Chapter 5

SUBSTRUCTURES

Smith Bridge, Houlton

West Bridge, Fairfield-Benton

5 SUBSTRUCTURES

5.1 Terminology

B	footing width
B’	effective footing width
C	point designating center of footing
D	height of soil in front of structure, which is applicable to passive resistance
$DL_{V,}$ LL_v	vertical structural/superstructure loads applied to abutment wall
D_f	depth to fixity
e	eccentricity of the resultant of all vertical forces at the bottom of the footing, measured from mid-width of footing
e_o	eccentricity calculated about the toe of the footing, to be used for overturning calculations
E_p	modulus of elasticity of pile
E_g	modulus of elasticity of end span beam/girder
F.G.	finished grade elevation
H	height of structure or failure plane
H_t	horizontal force required to translate pile
I_p	moment of inertia of pile
I_g	moment of inertia of end span beam/girder (composite I for composite beams)
K	effective length factor
K_a	active earth pressure coefficients for level or sloped backfill
K_{ho}	active earth pressure coefficient corresponding to a broken backslope
K_o	at-rest earth pressure coefficient
K_p	passive earth pressure coefficient.
L	heel length
L’	effective footing length
L_e	effective pile length from ground surface to the point of assumed fixity below ground, including scour effects.
L_s	length of end span
L_u	exposed pile length above ground
L_{us}	unsupported length
M	pile head moment
M_o	overturning moment
M_r	resisting moment
M_t	moment induced in the pile from the horizontal translation
O	point designating the toe of footing
$P_{h,q}$	horizontal traffic surcharge force behind abutment wall
P_h	horizontal soil active force behind abutment wall
P_L	allowable lateral load
P_p	horizontal passive force
P_t	pile reaction resulting from the earth pressure on the abutment

q_s	traffic live load surcharge pressure
Q	factored horizontal sliding force
$Q_{applied}$	applied load or stress
R	resultant force at base of footing
R_n	nominal resistance of footing, pile, shaft or micropile
R_R	factored resistance of a footing, pile, shaft or micropile
R_f	factored bearing or sliding resistance of a footing
R_g	beam/girder rotation (radians)
S_p	section modulus of the pile
t	footing thickness
w	water content (percent)
W	total beam/girder live load, end span
W_{c1}, W_{c2}	weight of abutment wall, footing
W_s	weight of soil above heel
W_{toe}	weight of soil above toe
X_{DL}	distance from the point of interest to the dead load reaction (centerline of bearing)
X_{LL}	distance from the point of interest to the live load reaction (centerline of bearing)
X_{WS}	distance from the point of interest to the centroid of W_s
X_{WC1}	distance from the point of interest to the centroid of W_{c1}
X_{WC2}	distance from the point of interest to the centroid of W_{c2}
X_{wtoe}	distance from the point of interest to the centroid of W_{toe}
y	the depth of seal from top of seal to bottom of seal
z	the depth of water from water surface to bottom of seal
α	batter angle from the horizontal plane
β	backfill slope
δ	friction angle between soil/bedrock and concrete
γ	soil weight
λ	column slenderness factor
η_i	factors to account for ductility, redundancy and operational importance
γ_i	load factor (general)
γ_p	permanent load factor
ϕ	soil internal angle of friction
σ_v	factored bearing stress at base of footing
τ	horizontal superstructure forces transmitted through bearing at wall top
Φ_c	resistance factor for axial compression
Φ_f	resistance factor for flexure
φ	resistance factor (general - geotechnical)
φ_{bc}	resistance factor for bearing resistance
φ_{dyn}	resistance factor for driven piles, dynamic analysis methods
φ_{stat}	resistance factor for piles, static analysis methods
φ_{ep}	resistance factor for passive soil resistance
φ_τ	resistance factor for sliding resistance between footing and soil/rock

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.

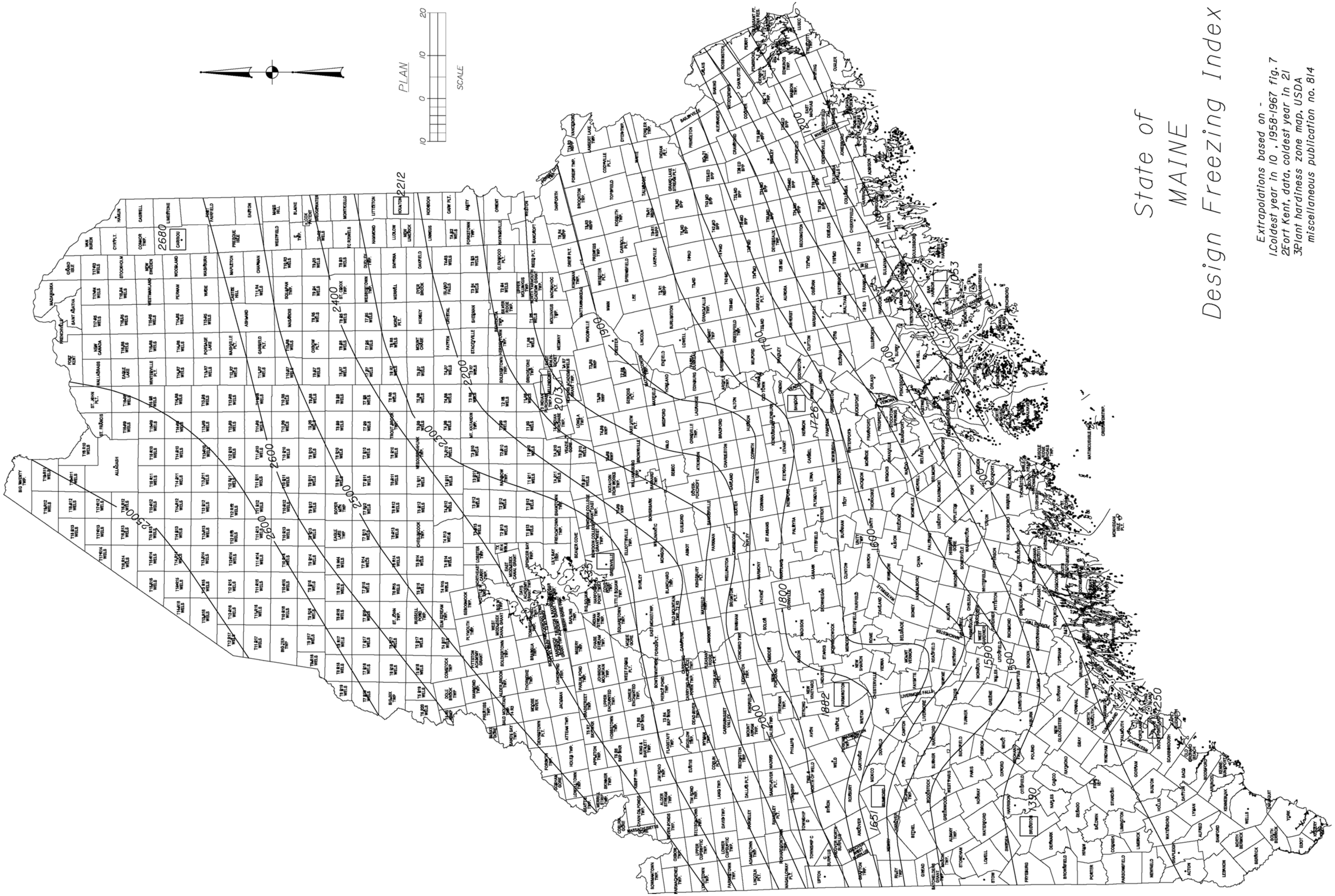


Figure 5-1 Maine Design Freezing Index Map

Example 5-1 illustrates how to use Table 5-1 and Figure 5-1 to determine the depth of frost penetration:

Example 5-1 Depth of Frost Penetration

Given: Site location is Freeport, Maine
Soil conditions: Silty fine to coarse Sand

Step 1. From Figure 5-1 Design Freezing Index = 1300 degree-days
Step 2. From laboratory results: soil water content = 28% and major constituent Sand
Step 3. From Table 5-1: Depth of frost penetration = 56 inches = 4.7 feet

Spread footings founded on bedrock require no minimum embedment depth. Pile supported footings will be embedded for frost protection. The minimum depth of embedment will be calculated using the techniques discussed in Example 5-1. Pile supported integral abutments will be embedded no less than 4.0 feet for frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

The final depth of footing embedment may be controlled by the calculated scour depth and be deeper than the depth required for frost protection. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.2.2 Seal Cofferdams

Seal cofferdams are used when a substructure unit must be constructed with its foundation more than 4 feet below the water table, to counteract the buoyant forces produced during pumping of the cofferdam. Once the cofferdam is constructed, the seal is placed under water and water is then pumped out of the cofferdam. This provides a dry platform for construction of the spread footing, or in the case of a pile foundation, the distribution slab. When a seal is needed, the top of footing or distribution slab is located approximately at streambed, and the depth of seal is calculated based upon the buoyancy of the concrete under the expected water surface during construction. The following formula can be used:

$$145 \cdot y = 62.4 \cdot z$$

where:

145 lb/ft^3 =	unit weight of concrete
62.4 lb/ft^3 =	unit weight of water
y =	the depth of seal from top of seal to bottom of seal
z =	the depth of water from water surface to bottom of seal

The depth of water in the above formula should be based on an appropriate flood event, but no less than Q10. The depth of water at tidal locations should be selected on a case-by-case basis, but no less than MHW. A note should be included on seal cofferdam sheets specifying the water elevation assumed in the design and specifying adjusting the seal depth should the water elevation at the time of construction be higher. To prevent seal buoyancy during a high water event after construction is complete, the Designer may specify vent holes at the design height of water, on a case-by-case basis.

Anchorage of the footing or distribution slab to the seal is required. For pile-supported foundations, this can be accomplished by extending the piles into the distribution slab. For seals founded on bedrock, dowels should be drilled and grouted into the seal after dewatering and prior to placement of the footing.

When sheet piling is used for a seal cofferdam, the minimum dimensions for the seal should be shown on the design drawings. These dimensions and details should be noted on the plans in conjunction with the appropriate notes in Appendix D Standard Notes Seal Cofferdams.

5.2.3 Cofferdams

Cofferdams are retaining structures with the retained material being water and soil. A separate cofferdam must be specified for the construction of each substructure unit (abutment or pier) that cannot be constructed completely in the dry. When water cannot be controlled so that footing concrete can be placed in the dry, a concrete seal must be placed below the elevation of the footing. Refer to Section 5.2.2 Seal Cofferdams.

Cofferdam design is the responsibility of the Contractor, and construction requirements are found in Standard Specification Section 511 – Cofferdams. Unless otherwise provided or approved, cofferdams are removed after the completion of the substructure, with care being taken not to disturb or otherwise damage the finished work.

Cofferdams should not be specified for substructure units that are constructed on dry land, such as on overpass structures. For large braced excavations a Special Provision should be included in the PS&E package to pay for braced excavations under the appropriate cofferdam item. Any temporary retaining structures that are required to support small structural excavations should be considered incidental to the appropriate structural excavation or substructure pay items.

Cofferdam requirements for culverts and other buried structures are found in Section 8.1.2 Construction Practices.

5.2.4 Concrete Joints

Concrete joints in a vertical plane are used in concrete construction to accommodate changes in the volume of concrete caused by such factors as drying shrinkage, creep, and the application of load. When concrete is restrained by internal or external forces, the stresses caused by concrete movement would be relieved by the formation of significant cracks, if joints were not provided. Construction joints are used to facilitate the sequence of construction, and are typically located in a horizontal plane for abutments, piers, and walls.

There are three types of joints commonly used in concrete construction. A concrete key is generally used with each joint for shear transfer, as shown in Standard Detail 502 (01). The Structural Designer should specify the proper concrete joint, depending upon its intended use.

- Contraction joints are normally used every 30 feet along a wall to control the location of cracks. Without these joints, the concrete would form cracks at unpredictable intervals. Reinforcing steel is normally not carried through the joint, except in rigid frame structures, where moment must be transferred from wall to slab.

- Expansion joints are used to prevent compression forces from abutting concrete from crushing or displacing the adjacent structure. It is good practice to locate expansion joints where expansion forces change direction, such as at wingwall turns. In retaining walls and abutment/wingwall systems, expansion joints should be spaced no more than 90 feet apart. Reinforcing steel is not carried through the joint.

- Construction joints are used between concrete placements when the sequence of construction requires more than one placement. The surface between placements becomes a construction joint. These joints may be designed to coincide with contraction or expansion joints. If not functioning as a contraction or expansion joint, reinforcing steel is normally carried through the joint.

- A horizontal construction joint in the abutment backwall should be shown on the plans to facilitate installation of the superstructure expansion device. This should normally be located at a minimum vertical distance of 1’-3” from the roadway surface, except for modular expansion devices, which must conform to the manufacturer’s recommendations (refer to Section 4.8.5 Modular Joints). Bent #5 bars at 1’-6” maximum spacing should be used in the top of the backwall. Welding to reinforcing steel is allowed in this area so that the Contractor can utilize the reinforcing steel to support the expansion device.

5.2.5 Seismic Considerations

Seismic analysis of bridges and foundations shall be performed in accordance with the LRFD Specifications or the AASHTO Guide Specifications for LRFD Seismic Design (herein referred to as the Guide Specification).

Seismic analysis is <u>not</u> required for the following:

- Single-span bridges, regardless of Seismic Zone or Seismic Design Category (SDC)
- Any bridge in Seismic Zone 1 or SDC A, with the exceptions described below.

For all bridges, including those for which seismic analysis is not required, superstructure connections and bridge seat dimensions should be satisfied per LRFD 3.10.9 and 4.7.4.4, respectively.

For critical or essential bridges, including those in Seismic Zone 1 or SDC A, the Department may specify a higher Seismic Zone or SDC than that specified by the LRFD Specifications and the Guide Specification or specify appropriate seismic provisions. Critical and essential bridges are not specifically classified in this Bridge Design Guide, but will be designated as such by the Department at its discretion.

In general, bridges that may be classified by the Department as critical or essential are as follows:

- Bridges that are required to be open to all traffic once inspected after the design earthquake and usable by emergency vehicles and for security, defense, economic or secondary life safety purposes immediately after the design earthquake.
- Bridges that should be open to emergency vehicles and/or for security, defense or economic purposes after the design earthquake and open to all traffic within days after that event.
- Bridges that are formally designated as critical for a defined local emergency plan.

For non-conventional bridges, including cable-stayed and suspension bridges, truss bridges, arch type bridges and movable bridges the Department will specify and approve appropriate seismic design provisions.

It is estimated that most bridge sites in Maine will be classified as Seismic Zone 1 or SPC A. The exception are bridges in the extreme northwest portion where the subsurface conditions might be classified as Site Class B, C or D, and bridge sites everywhere where the subsurface conditions are Site Class

E, except those in downeast coastal Maine. It is estimated these bridge sites will be classified as Seismic Zone 2 or SDC B.

