	.819	.0452				
		f _s = 20,000 a = 1.44				f _s = 22,000 a = 1.60				f _s = 24,000 a = 1.76			
2500	875.	120.	.306	.898	.0067	113.	.287	.904	.0057	107.	.269	.910	.0049
	1000.	149.	.336	.888	.0084	141.	.315	.895	.0072	133.	.296	.901	.0062
	1125.	179.	.362	.879	.0102	170.	.341	.886	.0087	161.	.321	.893	.0075
	1250.	211.	.387	.871	.0121	200.	.365	.878	.0104	191.	.345	.885	.0090
3000	1050.	152.	.326	.891	.0085	144.	.305	.898	.0073	136.	.287	.904	.0063
	1200.	188.	.356	.881	.0107	178.	.334	.889	.0091	169.	.315	.895	.0079
	1350.	226.	.383	.872	.0129	214.	.361	.880	.0111	204.	.341	.886	.0096
	1500.	265.	.408	.864	.0153	252.	.385	.872	.0131	240.	.365	.878	.0114
4000	1400.	221.	.359	.880	.0126	210.	.337	.888	.0107	199.	.318	.894	.0093
	1600.	272.	.390	.870	.0156	258.	.368	.877	.0134	246.	.348	.884	.0116
	1800.	324.	.419	.860	.0188	309.	.396	.868	.0162	295.	.375	.875	.0141
	2000.	379.	.444	.852	.0222	362.	.421	.860	.0191	347.	.400	.867	.0167
5000	1750.	292.	.383	.872	.0168	278.	.361	.880	.0144	265.	.341	.886	.0124
	2000.	358.	.415	.862	.0208	341.	.392	.869	.0178	326.	.372	.876	.0155
	2250.	426.	.444	.852	.0250	407.	.421	.860	.0215	390.	.400	.867	.0187
	2500.	496.	.470	.843	.0294	475.	.447	.851	.0254	456.	.425	.858	.0221
	3000.	641.	.516	.828	.0387	617.	.492	.836	.0335	595.	.470	.843	.0294
		f _s = 27,000 a = 2.00				f _s = 30,000 a = 2.24				f _s = 33,000 a = 2.48			
2500	875.	99.	.247	.918	.0040	92.	.228	.924	.0033	86.	.211	.930	.0028
	1000.	124.	.272	.909	.0050	115.	.252	.916	.0042	108.	.234	.922	.0036
	1125.	150.	.296	.901	.0062	140.	.275	.908	.0052	132.	.256	.915	.0044
	1250.	178.	.319	.894	.0074	167.	.296	.901	.0062	157.	.277	.908	.0052
3000	1050.	126.	.264	.912	.0051	117.	.244	.919	.0043	110.	.226	.925	.0036
	1200.	157.	.290	.903	.0064	147.	.269	.910	.0054	138.	.251	.916	.0046
	1350.	190.	.315	.895	.0079	178.	.293	.902	.0066	168.	.273	.909	.0056
	1500.	225.	.338	.887	.0094	211.	.315	.895	.0079	199.	.295	.902	.0067
4000	1400.	185.	.293	.902	.0076	173.	.272	.909	.0063	162.	.253	.916	.0054
	1600.	230.	.322	.893	.0095	215.	.299	.900	.0080	203.	.279	.907	.0068
	1800.	277.	.348	.884	.0116	260.	.324	.892	.0097	246.	.304	.899	.0083
	2000.	326.	.372	.876	.0138	308.	.348	.884	.0116	291.	.327	.891	.0099
5000	1750.	247.	.315	.895	.0102	231.	.293	.902	.0085	218.	.274	.909	.0073
	2000.	305.	.345	.885	.0128	287.	.321	.893	.0107	271.	.301	.900	.0091
	2250.	366.	.372	.876	.0155	346.	.347	.884	.0130	327.	.326	.891	.0111
	2500.	430.	.397	.868	.0184	407.	.372	.876	.0155	386.	.350	.883	.0132
	3000.	564.	.441	.853	.0245	537.	.415	.862	.0208	511.	.392	.869	.0178

***Balanced steel ratio** applies to problems involving bending only.