For bridges requiring seismic analysis, the effect of earthquake loading on the foundations shall be investigated using the extreme event limit state in LRFD Table 3.4.1-1 with resistance factors, φ, of 1.0 and an appropriate seismic analysis method as described in LRFD 4.7.4.3 and LRFD 3.10.9.2 through 3.10.9.4. The foundation design should consider the effect of wall inertia and amplification of active earth pressure by earthquake determined by the Mononobe-Okabe method. The Mononobe-Okabe method for determining equivalent static fluid pressure for seismic loads on walls is presented in LRFD 11.6.5 and Appendix A11. LRFD Appendix A10 gives additional guidance regarding seismic analysis and design of foundations.

For foundations on soil and rock, the location of the resultant of the reaction forces due to earthquake loading should be within the middle two-thirds (2/3) of the footing base for $\gamma_{EQ} = 0.0$ and within the middle eight-tenths (8/10) of the footing base for $\gamma_{EQ} = 1.0$. For in between values of $\gamma_{EQ,}$ the restriction for the location of the resultant is obtained by linear interpolation of the preceding values of γ_{EQ}.

For overall stability of a retaining wall when earthquake loading is included, a resistance factor, φ, of 0.90 should be used. For bearing resistance, a resistance factor, φ, of 0.80 should be used for gravity and semigravity walls and 0.90 for MSE walls.

Where the backfill or foundation soils are saturated, consideration should be given to address the possibility of soil liquefaction and lateral spreading. Liquefaction design guidance is provided in LRFD 10.5.4.2, 11.5.4.2 and Appendix A10.

5.3 Spread Footings

Spread footings should be designed and proportioned for the strength, service, and extreme event limit states such that the factored resistance is not less that the effects of the factored loads specified in LRFD Article 3.

Selection of foundation type is based on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, flood history, potential for liquefaction, undermining, scour or wave action, frost depth, and ease and cost of construction.

5.3.1 Service Limit States

Spread footings at the service limit state shall be investigated for:

- Settlement

- Horizontal movement
- Rotation
- Overall stability of slope with the footing
- Scour at the design flood, specified in LFRD 2.6.4.4.2 and 3.7.5

Settlement shall be investigated for the Service I Load Combination and rotations and horizontal movements shall be investigated at all applicable service limit states.

The tolerable level of ultimate settlement, differential settlement, rotation and horizontal movement shall be controlled by superstructure tolerance, rideability, span length, road classification, long-term maintainability and economy.

Bearing resistance estimated using presumptive allowable bearing resistances shall only be applied to address service limit state load combinations or for preliminary sizing of footings.

Service limit state analyses shall use unfactored loads. Resistance factors for the service limit state shall be taken as 1.0. The exception is the investigation of the overall slope stability of a retaining wall or an earth slope supporting a retaining wall footing or an abutment footing. In those instances, the earth slopes should be investigated at the Service I Load Combination, with a resistance factor, ϕ, of 0.65.

5.3.2 Strength Limit States

The design of spread footings at the strength limit states shall consider:

- Factored bearing resistance
- Eccentricity or loss of contact
- Sliding
- Loss of lateral and vertical support due to scour at the design flood event; the design flood is defined as the more severe of the 100-year even or an overtopping flood of lesser recurrence interval.
- Factored structural resistance

Resistance factors for the bearing resistance of spread footings at the strength limit state are provided in Section 5.3.5.3. Resistance factors for sliding are provided in Section 5.3.8.

A modified Strength Limit State analysis should be performed that includes in the ice pressures specified in Section 3.9 Ice Loads, with the appropriate strength limit state resistance factors. That Strength Limit State that results in the extreme force and moment effects should be selected.

5.3.3 Extreme Event Limit States

Spread footings should be designed for extreme events such as seismic loads, liquefaction, check flood for scour, vessel impact, vehicle or railway collision, and ice.

The ice pressures for the Extreme Event II Limit State should be unfactored and applied at Q1.1 and Q50 elevations as defined in Section 3.9 Ice Loads but with the ice thickness increased by 1 foot.

Resistance factors for extreme event limit states shall be taken as 1.0.

For the extreme event limit state, the Designer should consider scour due to the check flood event and should determine that there is adequate foundation resistance to support all applicable unfactored loads with a resistance factor of 1.0. Flood event loads should include debris loads, where applicable.

Extreme limit state design checks for spread footings shall include checks of:

- Bearing resistance
- Eccentricity
- Sliding
- Overall stability

5.3.4 Footing Depth

Footings should be embedded a sufficient depth to provide adequate bearing materials and protection against frost action, erosion and scour.

5.3.4.1 Bearing Materials

A footing should ideally be founded on a single material type throughout its bearing length. If a combination of materials is present underlying the footing (i.e., bedrock and granular material) the granular material should be removed to the bedrock surface and replaced with concrete fill. In special situations where constructing a footing on dissimilar materials cannot be avoided, see the Geotechnical Designer.

Footings should be founded on firm soils or bedrock. Any organic, loose, or otherwise unsuitable material encountered at the footing elevation should be removed to the full depth and replaced with compacted granular fill or concrete fill to the bottom of footing elevation. If concrete fill is used under a foundation, the pay limits should be shown as a vertical plane and should be designated as "Pay Limit for Structural Excavation and Concrete Fill". The distance outside the footing for the concrete fill pay limit should be determined for each individual case and must be shown on the design drawings. Foundation bearing conditions should be approved in the field by the Construction Resident or Geotechnical Designer.

5.3.4.2 Footings on Bedrock

For footings supported on bedrock the surface will be cleaned of all weathered bedrock, fractured material, loose soil, and/or ponded water prior to placement of the footing concrete. Smooth bedrock should be roughened or serrated prior to placing concrete to enhance sliding stability. The foundation bearing areas should be approximately level. Bedrock slopes that exceed 4H:1V should be step-serrated or suitably benched to create level steps or a completely level subgrade. For bedrock slopes between 4H:1V and 6H:1V consider dowels into bedrock to control sliding potential.

5.3.4.3 Frost Protection

Footings will be placed below the depth of frost penetration as discussed in Section 5.2.1 Frost. Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

5.3.4.4 Scour Protection

Spread footings on soil or erodible rock at stream crossings should be founded at a depth at least 2 feet below scour depth of scour determined for the check flood for scour. Spread footings supported on soil within the stream channel shall be located a minimum of 6 feet below the thalweg of the waterway. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.3.5 Bearing Resistance

5.3.5.1 General

Spread footings for abutments and retaining walls are to be proportioned to ensure stability against bearing capacity failure. Safety against deep seated foundation failure shall also be investigated per LRFD 10.6.2.3.

Bearing resistance should be investigated at the strength limit state.

LRFD Article 11.6.3.2 and Figures 11.6.3.2-1 and -2 provide examples for calculating the vertical bearing stress. In general, load factors selected should produce the total extreme force effect. Specific guidance for selection of load factors for bearing resistance is provided in LRFD Figure C11.5.6-1. Where there is a live load surcharge, the factored surcharge force is included over the backfill immediately above the wall base or footing.

Spread footings should be designed such that the factored design stress does not exceed the factored bearing resistance of the soil or rock. The nominal bearing resistance of footings on soil may be estimated using the Munfakh procedure outlined in LRFD Article 10.6.3.1.2. The use of Terzaghi, Meyerhof, or Vesic methods for estimating the nominal bearing resistance is also acceptable. Consideration of shape factors, inclined loads, ground surface slope, and eccentric loading should be included in the calculation, if applicable. A resistance factor shall be applied to the calculated nominal resistance. Structures should be designed such that the maximum factored pressure on the soil or rock under footings does not exceed the factored bearing resistance provided by the Geotechnical Designer.

The bearing resistance at the service limit state will be settlement controlled (typically 1 inch). Presumptive bearing resistance charts based on soil or rock type may be used to determine the service limit state bearing resistance.

For spread footings on bedrock, the design of the footing is typically controlled by overall stability, i.e., failure along discontinuities in the rock mass or eccentricity. Therefore, the Designer should verify overall stability by sizing the footing based on eccentricity at the strength limit state and then checking the nominal bearing resistance at the service and strength limit states.

5.3.5.2 Bearing Stress Distribution

The distribution of soil pressure should be consistent with the foundation material, whether it is soil or bedrock. When proportioning footing dimensions to meet settlement and bearing resistance requirements, the distribution of bearing stress on the effective footing area shall be assumed to be:

- Uniform for footings on soils
- Triangular or trapezoidal for footings on rock

For structural design of footings, a triangular or trapezoidal stress distribution based on factored loads should be used regardless if the footing bears on soil or rock.

When loads are eccentric, the bearing stress is distributed to the effective footing area, L’ x B’, where the reduced dimensions are taken as:

- $B' = B - 2e_B$
- $L' = L - 2e_L$

where e_B and e_L are the eccentricities relative to a point at the center of the footing, parallel to the B and L dimensions, respectively.

5.3.5.3 Bearing Resistance Factors

The resistance factors for bearing resistance are provided in Table 5-2.

Table 5-2 Bearing Resistance Factors

Method/Soil/Condition	Bearing Resistance Factor, φ_b
Theoretical method (Munfakh et al. 2001) in clay	0.50
Theoretical method (Munfakh et al. 2001) in sand using SPT	0.45
Semi-empirical methods (Meyerhof, 1957, Terzaghi, Vesic) all soils	0.45
Footings on rock	0.45
Plate Load Test	0.50

5.3.6 Settlement

The design of spread footings is frequently controlled by settlement at the service limit state. It is advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

Total and differential settlement should be evaluated. The total settlement includes elastic settlement, primary consolidation, and secondary compression. Elastic settlement results from the compression of the material supporting the foundation or from reduction in pore space in nonsaturated soils. Consolidation settlement occurs when saturated, fine-grained soils experience an increase in stress. Some soils, after experiencing primary consolidation settlement, continue to strain after excess pore-water pressures are dissipated. This process is termed secondary compression, or “creep”.

Immediate or elastic settlement should be determined using the Service I Load Combination, specified as unfactored dead load, plus the unfactored component of live loads assumed to extend to the footing level. Time-dependent settlements, i.e., primary consolidation and secondary compression settlement may be determined using the unfactored dead load only. Other factors that can affect settlement, such as embankment loading, lateral and/or eccentric loading, and dynamic or earthquake loads should also be considered, where applicable.

Differential settlement occurs when one load-bearing member of a structure experiences total settlement of a different magnitude than an adjacent load-bearing member. Transportation structures, especially bridges, are not exceptionally tolerant of differential settlements. Deformation limitations will form the upper bound of allowable differential settlements used to design shallow foundations.

5.3.6.1 Tolerable Settlement

Foundation settlement criteria should be consistent with:

- The type of structure
- The function of the structure
- Anticipated service life
- Consequences of unacceptable movement on structure performance
- Long-term maintainability

Tolerable movements are frequently described in terms of angular distortion between members. Angular distortion (δ'/ℓ) between adjacent foundations should be limited to 0.008 radians for simple span bridges and 0.004 radians for continuous span bridges, where δ' is the differential settlement and ℓ is the span length. Angular distortion limits may deviate on a project by project basis, depending on:

- The cost of mitigating settlement through larger foundations, realignment, lightweight fills or surcharge
- Rideability
- Aesthetics
- Safety

Tolerance of the superstructure to lateral movement will depend on the bridge seat or joint widths, bearing type and structure type.

5.3.6.2 Settlement Analyses

Settlement may be estimated using procedures described in LRFD 10.6.2.4 or other generally accepted methods. The soil parameters used shall be based on the results of laboratory or insitu testing, or both. Total and differential settlement should be evaluated.

Settlement of spread footings on sand can be predicted using calculation methods by Hough, Peck-Bazaraa, D’Appolonia, or Schmertmann, as applicable.

5.3.7 Overall Stability

The overall global stability of spread footings on or near an earth slope should be investigated using Service I Load Combination and an appropriate resistance factor. Where a slope supports or contains a structural element, such as a spread footing supporting a wall or abutment, the resistance factor, φ, shall be taken as of 0.65

For foundations on spread footings constructed along rivers and streams, scour of foundation materials is evaluated as specified in LRFD 2.6.4.4.2. Extreme limit state design should check that the nominal resistance of the footing and slope remaining after the scour due to the check flood for scour can support the unfactored strength limit state loads with a resistance factor, φ, of 1.0

The overall stability of retaining wall spread footings on or near a slope should be evaluated using limiting equilibrium methods of analysis, which employ the Modified Bishop, simplified Janbu, Spencer, or other generally accepted methods of slope stability analysis.

5.3.8 Sliding

Failure by sliding should be investigated for all spread footings bearing on soil or bedrock. Passive earth pressure exerted by fill in front of the footing should be neglected in consideration that the soil may be removed as the result of scour or during future construction, and in consideration that soils in front of the footing will be subject to freeze-thaw weakening over time. If passive pressure is included as part of shear resistance to sliding, consideration should be made to possible removal of the soil in front of the foundation in the future. If passive resistance is included in the resistance, its magnitude is commonly taken as 50% of the maximum passive pressure resistance computed using Rankine Passive resistance. This is the basis of a resistance factor for passive resistance of φ_{ep} of 0.50.

The factored resistance against failure by sliding is taken as:

$$R_r = \varphi R_n = \varphi_s R_f + \varphi_{ep} R_{ep}$$

where:

R_n = nominal sliding resistance
φ_s = resistance factor for shear resistance between soil and foundation specified in Table 5-3.
R_f = nominal sliding resistance between soil and foundation
φ_{ep} = resistance factor for passive resistance = 0.50
R_{ep} = nominal passive resistance of the soil available throughout the design life of the structure.

Table 5-3 Resistance Factors for Sliding of Spread Footings at the Strength Limit State

Soil/Condition	Sliding Resistance Factor, φ_s
Precast concrete on sand	0.90
Cast-in-place concrete on sand	0.80
Cast-in-place or precast concrete on clay	0.85
Soil on soil	0.90
Cast-in-place concrete on rock (based on reliability theory analysis of footings on sand)	0.80
Cast-in-place concrete on rock (calibrated to ASD Factor of Safety of 1.5)	0.90

Spread footings should be designed such that the factored resistance to sliding, R_f, is greater than the factored force effects due to the horizontal components of loads. Load factors selected should produce the extreme force effect. The live load surcharge is not included over the heel. Specific guidance for selection of load factors for sliding are provided in LRFD Figure C11.5.6-2.

The nominal sliding resistance between footings and cohesionless soils is taken as:

$$R_f = V \times \tan \delta$$

where:

$\tan \delta = \tan \phi$ for cast-in-place footings on soil
$\tan \delta = 0.80 \tan \phi$ for precast footings on soil
V = total vertical force

The coefficient of friction, $\tan \phi$, for sliding should be as shown in Table 3-3 for the soil type under the footing and LRFD Table 3.11.5.3-1.

The nominal sliding resistance between footings and silt and/or clay soils should be taken to be the lesser of: (1) the undrained shear strength of the silt/clay, or, (2) one-half of the normal stress on soil when the footing is founded on at least 6 inches of compacted granular fill on silt/clay.

For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding. If smooth bedrock is present at the bearing elevation or if the coefficient of sliding is insufficient to resist lateral forces, the bedrock should be doweled to improve stability. When a footing is doweled into rock, the dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the Designer. The spacing of the dowels should be no greater than 3 feet between rows and no less than two rows. If sloping bedrock is present (steeper than 4H:1V) at the bearing elevation, the bedrock should be benched to create level steps or doweled to improve stability.

5.3.9 Eccentricity

Load factors for eccentricity selected should produce the extreme force effect. The live load surcharge is not included over the heel of the footing. Specific guidance for selection of load factors for eccentricity are provided in LRFD Figure C11.5.5-2. The location of the resultant of the reaction forces shall be:

- within the middle two-thirds (2/3) of the footing width or length, B or L, for footings on soils, or
- within the middle nine-tens (9/10) of the footing width or length, B or L, for footings on rock.

5.3.10 Ground Water Condition

Footing excavations below the ground water table, particularly in granular soils having relatively high permeability, should be made such that the hydraulic gradient in the excavation bottom is not increased to a magnitude that would cause the foundation soils to loosen or soften due to upward flow of water. Dewatering or cutoff measures to control seepage should be used where necessary. Footing design should be calculated using the highest anticipated ground water level at the footing location.

5.3.11 Drainage Considerations

Adequate drainage of materials behind structures is of great importance and should be provided as described in Section 5.4.1.9 Drainage.

5.4 Abutments

5.4.1 Conventional Abutments

5.4.1.1 General Design Requirements

Abutment and wingwall design should 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, eccentricity (overturning), pile capacity (if applicable) and structural capacity. Abutments should be designed for extreme events such as vessel collisions, vehicle collisions, and seismic activities, along with changed conditions such as scour, as applicable. The design of abutments and walls should satisfy service, strength, and extreme limit state requirements.

5.4.1.2 Loads Combinations and Load Factors

Structural analyses and geotechnical evaluation of abutments should be performed in accordance with the AASHTO LRFD. Abutments should be designed and proportioned to resist all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and as outlined in Chapter 3 Loads.

Abutments should be evaluated for each of the applicable limit states:

- Strength I-construction. Strength Limit State I with the exception that bridge superstructure DC and DW, and vehicular live loads, LL, are neglected. Load factors for the dead load of other components shall not be less than 1.25. Live load surcharge is included to account for construction equipment live loading during structure erection and a construction load factor of not less than 1.5 should be assumed. The Strength I-construction analysis should investigate any anticipated construction loadings, such as looking at the abutment partially backfilled without the superstructure in place.

- Strength I-a: Strength Limit State I, which models the basic load combination related to normal vehicle use of the bridge without wind, dead load plus earth pressure, finished grade, including the vertical component of the superstructure, approach slab, live load effects of traffic on the approach (LS) the vertical component of the live load from superstructure. Minimum vertical permanent load factors and maximum horizontal load factors are selected to produce extreme force effects for abutment sliding and eccentricity, and structural design of the abutment stem.

- Strength 1-b: Strength Limit State 1 as described above, except maximum vertical permanent load factors, including earth loads, are selected to produce an extreme force effect for bearing capacity analyses.

- Strength III: Load combination relating to the bridge exposed to high wind velocity (100 mph) without live loads. Minimum and maximum load factors should be selected for permanent loads to investigate the most extreme force or moment effect.

- Strength IV: Load combination relating to very high dead load to live load force effect ratios exceeding about 7.0 Strength IV will likely govern for bearing failure on long span bridges. It also will likely govern for structural design of the footing. Minimum and maximum load factors should be selected for Permanent Loads to investigate the most extreme force or moment effect.

- Strength V: Load combination relating to the bride exposed to wind velocity of 55 mph with live loads. Minimum and maximum load factors should be selected for permanent loads to investigate the most extreme force or moment effect.

- Service I: Service Limit State I – Load combination relating to normal operational use of the bridge with a 55 mph wind and all loads taken at their unfactored values.

For the load combinations with all dead loads applied, with or without the superstructure live load, distribute the superstructure loads over the length of the abutment between the fascia lines of the superstructure.

Where abutments are to be designed to resist earthquake forces, collisions by roadway or rail vehicles, or vessel collision, the structures should be evaluated for the following additional limit states:

- Extreme Event I – Load combination including earthquake forces

- Extreme Event II – Load combination relating to collision by vehicles or vessels.

Certain permanent loads, including earth loads, should be factored using the load factors γ_p. Permanent load factors should be selected to produce the total extreme factored force effect. Typical load factors, load combinations and the analyses for which they will govern, are provided in Table 5-4.

Table 5-4 Typical Load Groups and Load Factors (γ_i) for Abutments on Spread Footings

Controlling Load Group	γ_{DC}	γ_{EV}	γ_{LS}	γ_{EH} **(active or passive)**	γ_{LL}	**Analysis Governed**
Strength I-a	0.90	1.0	1.75	1.5	1.75	- Sliding - Eccentricity (overturning) - Structural design of wall stem
Strength I-b	1.25	1.35	1.75	1.5	1.75	- Bearing Capacity
Strength IV	1.50	1.35	--	1.5	--	- Bearing capacity - Structural design of the footing
Service I	1.0	1.0	1.0	1.0	1.0	- Settlement - Lateral movement - Angular distortion

Longitudinal forces for abutment design should include any live load longitudinal forces developed through bearings such as braking forces, or others as specified in LRFD Article 3.0, unless limited by friction capacity.

5.4.1.3 General

The Designer should estimate the load combinations which could be imposed on the abutment or wall and estimate the nominal resistance of the structural component or ground. Abutment components shall satisfy the following equation for each limit state:

$$\sum \eta_i \, \gamma_i \, Q_i \leq \Phi \, R_n = R_f$$

where:

η_i = Factors to account for ductility, redundancy and operational importance
γ_i = Load factor (dim)
Q_i = Load or stress
Φ = Resistance factor (dim)
R_n = Nominal resistance
R_f = Factored resistance

5.4.1.4 Strength Limit State Evaluations

The above equation should be used to evaluate abutments at the strength limit states for:

- Bearing resistance failure
- Lateral sliding
- Excessive loss of base contact (eccentricity)
- Pile failure
- Structural failure

The factored resistance, R_f, calculated for each mode of failure, is to be calculated using the appropriate resistance factors for bearing resistance, sliding, eccentricity, axial pile resistance and structural resistance.

The Designer should consider the consequences of changes in abutment foundation conditions at the strength limit state resulting from scour due to the design flood event using appropriate resistance factors.

5.4.1.5 Service Limit State Evaluations

Abutments should be investigated at the service limit state using the load and resistance equation in Section 5.4.1.3 for:

- Settlement
- Lateral displacement
- Overall slope stability
- Overall stability at the design flood

A resistance factor, φ, of 1.0 is used to assess abutment design at the service limit state. Overall stability of abutments on or near earth slopes should be investigated using resistance factors in Section 5.3.7 Overall Stability.

Tolerable vertical and lateral displacement criteria for abutment shall be developed based on the function and type of wall, anticipated service life, and consequences of unacceptable movements of the wall and effect on nearby structures. To control bridge superstructure damage, a limiting horizontal movement of abutments less than 1.5 inch is recommended. Utilities may not be able to accommodate very large movements, in which case a project-specific limiting movement should be developed.

5.4.1.6 Extreme Limit State Evaluations

Extreme limit state design checks for abutments should include:

- Bearing resistance
- Eccentricity
- Sliding
- Overall stability

A resistance factor, φ, of 1.0 is used in the load and resistance equation in Section 5.4.1.3 to assess abutment design at the extreme limit state.

The extreme event limit state design should check that the nominal abutment foundation resistance after scour due to the check flood event can support all applicable unfactored loads with a resistance factor of 1.0. For abutments on spread footings, refer to 5.3.4.4. For pile-supported abutments, refer to 5.4.1.12.

5.4.1.7 Load Considerations

A. Earth Loads

For abutment and wingwall designs, use the appropriate soil weight shown for Soil Type 4 (Table 3-3) for soil properties for backfill material. Abutments and retaining walls should be designed as unrestrained and free to rotate at the top in an active state of earth pressure. An active earth pressure coefficient, K_a, should be calculated using Rankine Theory for long-heeled cantilever abutments and wingwalls, and Coulomb Theory for short heeled cantilever abutments and gravity shaped walls. Refer to Section 3.6.5.1 Coulomb Theory. Soil Type 4 properties are consistent with materials typically used for backfill behind abutments and retaining walls. For unconventional backfills, i.e., tire shreds, light weight fills, etc., consult the Geotechnical Designer or Report.

B. Unit Weight of Concrete

A unit weight of 150 lb/ft^3 should be used for design purposes.

C. Live Load Surcharge Loads

Abutments without approach slabs should be designed with a live load surcharge when computing horizontal earth pressure. This additional lateral pressure on walls is approximated by a uniform horizontal earth pressure due to an equivalent height of soil, H_{eq}. Refer to Section 3.6.8

Surcharge Loads for guidance in computing this additional lateral surcharge pressure.

Wingwalls and retaining walls should also be designed for surcharge loads in accordance with Section 3.6.8.

In the case a structural approach slab is specified, reduction, but not elimination, of the surcharge loads is permitted per LRFD 3.11.6.2.

D. Lateral Loads

Load conditions should include any additional lateral pressures on the walls. These loads may include but are not limited to impact loads transmitted to the retaining walls from distribution slabs supporting crash barriers.

E. Collision Forces

Unless the department determines that site conditions indicate otherwise, abutments within a distance of 30 feet to the edge of a roadway or within 50 feet to the centerline of railway track shall be investigated for collision. Collision loads and crashworthy barrier design criteria for abutments are identical to those provided for Piers in Section 5.5.1.10 Pier Protection.

5.4.1.8 Backfill

Abutment walls and footings should be backfilled with granular borrow for underwater backfill. Extend underwater granular backfill for a horizontal distance of at least 10 feet from the back face of the abutment wall and 1 foot behind the back face of the footings.

5.4.1.9 Drainage

The Designer should study total drainage design. Adequate drainage of fill behind structures is important to increase the longevity of retaining structures. Water should not drain into the underside of slope protection. Drainage should be provided as follows:

- Where possible, french drains should be used at the back face of walls with 4 inch diameter drain pipes (weep holes) at nominal 10 foot maximum spacing through the walls . Refer to Standard Specification Section 512 – French Drains.

- Underdrains or other means may be used where necessary to provide adequate drainage.

5.4.1.10 Reinforcement and Structural Design

The structural design of abutments should comply with the requirements of AASHTO LRFD. Earth loads for structural design should be calculated per Section 3.4, Earth Loads, and an appropriate load factor applied.

Concrete cover for footing reinforcement should be as specified by AASHTO LRFD, except that for "non-designed" footings, such as for stub abutments, 6 inches of cover should be used.

At the back corners of gravity abutments and wingwalls, horizontal rebar should be placed, #6 bars at 12 inches on center, with lengths of 8 feet and with 6 inches of cover. Also, four #6 bars, 8 feet long, should be placed at 6 inches below bridge seat elevation at the front corners.

5.4.1.11 Abutments on Spread Footings

A. General

Refer to Section 5.3 Spread Footings for guidance on the design of spread footings.

The general design process for spread footing design should follow the steps below:

1. Determine the nominal and factored footing resistances at the service, strength and extreme limit states assuming footing dimensions and depth (consult Geotechnical Design Report)

2. Determine the loads applied to the footing, including lateral earth pressure loads for the abutment

3. Initially size and design the footing at the service limit state

4. Check the bearing pressure of the footing at the strength limit state

5. Check the eccentricity of the footing at the strength limit state

6. Check the sliding resistance of the footing at the strength limit state

7. Check the bearing pressure and eccentricity and sliding resistance of the footing at the extreme limit state

8. Check the footing bearing resistance at all limit states and overall stability in light any refined/new footing dimensions, depth and loads provided by the Designer.

9. Reassess steps 4 thru 7 based on the revised nominal and factored footing bearing resistance calculated

B. Spread Footings on Bedrock

Refer to Section 5.3.4.2 for guidance on the design of spread footings on bedrock.

C. Vertical and Horizontal Displacement

Vertical and horizontal movement criteria for abutments should be developed consistent with the function and type of structure, consequences of unacceptable movements on structure performance and the cost of mitigating movements and/or rotations by larger foundations. Angular distortions and settlements should be designed per Section 5.3.6 Settlement.

D. Global Stability

Global stability of slopes with abutments or walls should be considered part of the design of the wall or abutment. Evaluation of the global stability of an abutment is important when the abutment is located close to or on an inclined slope, or close to an embankment, excavation, or retaining wall.

The evaluation of the overall stability of earth or rock slopes with walls and abutments shall be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65. Refer to 5.3.7 Overall Stability for additional guidance.

E. Bearing Stress

Maximum bearing stress under footings at the strength limit load combination should be determined per Section 5.3.5 Bearing Resistance. Structures should be designed such that the calculated factored bearing stress under footings does not exceed the factored soil or rock bearing resistance in accordance with recommendations of the Geotechnical Designer. This requirement is expressed below:

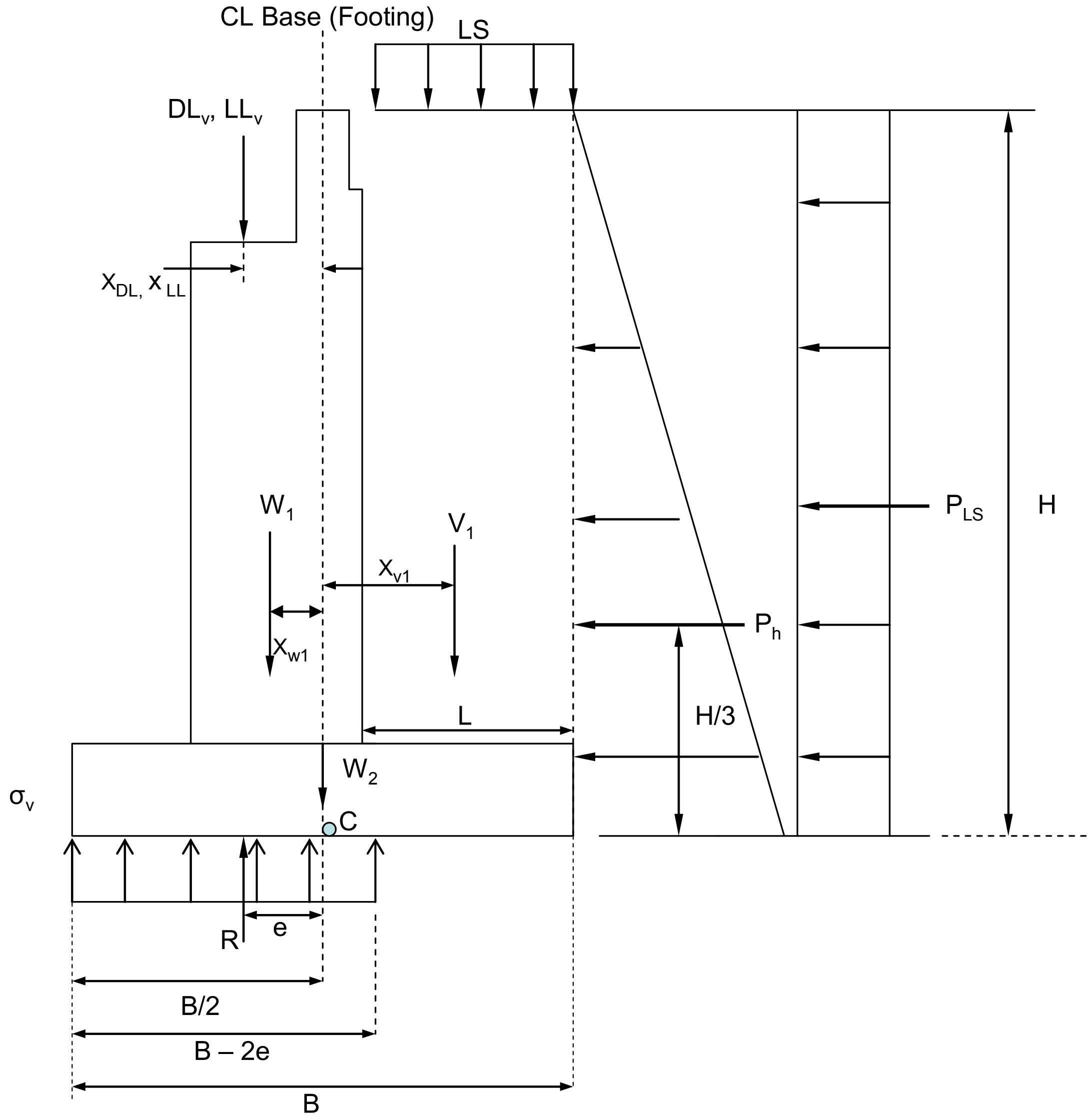
$$\sigma \leq \varphi R_n = R_f$$

where:

σ = factored vertical stress (ksf)
φ = bearing resistance factor (dim)
R_n = nominal bearing resistance (ksf)
R_f = factored bearing resistance (ksf)

The weight of the earth in front of a wall should be considered in computing maximum bearing pressure. When loads are eccentric, the effective footing dimension should be used for the overall dimension in the equation for bearing resistance. Refer to Procedure 5-1 and Procedure 5-2 for how to calculate applied bearing stress.

Procedure 5-1 Bearing Stress on Soil
For Wall or Conventional Abutment



Step 1. Calculate eccentricity, e_c, about point C, where:

M_o = sum of moments of factored overturning forces acting about point C:

$$M_o = P_h \cdot \frac{H}{3} + W_1 \cdot x_{w1} + P_{LS,h} \cdot \frac{H}{2} + DL_V \cdot x_{DL} + LL_v \cdot x_{LL}$$

M_r = sum of moments of factored resisting forces acting about Point C:

$$M_r = V_1 \cdot x_{V1} + LS \cdot L \cdot x_{V1}$$

$\sum V$ = sum of factored vertical forces acting on the footing and wall:

$$\sum V = V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L$$

and,

$$e_c = \frac{M_o - M_r}{\sum V}$$

$$e_c = \frac{P_H \cdot \frac{H}{3} - V_1 \cdot x_{V1} - LS \cdot L \cdot x_{V1} + W_1 \cdot x_{W1} + P_{LS,h} \cdot \frac{H}{2} + DL_v \cdot x_{DL} + LL_V \cdot x_{LL}}{V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L}$$

Step 2. The factored vertical stress should be calculated assuming a uniformly distributed pressure over an effective base area shown in the Figure above. The vertical stress should be calculated as follows:

$$\sigma_v = \frac{\sum V}{B - 2e_c}$$

Note that B-2ec is considered to be the effective footing width.

Step 3: Compare σ_v which already has the load factors included, to the factored bearing resistance of the soil, provided in the Geotechnical Report. The maximum factored stress should be less than the factored bearing resistance.

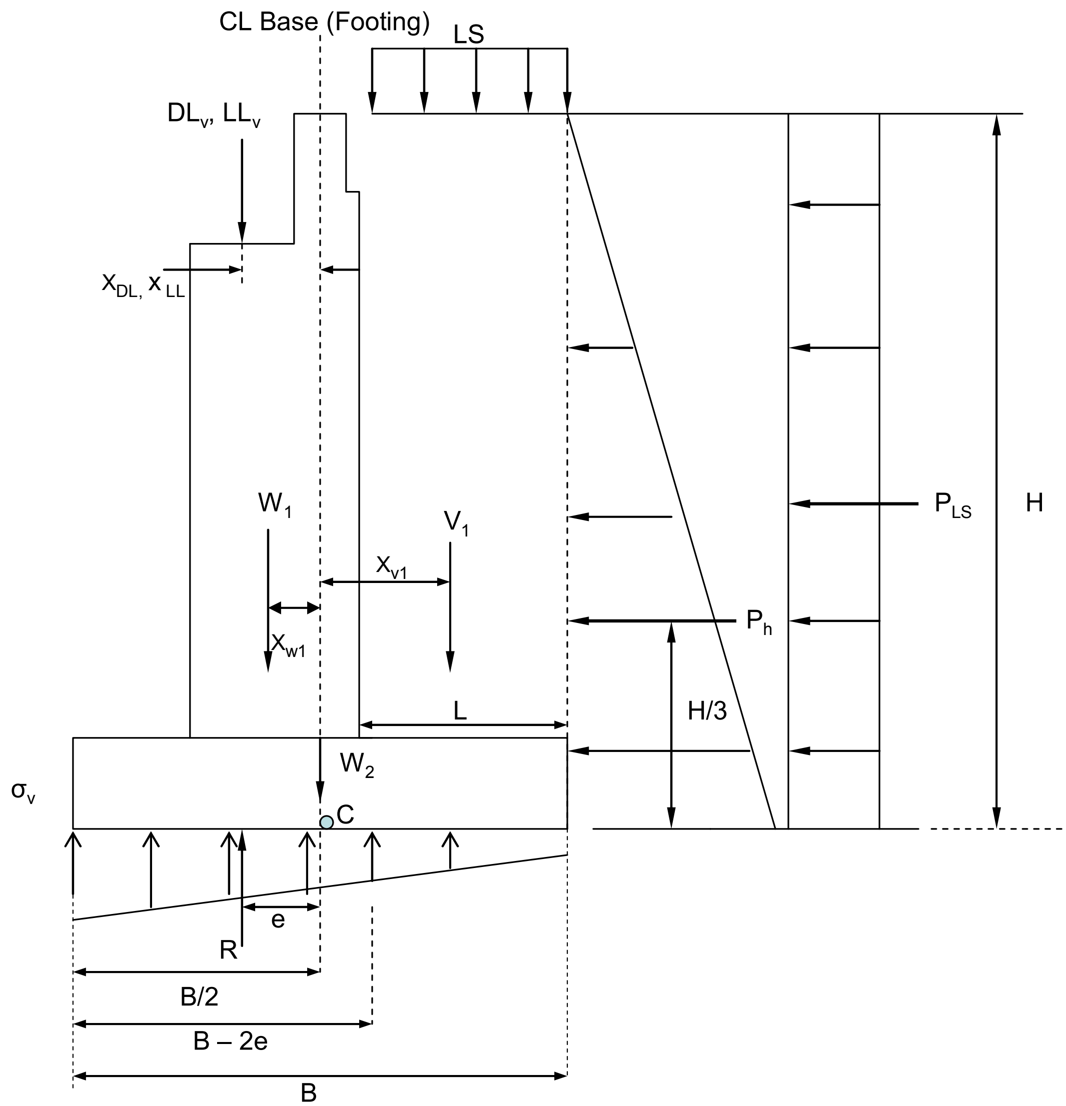
$$\sigma_v \le \varphi_{bc} \cdot R_n = R_f$$

where:

σ_v = factored vertical stress (ksf)
φ_{bc} = bearing resistance factor (dim)
R_n = nominal bearing resistance (ksf)
R_f = factored bearing resistance (ksf)

Note: The case shown for this procedure is the construction load with full backfill and live load surcharge on the approach, and superstructure dead load. For other load combinations, the appropriate loads must be included in the analysis.

Procedure 5-2 Bearing Stress on Bedrock
For Conventional Abutment



Step 1: Calculate the eccentricity about point C, e_c,, where:

M_o = sum of moments of factored overturning forces, acting about point C:

$$M_o = P_h \cdot \frac{H}{3} + W_1 \cdot X_{w1} + P_{LS,h} \cdot \frac{H}{2} + DL_V \cdot X_{DL} + LL_v \cdot X_{LL}$$

M_r = sum of moments of factored resisting forces about Point C:

$$M_r = V_1 \cdot X_{V1} + LS \cdot L \cdot X_{LS}$$

$\sum V$ = sum of factored vertical forces acting on the footing and wall:

$$\sum V = V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L$$

and,

$$e_c = \frac{M_o - M_r}{\sum V}$$

$$e_c = \frac{P_H \cdot \frac{H}{3} - V_1 \cdot X_{V!} - LS \cdot L \cdot X_{LS} + W_1 \cdot X_{W1} + P_{LS,h} \cdot \frac{H}{2}}{V_1 + W_1 + W_2 + DL_v + LL_v + LS \cdot L}$$

Step 2: The factored vertical stress should be calculated assuming a linearly distributed pressure over an effective base area shown in the figure above. If the resultant is within the middle 1/3 of the base, the maximum and minimum factored vertical stress is calculated as follows:

$$\sigma_{v\max} = \frac{\sum V}{B} \cdot \left(1 + 6 \cdot \frac{e_c}{B}\right)$$

$$\sigma_{v\min} = \frac{\sum V}{B} \cdot \left(1 - 6 \cdot \frac{e_c}{B}\right)$$

If the resultant is outside of the middle 1/3, of the base, i.e., if 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.

$$\sigma_{v\max} = \frac{2 \cdot \sum V}{3 \cdot \left(\frac{B}{2} - e_c\right)}$$

$$\sigma_{v\min} = 0$$

Step 3: Compare σ_{vmax} to the factored bearing resistance, $q_{r,}$ provided in the Geotechnical Report. The maximum factored bearing stress should be less that the factored bearing resistance.

$$\sigma_{v\max} \le \varphi_{bc} \cdot R_n = R_f$$

where:

σ_{vmax} = maximum factored vertical stress (ksf)
φ_{bc} = bearing resistance factor (dim)
R_n = nominal bearing resistance (ksf)
R_f = factored bearing resistance (ksf)

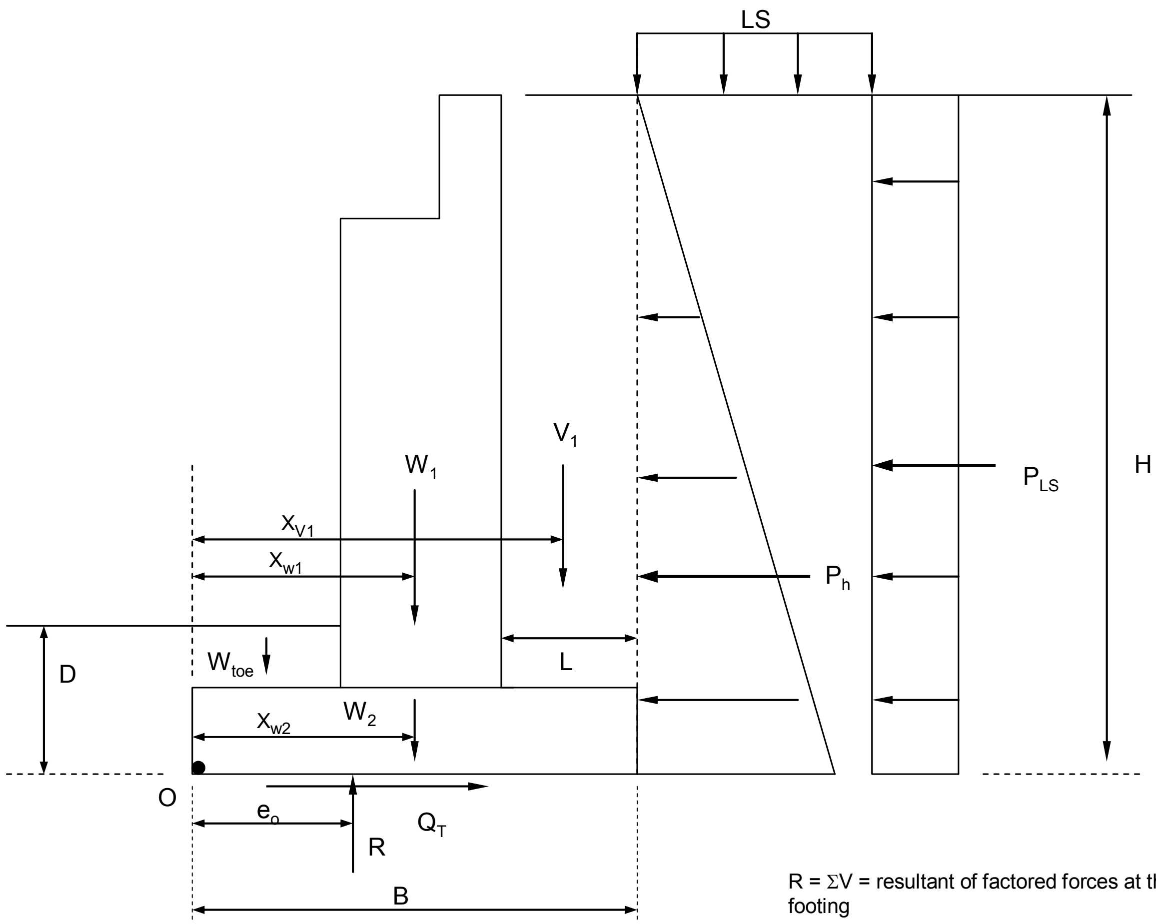
Note: The case shown for this procedure is the construction load with full backfill and live load surcharge on the approach. For other load combinations the appropriate loads must be included in the analysis.

F. Sliding

Failure by sliding should be investigated for all abutments founded on spread footings bearing on soil or bedrock. Passive earth pressure exerted by fill in front of the footing is neglected in consideration that soil may be removed during future construction. Refer to Section 3.6.9 Passive Earth Pressure Loads for guidance. The factored resistance against failure by sliding of abutments and walls on spread footings shall be calculated as described in Section 5.3.8 and LRFD 10.6.3.4. Resistance factors for sliding of spread footings at the strength limit state are provided in Table 5-3.

The coefficient of friction for sliding should be as shown in Table 3-3 for the appropriate soil type under the footing. For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding, based upon the bedrock characteristics.

Procedure 5-3 Eccentricity and Sliding Check for Conventional Abutment on Spread Footing



R = ΣV = resultant of factored forces at the base of footing

e_o = eccentricity of resultant, calculated around point O (toe of footing)

Step 1: Calculate the eccentricity about Point O in the Figure above to locate the resultant force R. Forces and moments resisting overturning are to be considered negative, and the maximum load factors should be used (Table 5-4)

M_o = sum of moments of factored overturning forces acting about Point O:

$$M_o = P_h \cdot \frac{H}{3} + P_{LS} \cdot \frac{H}{2}$$

M_r = sum of moments of factored resisting forces acting about Point O:

$$M_r = V_1 \cdot x_{V1} + W_1 \cdot x_{w1} + W_2 \cdot s_{w2}$$

$\sum V$ = sum of factored vertical forces acting on footing and wall, as defined in the Figure above.

$$\sum V = V_1 + W_1 + W_2$$

Step 1: Check eccentricity (overturning) about Point O:

$$e_o = \frac{M_o - M_r}{\sum V}$$

For footings on soil, the location of the resultant force shall be within the middle two-thirds (2/3) of the base width. For footings on bedrock, the location of the resultant force shall be within the middle nine-tenths (9/10) of the base width. For footings subjected to biaxial loading, these eccentricity requirements apply in both directions.

Step 2: Compare the factored resistance to sliding to the factored applied horizontal loads. The factored resistance to sliding should be greater than the factored applied horizontal loads:

$$R_n = \sum V \cdot \tan \delta$$

$$R_f = R_n \cdot \varphi_s > Q$$

where:

R_n = Nominal sliding resistance between soil and foundation (kips)
δ = friction angle between the footing base and the soil (refer to Table 3-3 or LRFD Table 3.11.5.3-1.)
Φ_s = resistance factor for shear resistance between the soil (or rock) and foundation
Q = factored horizontal applied loads

Note: The load combination shown for this strength limit state is Strength I-a, which does not consider superstructure dead loads (DC and DW) and vehicular live loads (LL) For other load combinations the appropriate loads and load factors must be included in the analysis.

G. Eccentricity

Abutments and walls on spread footings should be designed to resist overturning which results from lateral and eccentric vertical loads. The eccentricity should be evaluated as shown in Procedure 5-3. The location of the resultant of the reaction forces of at the strength limit state, based on factored loads, shall be within the middle two-thirds (2/3) of the footing width for footings on soil or the middle nine-tenths (9/10) of the footing width for footings on rock.

If construction loading is critical, the backfill height may be restricted until the superstructure or other parts are constructed.

5.4.1.12 Abutments Supported on Pile Foundations

Piles should be designed in accordance with the requirements of Section 5.7 Piles.

For pile supported abutments, the factored load combination causing the maximum and minimum compression in the piles should be determined, and the resulting pile reactions and pile stresses determined. The maximum factored axial pile load should not exceed the lesser of the factored geotechnical resistance and factored structural resistance for a single pile. In accordance with LRFD Article 6.5.4.2, the factored pile loads should not exceed the factored structural resistance using the resistance factors provided in 5.7.2 H-Piles and 5.7.5 Steel Pipe Piles. If greater loads result, more piles, or larger piles, should be considered.

For the Service Limit State, the unfactored lateral pile loads for H-piles should not exceed the lateral loads resistances specified in 5.7.2.2

Load combinations that do exceed the lateral load limits established for the service limit state should be evaluated by the Geotechnical Designer by means of a project-specific pile lateral load analysis using LPILE® software. The maximum lateral loads for all piles other than steel H-piles should be evaluated by the Geotechnical Designer. Buckling analyses of piles should be performed by the Structural Designer. Piles should also be checked for resistance against combined axial loads and flexure per LRFD 6.15 and 6.9.2.2. Pile resistance should be determined for compliance with the LRFD interaction equation.

Where abutments are required in water channels, the bottom of seal should be a minimum of 2 feet below the calculated scour depth from the check flood for scour. Where the calculated scour depth is significant, the Designer may consider designing the deep foundation elements for an unsupported length. The unsupported length should be the vertical distance from the bottom of the seal to the check flood scour depth. In designing deep foundation elements for an abutment with an unsupported length, a complete analysis of the foundation should be performed using actual loading and soil conditions.

Vertical and horizontal movement criteria for abutments supported by pile foundations should be developed consistent with the function and type of structure. The effect of lateral squeeze in the pile-supported abutments should be considered by the Geotechnical Designer, if applicable. Refer to Sandford, October 1994.

5.4.1.13 Bridge Seat Dimensions

As a minimum, the bridge seat dimensions should meet the requirements of LRFD Article 4.7.4.4. Otherwise, for bridge seats supporting steel superstructures exceeding 200 feet, use a minimum of 2 feet between the centerline of bearings and the face of breastwall and a minimum of 2'-3" between the centerline of bearings and the face of backwall. The masonry plate of the bearings should be no closer to the face of breastwall than 3 inches and should clear the face of backwall by at least 2 inches. For steel superstructures between 100 and 200 feet use a minimum 3 foot bridge seat. For steel superstructures less than 100 feet, the bridge seat dimensions should be large enough to accommodate the bearing masonry plate and the previous clearance dimensions. For major steel structures, all precast concrete structures, and structures with skews exceeding 45°, the bridge seat dimension should be determined based upon the project requirements.

All bridge seats, regardless if protected from roadway drainage by sealed bridge joints, should be concrete pedestal type with a minimum width along the centerline of bearing of 3 feet. The clear distance between the ends of bearing masonry plates and the ends of concrete pedestals should be at least 6 inches. The bridge seat between concrete pedestals should be sloped downward toward the face of breastwall at a slope of at least 15%.

Top of abutment backwalls should be 1'-6" wide, excluding the 6 inch approach slab seat, except when the concrete superstructure slab extends over the top of the backwall and the back of the backwall is battered. In that case, the backwall should be 1'-6" plus the effect of the batter.

5.4.2 Integral Abutments

5.4.2.1 Introduction

There are two categories of integral abutments: (1) full integral abutments, where the bridge beams are rigidly cast into an end diaphragm and (2) integral with hinge abutments, where butted boxes or voided slabs are connected to the abutment with dowels.

Integral abutment bridges (IABs) should be evaluated for use on all bridge replacement projects. MaineDOT most commonly uses 4 piles for each integral abutment substructure unit and traditionally uses the following piles:

- HP 10x42
- HP 12x53

- HP 14x73
- HP 14x89

Design is not limited to these piles. If the Structural Designer elects to use a pile not listed, the appropriate design analysis must be conducted.

Although HP 14 x 73 pile flanges are non-compact and do not meet the slenderness requirements of LRFD 6.9.4.2, Designers can account for pile slenderness in the design process, and this pile size should still be considered for pile supported integral abutments.

5.4.2.2 Loads

Analysis and design of integral abutment substructures will be in accordance with AASHTO LRFD, and include structural design and analysis of reinforced concrete abutments and wings, global stability of the channel slope with abutment, and pile design. Load combinations are presented in Section 5.4.1.2. Additional appropriate load combinations that investigate the effects of thermal gradients and abutment displacement may be required in accordance with LRFD Section 3.

5.4.2.3 Historical ASD Design Practice and Bridge Lengths

Commentary: Design of integral abutment bridges has evolved over the years as transportation departments have gained confidence with the system. Bridge lengths have gradually increased without a rational design approach. Tennessee, South Dakota, Missouri and several other states allow lengths in excess of 300 feet for steel structures and 600 feet for concrete structures.

Thermally-induced pile head translations in bridges with the lengths stated above will cause pile stresses which exceed the yield point. Research performed during the 1980's (Greimann, et. al.) resulted in a rational design method for integral abutment piles, which considers the inelastic redistribution of these thermally induced moments. This method is based upon the ability of steel piles to develop plastic hinges and undergo inelastic rotation without local buckling failure. This method is not recommended for concrete or timber piles, which have insufficient ductility.

Past practice was based on evaluation of the four steel piles most commonly used by MaineDOT and maximum bridge lengths and maximum design pile load design guides were developed based upon the Greimann research. The pile were evaluated as beam-columns without transverse loads between their ends, fixed at some depth and either pinned or fixed at their heads.

Greimann, et. al., developed design criteria by which the rotational demand placed upon the pile must not exceed the pile's inelastic rotational capacity. The following system variables affect the demand:

- Soil type
- Depth of overlying gravel layer
- Pile size
- Pile head fixity
- Skew
- Live load girder rotation

In order to simplify the design, past practice assumed that piles would be driven through a minimum of 10 feet of dense gravel. Material below this level has very little influence on pile column action. It was also assumed that the live load girder end rotation stresses induced in the pile head do not exceed 0.55 F_y (which provides a known live load rotational demand). Based upon the above assumptions and the pile's inelastic rotational capacity, the maximum pile head translation, Δ (in inches) was established for each of the four piles. Based on allowable stress design, the maximum bridge lengths historically were as follows:

- $MaxBridgeLength \cdot ft = \dfrac{4 \cdot \Delta \cdot in}{0.0125}$ for steel bridges
- $MaxBridgeLength \cdot ft = \dfrac{4 \cdot \Delta \cdot in}{0.075}$ for concrete bridges

Maximum bridge lengths vary from 70 feet to 500 feet for some piles. The past practice for maximum bridge lengths was 200 feet for steel and 330 feet for concrete. FHWA allows maximum bridge lengths of 300 feet for steel bridges, 500 feet for cast-in-place concrete bridges, and 600 feet for prestressed or post tensioned concrete bridges (FHWA Technical Advisory, January 28, 1990). Refer to BDG 5.4.2.6 for current bridge length limits.

5.4.2.4 Pile Design

A. Pile Loads

Piles should be modeled and evaluated as either fixed at the pile head for fully integral abutments (bridge beams are rigidly cast into an end diaphragm) or as pinned for integral abutments with hinge, such as the case when butted boxes or voided slabs have dowel connections to the abutment.

Piles for full integral and integral with hinge abutments shall be designed to resist all vertical superstructure dead and live loads, abutment and pile dead loads, live load girder rotation moments, lateral displacements, live load impact and moments caused by superimposed dead loads and live loads, as appropriate for the type of integral abutment.

Until the behavior of integral abutments with hinged connections to the superstructure is better understood, the pile design criteria for that type of integral abutment may assume that the moment at the top of the pile is zero, and that there is no moment from either the superstructure or earth loads.

The effect of thermal displacements and moments on piles can be investigated by running LPILE® software.

Secondary thermal forces only need be considered for multi-span structures only.

Appropriate load combinations and load factors should be determined per LRFD 3.4.1.

For the strength limit state analysis, design of the piles should consider the factored structural pile resistance, P_r, the factored structural flexural resistance, pile unbraced length, pile moments, the interaction of combined axial and flexural load effects, the structural shear resistance and the factored geotechnical resistance.

For service limit state evaluations, if piles will be driven to practical refusal in bedrock, settlement will not be a concern. However, all designs should consider horizontal movement, overall stability and scour for the design flood event.

B. Resistance Factors for Integral H-Piles

Pile will typically be end bearing on bedrock. For the strength limit state, use the following resistance factors:

- Use $\Phi_c = 0.50$ for axial resistance in compression and subject to severe pile driving condition; this condition should be assumed when analyzing the lower portions of the pile
- Use $\Phi_c = 0.60$ for axial resistance in compression under good driving conditions; this condition should be assumed when analyzing the upper portion of the pile
- For combined axial and flexural resistance in the upper zone of pile, use:

- Φ_c = 0.70 for axial resistance
- Φ_f = 1.00 for flexural resistance

C. Design Steps

The following steps should be followed during design of piles supporting full integral abutments, for the strength limit state:

1. Determine the foundation displacements, and the load effects (P_u and M_u) from the superstructure and substructure designs.

2. If applicable, determine the magnitude of scour.

3. Select preliminary pile size:

   a. Determine the factored applied superstructure vertical dead and live load (P_u) distributed to each pile

   b. Select the steel pile strength
   c. Select pile orientation; typically weak axis bending
   d. Determine resistance factors (Φ_c and Φ_f) for the structural strength in the upper and lower zones of the pile.
   e. Determine the maximum, required nominal axial pile resistance, P_u/Φ_f
   f. Estimate an initial pile area using the approximation

   $$A_s = \frac{Ru}{0.80 \times F_y}$$

   This approximation is based on weak axis bending and an assumed unbraced length of 15 feet based on typical integral abutment pile deflection and moment with depth curves. Select a pile size with an area A_s or greater.

4. Determine the pile unbraced length and maximum moment at the top of the pile by running LPILE® software for the design displacement from Step 1, P_u, and live load rotation

5. Determine if the applied moment on the pile will cause pile head plastic deformation by using the Interaction of combined axial and flexural load effects on a single pile (LRFD 6.9.2.2)

   a. Obtain the unbraced lengths of the top and lower segments of the pile and calculate the column slenderness factor (λ) for each segment. (LRFD 6.9.4.1)
   b. Determine K values for the top and bottom of the pile per LRFD Table C4.6.2.5-1

g. Calculate the nominal and factored structural pile resistance P_n, per LRFD 6.9.4.1 using the λ values
h. Compare the ratio of P_u to the structural resistance in the upper portion of the pile – the pile size should be such that the ratio is not less than 0.20.
i. Determine the nominal and factored flexural resistance about H-Pile weak axis, (LRFD 6.12.2.2)
j. Calculate the moment that will cause a plastic hinge at the top of the pile (M_p')
k. If the applied moment exceeds the moment that would cause a plastic hinge, a plastic hinge forms, and the moment that can be applied cannot exceed that moment (M_p')

6. For fixed head piles, run a second LPILE® analysis with displacement and plastic moment (M_p') as load conditions and P_u, and calculate new unbraced lengths from the moment with depth curve.

   a. Repeat steps 5.a. through 5.d., above
   b. If the pile size is such that the ratio of P_u to structural resistance exceeds 0.2, check the upper zone of the pile with the interaction equation of LRFD 6.9.2.2. If a plastic hinge forms at the top of the pile, the K value of the upper segment (that portion between the top of the pile and the first inflection point on the moment vs. depth curve) changes from 1.2, for a pinned condition, to 2.1, for a free condition at the top. With the new K value repeat Step 5, and check the interaction equation for pile overstress.

7. Because the piles have weak axis orientation and the flanges resist the shear as opposed to the web, check the maximum shear from the LPILE® output to the structural shear resistance per AISC G7.

8. Check that the maximum factored applied pile load does not exceed the factored geotechnical pile resistance or pile drivability resistance (LRFD 10.5.5.2.3 and 10.7.3.13) provided in the Geotechnical Design Report.

5.4.2.5 Pile Length Requirement

A. General Requirements

Piles may be end bearing or friction piles. In order to obtain the pile behavior associated with the equivalent length, piles should be installed 1 to 5 feet beyond the pile length required to achieve fixity. The practical depth to pile fixity is defined as the depth along the pile to the point of zero lateral deflection.

A minimum pile length of 10 feet is recommended, however soil conditions and loading conditions may require additional pile embedment to achieve fixity. Additional embedment length may be required for the use of friction piles. Also, axial loads may govern and additional embedment length may be required in order to achieve the factored design axial load with appropriate resistance factor applied. For pile lengths less than 14 feet, consideration should be given to the pile translating as a column and the pile tip walking. More vigorous driving shoes designed to properly seat piles and hold the pile and point in place are available. Refer to paragraph B. Short Pile Usage Guidelines, below.

If site-specific soil properties and loading conditions exist, an evaluation of minimum embedment length to achieve fixity using LPILE® software or the Davisson and Robinson equation in LRFD 10.7.4.2 is recommended. Consult the Geotechnical Designer for these analyses.

Piles should be driven with their weak axis perpendicular to the centerline of the beams, regardless of skew. Refer to Section 5.7.2 H-Piles for additional design requirements.

When scour is anticipated, the minimum pile length should be provided beyond the depth of computed scour.

B. Short Pile Usage Guidelines

The MaineDOT and the University of Maine at Orono (UMaine) have investigated the performance of integral abutment bridges at sites with shallow bedrock and have instrumented and monitored Nash Stream Bridge in Coplin Plantation, Maine, (Hartt, et. al., 2006 and Delano, et. al., 2005)). Preliminary evaluation of the field data from the research study indicate that integral abutment bridges with ‘short’ steel piles (14 feet or less) may not develop fixity but perform adequately and do not experience stresses larger than those seen by longer piles. The shortest pile instrumented by the researchers was a 14-foot long H-pile.

To accommodate integral abutment piles at sites with shallow bedrock, the following design features are recommended:

- In consideration of (a) the consequences of scour and pile exposure, (b) the need to limit pile tip movement, and (c) obtaining pile behavior associated plastic stress redistribution and inelastic rotation in the pile, a minimum pile length of 10 feet is recommended. This recommendation is based on finite element analyses and limited field data from the UMaine studies (Delano, et. al. 2005 and Hartt, et. al. 2006). If the depth to bedrock is so shallow that 10 feet of embedment in soil cannot be achieved, piles should be installed in bedrock sockets to provide the minimum 10-foot pile length recommended. If a fixed condition at the pile tip is desired, the bottom 6-inches of the rock sockets should be tremie-filled with concrete. However, the UMaine research indicates some rotation at the pile tip is acceptable.

- o Short piles supporting integral abutments should be designed in accordance AASHTO LRFD criteria and checked for pile tip movement by conducting a LPILE® analysis, or as described in the design example found in Appendix B of Technical Report ME 01-7 (Delano, et. al. 2005), and Chapter 5 of that report. Achievement of an assumed pinned condition at the pile tip should also be confirmed with an LPILE® analysis.

- o Since the abutment piles will be subjected to lateral loading, the piles should be analyzed for combined axial compression and flexure resistance as prescribe in LRFD Articles 6.9.2.2 and 6.15.2 and checked for compliance with the interaction equation. An LPILE® analysis is recommended to evaluate the soil-pile interaction with factored axial loads, moments and pile head displacements applied.

- o Driven piles should be fitted with special driving points to improve penetration into bedrock and improve friction at the pile tip to support a pinned pile tip assumption.

- o The stream velocity should be low and there should be low potential for removal of any dams, scour action, wave action, storm surge and ice damage. This is to ensure the long-term integrity of the bridge approach fills and riprap abutment slopes, which provide the only lateral support to pile groups.

- o Minimum 1.75H:1V slopes in front of integral abutment pile groups should be protected with riprap over an erosion control geotextile or concrete slope protection.

5.4.2.6 Maximum Bridge Lengths

The criteria for the maximum bridge lengths provided in Table 5-5 are based on the following assumptions:

- Steel H-piles are used with their webs oriented normal to the centerline of the bridge (longitudinal translation about the weak axis).

- The piles are driven through gravels or through clays with a minimum of 10 feet of gravel overburden.

- For skews greater than 20°, abutment heights are <12 feet and pile spacing is < 10 feet.

- Total thermal movement is 1-1/4”/100 feet bridge length for steel structures and 3/4”/100 feet bridge length for concrete structures (FHWA Technical Advisory, January 28, 1990).

- Factored pile loads do not exceed the factored compressive structural pile resistance, the factored flexural pile strength and the factored geotechnical and drivability resistance of the pile section.

- Steel H-Piles are made of Grade 50 steel.

Bridge lengths in excess of the limitations below may be used with the approval of the Engineer of Design when special design features are provided.

Table 5-5 Recommended Maximum Lengths for Fully Integral Abutment Bridges (feet)

	Skew ≤ 20°	
Pile Size	**Steel**	**Concrete**
Piles per 5.4.2.1 with fully fixed heads	300	500

5.4.2.7 ‘Best Practices’ for Moderate to Long Span IABs

The following ‘best practices’ should be considered as design features for moderate to long span integral bridges, defined as integral steel bridges longer than 200 feet and concrete bridges longer than 330 feet:

- Only straight stringers/beams should be used on long span IABs.

- The annual thermal cyclic movement of the IAB abutments results in the development of a settlement trough adjacent to each abutment as backfill soil slumps downward and toward the abutment in its winter position. To prevent the settlement of the pavement structure, approach slabs must be included in the design of moderate to long span IAB structures, to span over the void created by the settled soil.

- Provide 2 layers of polyethylene sheets, or other bond breaker, under the approach slab to minimize friction against horizontal movement. Many States recommend two layers of 4 to 6 mil thick polyethylene sheets.

- Consider pavement expansion joints to reduce distress of the approach pavement, caused by the thermal cyclic movement of the abutments and the approach slabs. Recommended cycle control joints systems that employ a combination of asphaltic plugs, asphalt impregnated fiber board, and sleeper slabs at the end of the at-grade approach slabs, or at the end of the abutment (in the case where the slab is buried).

- A bridge with a total length in excess of 300 feet will have larger movement demands. If the anticipated abutment movements are in excess of 1.0 inch, consider strong axis pile orientation to prevent a plastic hinge under weak axis bending.

- Approach slabs should also be positively attached to the abutment to prevent slabs from “walking off” corbels during annual thermal movements of the abutment.

- Pavement geotextiles can be used to add tensile strength to pavement over the abutment backwall.

- Provide adequate drainage of the abutment backfill to prevent damage due to frost action and piping of the backfill material.

- Bridge abutments with movements in excess of 1 inch may require a higher level of pile analysis to consider all applicable forces and moment demands, including thermal, skew effects and deflections of the superstructure. A dedicated check of pile capacity for combined axial loading due to dead and live load and bending stresses due to thermal superstructure movement, using LPILE® software may be required.

- Pre-auger to a depth of 10 feet for the top portion of piles and then fill the hole with a non-compacting backfill material, such as underdrain backfill Type C. This creates a hinge effect in the substructure and has the effect of reducing the lateral soil stiffness by increasing the depth to fixity and reducing bending moment stress in the pile.

- Long-span integral bridges receive significant support from the embankments, and therefore, they only should be built in conjunction with stable approach embankment foundation soils.

- To mitigate excessive earth pressures, limit abutment heights.

- Avoid abutments of differing height; as such a practice may promote unequal movements at the two abutments.

- Select a span arrangement and bearing types that result in approximately equal movements at each abutment.

- As a result of the soil movement, the summer lateral earth pressures tend to increase over time as the soil immediately adjacent to each abutment becomes increasingly wedged in. This phenomenon of soil wedging and long-term buildup of lateral earth pressures is referred to as “ratcheting”. To avoid potential problems, abutments should be designed for full passive pressure using Coulomb Theory.

- Limit the use of long span integral abutments to bridges with skews less than 20 degrees to minimize the magnitude and lateral eccentricity of potential longitudinal forces.

- Make wingwalls as small as practical to minimize the amount of structure and earth that have to move with the abutment.

- Configure wingwalls to minimize resistance to abutment movement.

5.4.2.8 Abutment Details

Typical integral abutment details for steel and concrete superstructures are shown in Figure 5-2 and Figure 5-3, respectively. For steel superstructures, fixed head integral abutments are preferred but pinned head abutments are allowed.

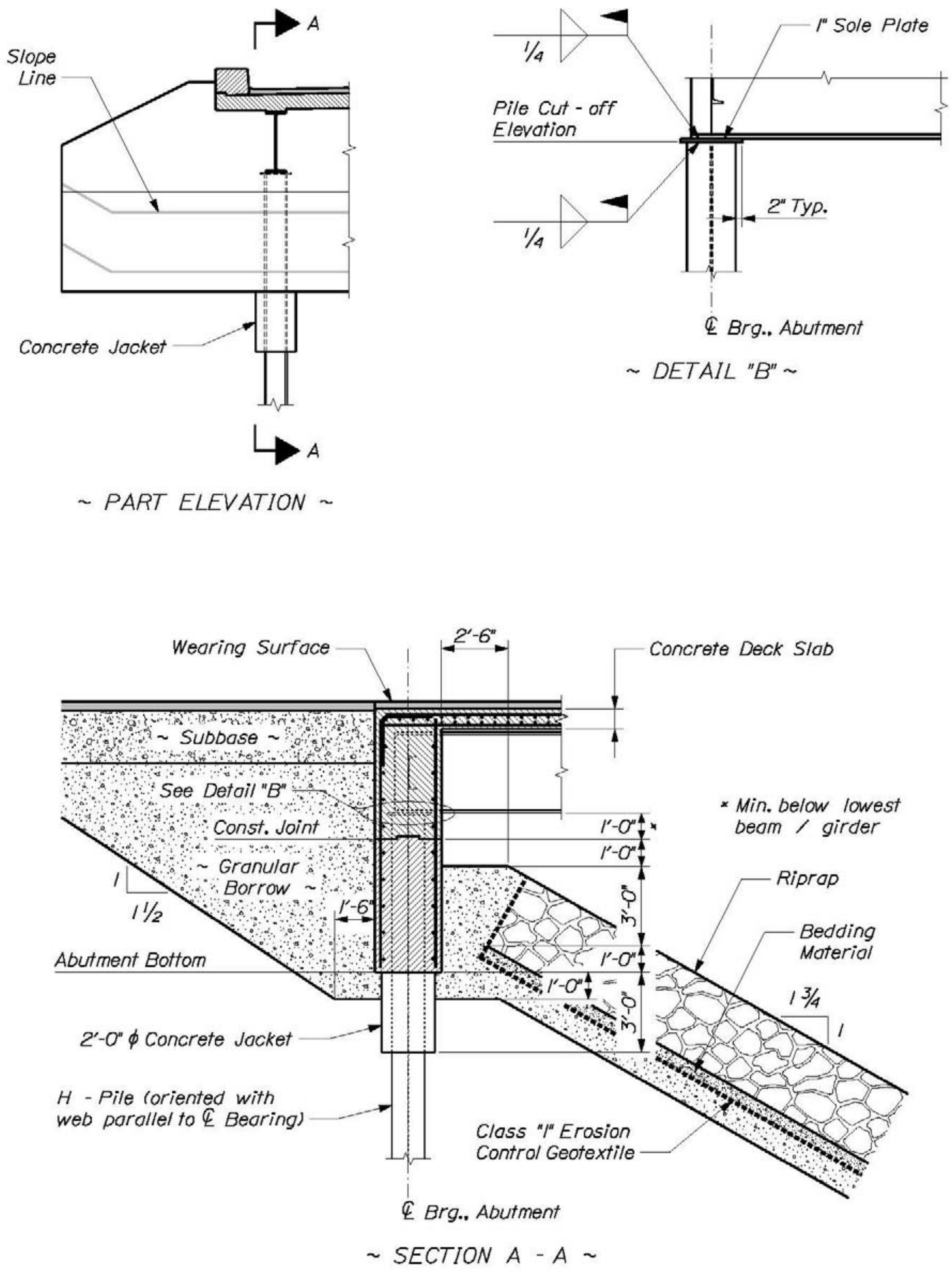
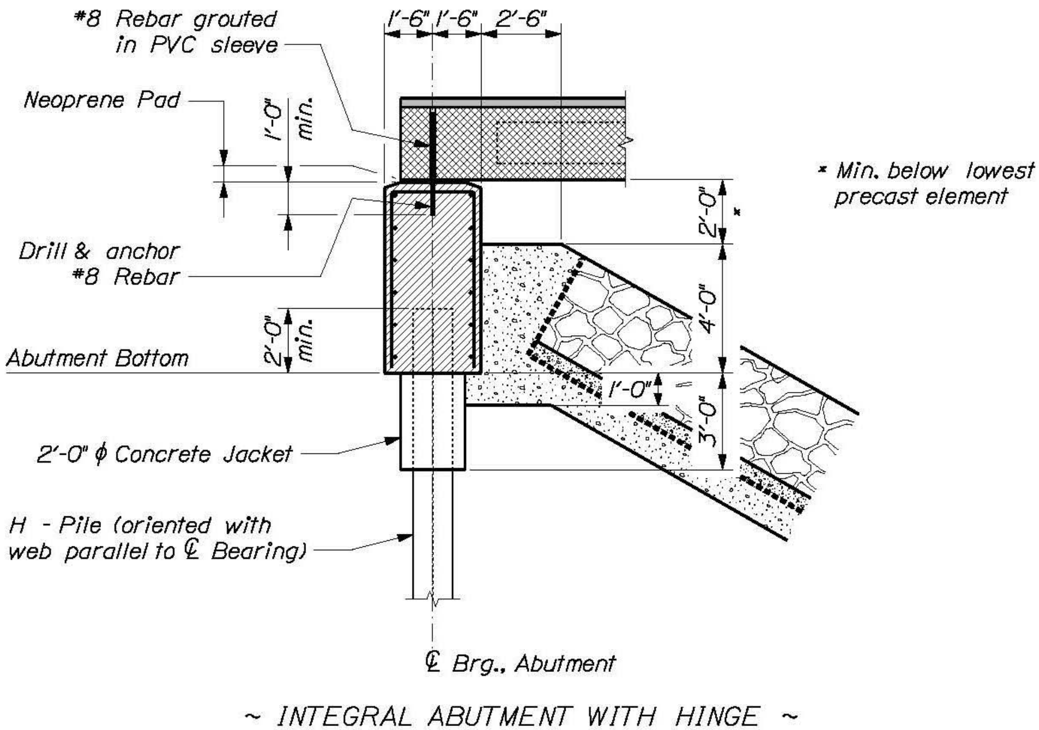


Figure 5-2 Fixed Pile Head, Full Integral Abutment Details-Steel Superstructures



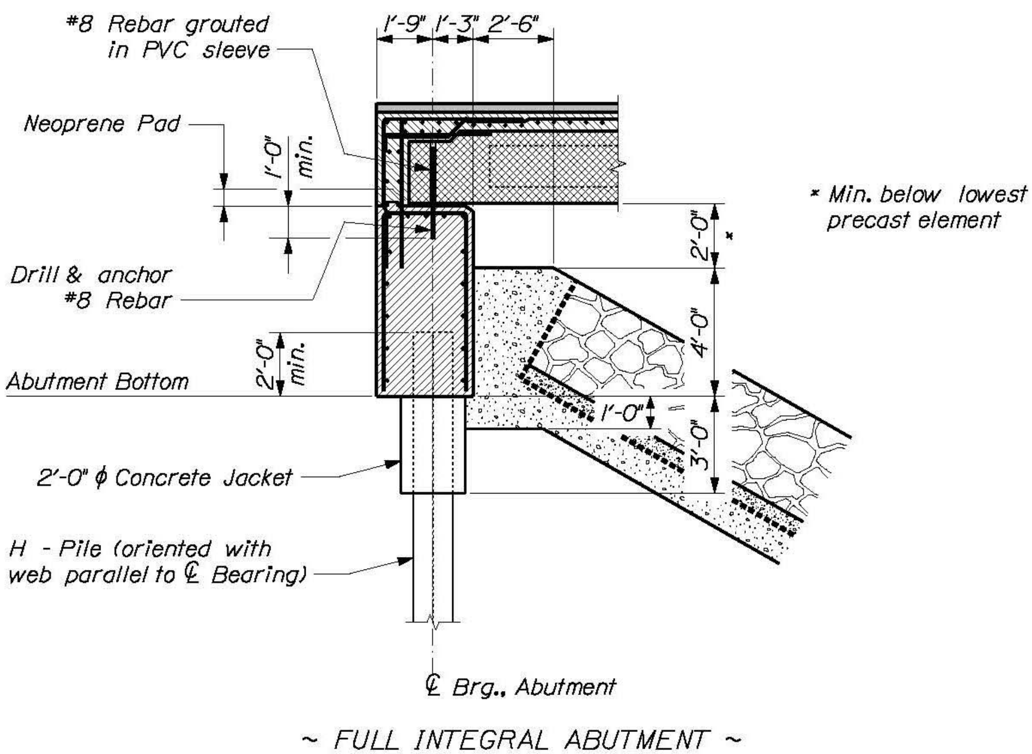


Figure 5-3 Integral Abutment with Hinge and Full Integral Abutment Details – Precast Superstructures

5.4.2.9 Alignment

Curved bridges are allowed, provided the stringers are straight. Beams should be parallel to each other. All substructure units should be parallel to each other.

The maximum vertical grade between abutments is limited to 5%.

5.4.2.10 Superstructure Design

No special considerations should be made for integral abutment designs. Fixity at the abutments should not be considered during beam/girder design.

When selecting span ratios for multi-span bridges, consideration should be given to providing nearly equal movement at each abutment.

5.4.2.11 Abutment and Wingwall Design

Design abutment and wingwall reinforcement for the passive earth pressure (P_p) which results on the back face of the wall when the bridge expands. Refer to Section 3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient (K_p) and Table 5-4 for the passive earth pressure load factor (γ_{EH}).

Design bars for the backwall for full passive pressure due to the abutment backfill material. The backwall acts as a continuous horizontal beam supported on the piles, i.e., with spans equal to the girder spacing. Design the bars for 1) the maximum factored shear due to the factored passive earth pressure and, 2) flexure due to the moment from the factored passive soil pressure Determine the passive pressure P_p acting on the full height of the abutment backwall (H_{abut}) from the bottom of the approach slab to the bottom of the abutment/pile cap. The passive pressure acts in a triangular pressure distribution:

$$P_p = \frac{1}{2} \cdot \gamma_{soil} \cdot H_{abut}{}^2 \cdot k_p$$

Design for a factored moment equal to:

$$M_{up} = \gamma_{EH} \cdot \frac{P_p \cdot l^2}{8}$$

A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, γ_{EH} = 1.50.

Design the abutment wall top and bottom horizontal bars for vertical loads, considering the wall to be a continuous beam with piles as supports.

Wingwalls should preferably be straight, cantilevered extension wings not to exceed 10 feet in length. Design wingwall reinforcement for the passive earth pressure (P_p) which results on the back face of the wall when the bridge expands, using the Coulomb passive earth pressure state and a passive earth pressure load factor (γ_{EH}) of 1.5. The use of flared wingwalls may be considered at stream crossings where the alignment of the stream would make in-line walls subject to scour. Piles should never be placed under wingwalls that are integral with the abutment stem. Generally the design is controlled by the horizontal bending in the wingwall at the fascia stringer caused by large passive pressures bending the wingwall.

Because of the high bending moments due to passive pressure in wingwalls 10 feet or longer, it may be necessary to support longer wingwalls on their own foundations, independent of the abutments. A flexible joint must be provided between the wingwalls and the backwall. U-wingwalls cantilevered from the abutment stem should only be considered to address right-of-way or wetlands encroachment. U-wingwalls should be no longer that 10 feet and tapered to reduce earth pressures. If an approach slab must extend to a U-wingwall, use a 2 inch joint with filler to separate the slab and the wall.

Developing full passive earth pressure requires that wall rotation, i.e. the ratio of lateral abutment movement to abutment height (y/H), exceeds 0.005. If the calculated rotation is significantly less than that required to develop full passive pressure, the Designer may consider using the Rankine passive earth pressure case, which assumes no wall friction. For the passive earth pressure case, wall friction acts downward against the passive wedge and increases passive pressure in the Coulomb state.

5.4.2.12 Approach Slabs

In addition to the requirements of Section 5.4.4, approach slabs should be used when integral bridge lengths exceed 80 feet for steel structures and 140 feet for concrete structures.

Provisions for movement between the approach slab and approach pavement is not necessary until bridge lengths exceed 140 feet for steel structures and 230 feet for concrete structures. Approach slabs below grade should be attached to the abutment. For at grade approach slabs, consideration should be given to the installation of an expansion device between the approach slab and the abutment. Refer to recommendations for approach slabs for moderate to long span integral bridges in Section 5.4.2.7.

5.4.2.13 Drainage

The area behind integral abutments should be backfilled with granular borrow for underwater backfill. A proper drainage system as described in Section 5.4.1.9 should 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. In these situations, consideration should also be given to backfilling integral abutments with gravel borrow or aggregate subbase course - gravel.

5.4.2.14 Scour

The Designer should ensure the stability of the structure for anticipated scour, as defined by LFRD 2.3.11. This may require driving the piles deeper than what is required by geotechnical criteria. The minimum pile length should be provided beyond the depth of computed scour for the check flood for scour.

5.4.2.15 Integral Abutment on Spread Footing Design

Spread footing abutments may be used only if designed and detailed as a semi-integral bridge abutment. Refer to Section 5.4.3 Semi-Integral Abutments.

5.4.3 Semi-Integral Abutments

A semi-integral bridge is defined as a “single span or multiple span continuous deck-type bridge with rigid non-integral abutment foundations, and with a movement system composed primarily of reinforced concrete end-diaphragms, backfill, approach slabs, and movable bearings located in horizontal joints at the superstructure/abutment interface” (TRB, 1996).

A semi-integral abutment bridge is characterized by:

- Elimination of expansion joints in the deck and roadway
- The superstructure backwall (end diaphragm) is not connected to the abutment, but moves along a bearing and horizontal joint below ground
- Thermal movement is accommodated by expansion bearings and a small vertical gap between the end diaphragm and the abutment

- The abutments are typically supported on spread footings or multiple rows of piles

Semi-integral abutments should typically be designed for active earth pressure over the rigid abutment height and a uniform pressure distribution due to the height of soil behind the superstructure. The superstructure backwall should typically be designed for full passive pressure only. In designing for active pressure, a Rankine active earth pressure coefficient, K_a, is recommended.

Semi-integral bridge design is still considered experimental, and must receive approval from the Engineer of Design during the preliminary design phase as a design exception.

Research findings have resulted in TRB design recommendations that include the following:

- Utilization of attached approach slabs and return wingwalls to lock the superstructure into the backfill
- Deliberate construction of an air space below the end diaphragms to prohibit an undesirable shift in the end reaction location

5.4.4 Approach Slabs

Approach slabs should be used on collectors and arterials, where:

- the design hour volume (DHV) is greater than 200,
- abutment heights (bottom of footing to finish grade) are greater than 20 feet, or,
- poor soil conditions are encountered and settlement is anticipated in the vicinity of the abutment.

Additional requirements for the use of approach slabs on integral abutment bridges are provided in Section 5.4.12.

Approach slab seats should be 6 inches wide and specified to have a roughened surface. Approach slab seat dowels should not be used except on integral abutments as discussed in Section 5.2.4.12. Approach slab seats should be a minimum vertical distance of 2’-9” from the roadway surface. If the backwall is very high, the Structural Designer may elect to make an optional horizontal construction joint at the approach slab seat elevation.

When a structural approach slab is specified, reduction, but not elimination, of the vehicle surcharge loads may be considered per LRFD 3.11.6.5.

5.5 Piers

5.5.1 Mass Piers

Mass piers are intermediate vertical supports, which extend from the foundation, either a spread footing or deep foundation, to a pier cap, which supports the superstructure. The primary functions of pier are:

- Support dead loads, live loads and other loads from the superstructure
- Support its own weight and other loads acting directly on the pier
- Transmit all loads to the underlying foundation

The connection between the pier and the superstructure may be pinned, fixed, or free. Mass piers are typically constructed from reinforced concrete, but may be precast. Mass piers may consist of gravity, solid wall, single-column, or multiple-column piers. Single-column and multiple-column piers are usually designed in a “hammerhead” configuration at the pier cap.

5.5.1.1 Pier Selection Criteria

Selection of the mass pier configuration is based on the following factors:

- Loading conditions
- Skew
- Slenderness, with respect to buckling
- Aesthetics
- Likelihood of debris. The use of multiple-column piers in areas where floating debris may lodge between columns should be avoided.

5.5.1.2 Load Combinations and Load Factors

Mass piers should be designed in accordance with <u>AASHTO LRFD</u>, including, structural design of reinforced concrete and geotechnical analysis and design, such as bearing capacity, sliding, and eccentricity (overturning). Piers should be designed and proportioned to resist all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5 and as outlined in Chapter 3.

The following load combinations should be considered as a minimum for geotechnical analysis:

- Strength I-construction. Strength Limit State I with the exception that bridge superstructure, or a portion of that, and vehicle live loads are neglected. Any anticipated staged construction loading should be investigated. Load factors for the dead load or other components shall not be less than 1.25. Live load surcharge is included to account for construction equipment live loading during structure erection, and a construction load factor of 1.5 should be assumed.

- Strength I: Normal vehicular use without wind: dead load, all applicable live load combinations, impact; braking force (for one and two lanes) centrifugal forces, static water pressure, buoyancy and stream pressure. For Strength 1, the minimum and maximum permanent load factors are selected to create the greatest force and moment effects for the mode of stability being investigated. .

- Strength III: Load combination relating to high wind velocity (100 mph) without vehicular live load: dead load, earth pressure, if applicable; buoyancy; stream flow pressure; wind; wind on live load; and longitudinal force from thermal displacements. Minimum and maximum load factors for permanent loads (γ_p) are selected to produce the extreme force or moment effect for sliding, eccentricity or axial loading analyses.

- Strength IV: Load combination relating to very high dead load to live load force effect ratios exceeding about 7.0. Minimum and maximum load factors for permanent loads (γ_p) are be selected to produce the extreme force or moment effect for sliding, eccentricity or axial loading analyses.

- Strength V: Load combination relating to the bridge exposed to 55 mph wind velocity with live loads: dead load; live load plus impact; centrifugal force; earth pressure; buoyancy and stream flow pressure. . Minimum and maximum load factors for permanent loads (γ_p) should be selected to produce the extreme force or moment effect for sliding, eccentricity or axial loading analyses.

- Service I: Normal vehicular use of the bridge with a 55 mph wind load. All loads are taken at their unfactored values.

Debris loading shall be accounted for in water pressure loads by a 25% increase in the exposed surface area of the pier.

A Maine-modified Strength Limit State analysis should be performed that includes in the ice pressures of past practice, specified in Section 3.9 Ice Loads, with the appropriate resistance factors applied to the pier components. The Strength Limit State that produces the extreme force or moment should be selected.

Where piers are to be designed to resist earthquake forces, collisions by roadway or rail vehicles, vessel collision or ice, the structures should be evaluated for the following additional limit states:

- Extreme Event I – Load combination including earthquake forces, using permanent load factors, γ_p, which produce the greatest load and moment effects for the mode of stability being analyzed.
- Extreme Event II – Dead load; live load; buoyancy; static water pressure; stream flow pressure; ice pressure; vessel impact and vehicle or railway impact, using permanent load factors, γ_p, which produce the greatest load and moment effects for the mode of stability being analyzed.

For Extreme Event II apply ice force effects, and vessel, vehicle and railway collision forces one at a time since the joint probability of these events is extremely low.

The ice pressures for Extreme Event II shall be applied at Q1.1 and Q50 elevations as defined in Section 3.9 Ice Loads with the design ice thickness increased by 1 foot and a load factor of 1.0.

The critical load conditions for the evaluation of foundation bearing capacity, overturning (for pile foundations assess uplift loading of piles), eccentricity, and sliding (lateral loading for deep foundations) are those combinations of minimum or maximum loads and moments which produces the maximum force or moment effect.

With regards to vehicular live load (LL and IM) lane placement is important and multiple presence factors (MPF) are applicable. Impact forces should only be applied to truck or tandem loads:

- IM = 0.33 for cap and stem
- IM = 0 for buried footings

In consideration of the potential deflections due to bending of a pier about its weak (transverse) axis may result in magnification of the longitudinal moments on the pier, the Designer should compute longitudinal moment magnification factors for each load combination and Strength Limit State based on the factored loads and pier stiffness. The Moment Magnifications Factors are provided in LRFD 4.5.3.2.2.

5.5.1.3 General

The designer should estimate the load combinations which could be imposed on the pier and estimate the nominal resistance of the structural component or ground. Pier components shall satisfy the following equation for each limit state:

$$\sum \eta_i \gamma_i Q_i \leq \Phi R_n = R_f$$

where:

η_i = Factors to account for ductility, redundancy and operational importance
γ_i = Load factor (dim)
Q_i = Force effect or stress (kip)
Φ = Resistance factor (dim)
R_n = Nominal resistance (kip)
R_f = Factored resistance (kip)

5.5.1.4 Strength Limit State Evaluations

The above equation should be used to evaluate piers and pier foundations at the strength limit states for:

- Bearing resistance failure
- Lateral sliding
- Excessive loss of base contact (eccentricity)
- Pile group failure
- Structural failure

The factored resistance, R_f, calculated for each mode of failure, is to be calculated using the appropriate resistance factors for bearing resistance, sliding, eccentricity, axial pile resistance and structural resistance.

The Designer should consider the consequences of changes in the pier foundation conditions from scour due to the design flood event using appropriate strength limit state resistance factors. Debris loading during flood events should be accounted for in water pressure loads by assuming a 25% increase in the exposed surface area of the pier.

The investigation of piers at the strength limit states for structural failure should be in accordance with LRFD Article 5.7 and carry all flexure and axial loads anticipated. Appropriate consideration should be given to the effects of slenderness on both aesthetics and load-carrying capacity.

For piers founded on piles, the shear on the critical section should be investigated at the strength limit state in accordance with AASHTO LRFD Section 5.13.3.6.

5.5.1.5 Service Limit State Evaluations

Piers should be investigated at the service limit state for:

- Settlement
- Lateral displacement
- Overall slope stability
- Foundation stability, settlement and horizontal movement at the design flood for scour

A resistance factor, φ, of 1.0 is used to assess pier design at the service limit state. The overall global stability of the foundation should be investigated at the Service Load Combination with a resistance factor, φ, of 0.65.

Tolerable vertical and lateral displacement criteria for piers shall be developed based on the function and type of pier, anticipated service life, and consequences of unacceptable movements of the pier and effect on the superstructure and bearings.

5.5.1.6 Extreme Event Limit State Evaluations

Extreme event limit state design checks for piers should include:

- Bearing resistance
- Eccentricity
- Sliding
- Overall stability
- Pile group failure
- Structural failure

A resistance factor, φ, of 1.0 is used in the load and resistance equation in Section 5.4.1.3 to assess pier design at the extreme limit state.

Resistance factors for extreme event limit states shall be taken as 1.0.

For the extreme event limit state, the Designer should consider scour due to the check flood event and should determine that there is adequate foundation resistance to support all applicable unfactored loads with a resistance factor of 1.0 or less. Debris loading during flood events should be accounted for in water pressure loads by a 25% increase in the exposed surface area of the pier.

5.5.1.7 Structural Design

The structural design of piers shall be in accordance with the provisions of LRFD Sections 5, 6, 7, and 8, as appropriate.

The investigation of piers at the strength limit states for structural failure should be in accordance with LRFD 5.7 and carry all flexure and axial loads anticipated. Appropriate consideration should be given to the effects of slenderness on both aesthetics and load-carrying capacity.

For piers founded on piles, the shear on the critical section should be investigated at the strength limit state in accordance with AASHTO LRFD 5.13.3.6.

5.5.1.8 Structural Design of Columns

The primary checks for a pier shaft or column structural design consist of:

- Determine maximum moments and shears in the shaft/column
- Check limits for reinforcement (LRFD 5.7.4.2)
- Calculate the factored axial resistance (LRFD 5.7.4.4)
- Check slenderness provisions for compression members (5.7.4.3)
- Calculate the moment magnification factors (LRFD 4.5.3.2.2b) Develop shaft or column interaction curve
- Check biaxial flexure provisions for non-circular members (LRFD 5.7.4.5)
- Determine transverse reinforcement for compression members (LRFD 5.10.6 or 5.7.4.6)

5.5.1.9 Geotechnical Design of Pier Foundations

A. Spread Footings

In using spread footings for foundation support for mass piers, either on soil or bedrock, the design should be in accordance with the AASHTO LRFD and Section 5.3 Spread Footings.

B. Deep Foundations

Deep foundations for mass piers may consist of piles or drilled shafts. Piles may consist of H- or pipe pile steel sections, or precast concrete. In founding a mass pier on a deep foundation, design should be in accordance with the AASHTO LRFD, and BDG Sections 5.7 Piles and 5.8 Drilled Shafts. In designing deep foundation elements for a mass pier with an unsupported length, a complete analysis of the foundation should be performed using actual loading and soil conditions.

For strength and extreme limit state analyses, maximum factored axial pile loads and stresses should be computed using 3-D pile group analysis software, such as FB-Multipier®.

For service limit state design of deep foundation, a complete deflection analysis of a driven pile foundations should be performed using LPILE® or FB-Multipier® software.

C. Scour

For scour protection of mass piers in water channels, the following treatments should be considered: 1) the use of a deep seal placed minimum of 2 feet below the scour depth determined for the check flood for scour, or 2) designing the deep foundation elements for an unsupported length. The unsupported pile length should be the vertical distance from the bottom of the seal to the scour depth determined for the check flood event. Piles should achieve axial capacity and lateral capacity/fixity below the scour depth determined for the design flood event.

5.5.1.10 Pier Protection

A. Collision Forces

Where the possibility of collision exists from vehicular, railroad, or water traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system.

Unless the department determines that site conditions indicate otherwise, or unless protected by collision walls as specified in paragraph B. below, piers located within a distance of 30 feet to the edge of roadway or within a distance of 50 feet to the centerline of a railway track shall be designed for an equivalent static force of 400 kips, which is assumed to act in any direction in a horizontal plane, normal to the wall, at a distance of 4 feet above the ground.

B. Collision Walls

The provisions of the paragraph above need not be considered for piers or abutments protected by an:

- An embankment
- A structurally independent crashworthy ground mounted 54 inch high barrier, located within10 feet of the pier, or
- A 42 inch high barrier located at more than 10 feet from the pier

C. Vessel Collision

All bridge components in navigable waterway crossings where vessel collision is anticipated shall be designed for a specified degree of vessel impact damage in accordance with LRFD 3.14, or adequately protected by dolphins, fender systems or other sacrificial devices.

D. Scour

The majority of bridge failures in the United States are the result of scour. The added cost of making a bridge less vulnerable to scour is small in comparison to the total cost of a bridge failure.

LRFD 3.7.5 requires that scour at bridge piers be investigated for two conditions:

- For the design flood for scour, the streambed material above the total scour line shall be assumed to have been removed. The design flood storm event shall be the more severe of the 100-year event or from an overtopping flood of lesser recurrence interval. The strength and service limit states apply.
- For the check flood for scour, the stability of pier foundations shall be investigated for scour conditions resulting from a designated flood event, not to exceed the 500-year event or from an overtopping flood of lesser recurrence. The extreme event limit state shall apply. Reserve capacity beyond that required for stability under this condition is not necessary. The exception is

spread footings on soil or erodible rock, which shall be located to that the bottom of the footing is below the scour depth determined for the check flood for scour.

Refer to Section 2.3.11 Scour for additional guidance.

E. Facing

Where appropriate, the nose of the pier should be designed to effectively break up or deflect floating ice or debris. Pier life can be extended by facing the nose with steel plate/angle or by facing the pier with granite.

5.5.2 Pile Bent Piers

Pile bent piers are significantly less expensive than mass concrete piers and provide environmental advantages by eliminating cofferdam work and its associated impacts. Pile bents should be used wherever possible based upon the criteria below.

5.5.2.1 Pile Bent Use Criteria

Pile bent piers should not be used in the following locations:

- In rivers known for severe ice conditions - Allagash, Androscoggin, Aroostook, Kennebec, Penobscot, St. Croix, and St. John
- Other locations with severe ice conditions
- In shipping channels
- Where the pier is not aligned with the design flow

Pile bent piers should be considered for structures in the following locations:

- In tidal rivers
- In environmentally sensitive areas
- For grade-separated structures
- Within the headwater or tailwater of dams or lakes, except when ice has been known to form predominantly on one side of the pier with an open channel in the adjacent span, resulting in static ice forces on all piles.

The following issues affect the design of pile bent piers and must also be considered when evaluating the appropriateness of this system.

- Pile length - The pile length is a function of the depth to bedrock, loading conditions, the type of overburden material, the depth of scour, degree of pile fixity and restraint, and the pile bracing.

- Pile loads - The following issues affect pile loads:

  1. Application location and magnitude of ice load

  2. Skew - Longitudinal superstructure forces are transmitted into the longitudinal pier axis and increase with greater skew angles.

  3. Bridge width - Pier cap shrinkage forces increase with increasing bridge width.

  4. Span length - Dead and live load axial forces are dependent upon span length.

  5. Seismic forces.

An additional issue to be considered when evaluating the appropriateness of pipe pile pier bents is corrosion. Special consideration should be given to corrosion and abrasion of steel pile bent piers to ensure a minimum 75 year structure life is achieved. This is of particular concern in locations where there is insufficient water to install cathodic protection in accordance with Section 5.5.2.6, and in locations where debris or sediment loads may abrade pile protective coatings. In these locations the design should consider additional protection such as encased H-piles with sacrificial steel pipe pile or sacrificial fiber reinforced polymer (FRP) composite pipe pile casings.

5.5.2.2 Loads and Load Combinations

Pile bent piers should be designed in accordance with AASHTO LRFD. Structural analysis and design of reinforced concrete should include pile bent cap flexure and shear checks, pile structural resistance and buckling and lateral stability of piles. Geotechnical design checks should include strength limit state checks and service limit state checks such as global stability, horizontal bent displacement and pile settlement.

Where applicable, consideration should be given to other loading conditions, including seismic forces resulting from earthquake loading and debris lodged against pier, as outlined in 5.5.1.2 Load Combinations.

Pile bent piers should be designed and proportioned to resist all applicable load combinations specified for mass piers in 5.5.1.2 Load Combinations and Load Factors, and as outlined in Chapter 3 Loads and LRFD Articles 3.4.1, 11.5 and 11.7.

A. Live Loads

Vehicular live loads must be located within the design lanes on the superstructure such that maximum forces occur in the pile cap and piles.

Impact should be applied to pier caps and that the portion of the piles that are acting as columns, defined as the vertical distance from the pile cap to the point of fixity below grade. Impact should be applied at or above Q1.1.

B. Ice Loads

For the Extreme Event II load combination, unfactored ice loads should be placed at the Q50 stage elevation and checked at a lower elevation that will cause maximum moment in the nose pile, provided the elevation is at or above Q1.1. The ice thickness of past practice should be increased by 1.0 foot.

Transverse ice loads should be applied to only the nose pile when ice is directly applied to the nose pile, or be uniformly distributed over the cap when ice is applied to the cap.

A modified Strength Limit State analysis should also be performed with factored ice loads following the criteria specified in 3.9 Ice Loads, with appropriate strength limit state resistance factors for the pier component being analyzed.

C. Water Loads

Stream pressure should be reduced when the ice elevation is lowered to check maximum moment in the nose pile.

Stream pressure should be applied to each pile in the bent, using an appropriate stream flow velocity.

D. Wind Loads

Longitudinal components of wind on superstructure and wind on live load should be distributed to the abutments when structure fixity is at the abutments.

E. Seismic Loads

Seismic loads transverse to the bridge should be shared between all substructure units based upon their stiffness.

Longitudinal seismic loads should be distributed to the abutments where there is at least one fixed abutment with no forces applied to the pier.

F. Shrinkage and Temperature Forces

Shrinkage and temperature forces affect pile bents in two ways:

- o Pile cap shrinkage and temperature actions are applied to the longitudinal axis of the pier.
- o Thermal forces are induced by the superstructure are applied along both the transverse and longitudinal pier axes, with the magnitude dependent upon the skew angle.

Two-span integral abutment bridges will have no associated thermal forces applied, as the forces are assumed to be balanced at the pier. The Structural Designer may want to include thermal forces for two-span integral abutment bridges on steep grades, assuming that the bridge will expand and contract downhill.

For non-integral abutment bridges, thermal forces induced by the superstructure bending the pile bents must be considered in the design of the fixed abutment.

G. Braking Forces

If the structure is fixed at an abutment, the longitudinal braking forces will have no effect on the pier, as the forces are assumed to be distributed to the abutments.

H. Friction Forces

Friction forces resulting from all longitudinal superstructure forces should be applied to pile bents with expansion bearings.

I. Collision Loads

Where the possibility of collision exists from vehicular, railroad, or water traffic, an appropriate risk analysis should be made to determine the degree of impact resistance to be provided and/or the appropriate protection system.

5.5.2.3 Pile Cap Design

Pile bent cap design should consider the following design features:

- Piles should be embedded at least 12 inches
- Pile clearance with 6 inches of concrete cover
- Tolerance on pile installation misalignments > or = 2 inches
- Consider concrete pile anchorage
- Pile spacing should be at least 30 inches or 2.5 times the pile diameter

5.5.2.4 Pile Type Selection Criteria

Concrete filled pipe piles, precast concrete piles, combination H-piles encased with pipe piles filled with concrete, and drilled shafts may be considered for pile bent piers under the following conditions:

A. Shallow overburden depth (embedment less than or equal to the fixity depth)

  - Footing-encased pipe or precast concrete piles
  - Rock-socketed pipe piles
  - Rock-socketed H-piles, with pipe pile encasement to top of bedrock
  - Rock-anchored/doweled pipe piles (Note: AASHTO LRFD is absent of discussion on the use of rock-anchor pipe piles. The use of rock-anchored pipe piles should be considered only when the preceding alternatives are found not feasible. Rock anchors or dowels should have double corrosion protection.)
  - Rock-socketed drilled shafts

B. Intermediate overburden depth (embedment greater than depth to fixity and less than 3 times fixity depth)

  - Pipe piles filled with concrete and a reinforcing cage (The reinforcing cage may be eliminated with the approval of the Engineer of Design.)
  - Precast concrete piles

- o Drilled shafts

C. Deep overburden depth (embedment greater than 3 times fixity depth)

- o Pipe piles filled with concrete and a reinforcing cage (The reinforcing cage may be removed with the approval of the Engineer of Design.)
- o H-piles with pipe pile encasement to pile fixity depth
- o Precast concrete piles
- o Drilled shafts

The choice of steel versus concrete piling in intermediate and deep applications should be determined by a cost analysis. Issues include the relative costs of H-piles to precast concrete piles or pipe piles, encasement and the relationship between the exposed length (including the scour depth), the depth to fixity, and the total depth to bearing.

D. Pier Bent Pile Alternatives

Because of ongoing corrosion and durability issues with steel pipe piles, Geotechnical Engineers and Designers should routinely examine the feasibility and practically of other pier bent pile-types, namely:

- o precast concrete piles
- o drilled shaft pier bents
- o encased H-piles with a sacrificial steel pipe pile or a sacrificial fiber reinforced polymer (FRP) composite pipe pile casing

5.5.2.5 Pile Protection

A. Encased H-Piles

Steel H-piles should not be used for piers without full encasement protection. The encasement usually is a steel pipe pile filled with concrete. H-piles should be protected by a minimum of 3 inch clear encasement from the pier cap to a minimum of 10 feet below streambed or 2 feet below the total scour depth. Due to the significant additional load section provided by the composite steel and concrete section, the pipe pile should be used for strength. If the pipe pile is used for strength, it should extend to the point of fixity below streambed.

The pipe pile should be protected and designed as detailed in Paragraph B. Pipe Piles, below.

B. Pipe Piles

Pipe piles bents in fresh water environments should be hot-dipped galvanized with UV-resistant epoxy top coat. Pipe pile bents in brackish or salt water should be coated with fusion bonded epoxy paint with a coat thickness of 18-20 mils. This is an increase in the previous standard of 12 mils.

Fusion-bonded epoxy coatings or galvanized surfaces should be applied to a minimum of 10 feet below streambed or 2 feet below the total scour depth.

Cathodic protection (aluminum anodes) should always be used in addition to the protective coatings in salt and fresh water environments as long as there is sufficient water to submerge the anodes at low water.

Refer to 5.5.2.6 Pipe Pile Coatings and Cathodic Protection for detailed recommendations.

C. Precast/Prestressed Concrete Piles

Concrete cover for rebar should be a minimum of 2 inches for fresh water locations and 3 inches for salt water locations.

5.5.2.6 Pipe Pile Coatings and Cathodic Protection

A. Standardized Anodes

Pipe pile pier bents and cargo/ferry piers should specify a standard anode ingot length, composition (aluminum alloy plus minor constituents) and weight.

The standard should be a 34-lb, aluminum alloy anode, approximately 3 feet long. Larger, heavier anodes are not easy to handle and should be avoided unless the bent has a lot of uncoated steel or the project is a significant sheet pile structure where there is a greater chance for exposed steel. On large piles with long exposed lengths (deep water), consideration should be given to installing more than one anode rather than using a heavier anode.

There are a lot of variables in the rate of corrosion between sites, and it may happen that the standard anode may not be suitable for all sites. Larger anodes may be necessary for more aggressive environments (brackish and saltwater). Specifying a heavier anode may be required.

B. Anode Location

The top of the 34-lb, 3-foot long anode should be 3 feet below Low Water, so that it is always submerged. This implies the water channel needs to have at least 6 feet of water at Low Water.

Anodes should be installed on the more protected side of the pile: on the underside on battered piles, on the downstream side on plumb piles in rivers, and on the more protected side (if there is one) on plumb piles in tidal crossings. If possible, show the location of the anodes on the Plan drawings, so there is no debate in the field about what constitutes the ‘more protected side’.

C. Shallow Water Situations

If there is not enough depth of water to submerge the anodes at all times, the anodes are not as effective in protecting the pile segment above the waterline.

Where the water is shallow and there is no submerged portion or a limited submerged portion of pile for anodes, Designers should consider:

- specify a non-standard, shorter ingot if that permits installation on a pile in shallow water
- fusion-bonded epoxy treatment over hot-dipped galvanized piles
- encasing H-piles with a sacrificial steel pipe pile or a sacrificial FRP composite pipe pile casing

D. Anode Attachment

Plans should specify a 2-inch clearance between the anode and the pile. This allows Bridge Inspectors to get a clear view of the anode, and the pile surface is more “inspectable” and the anodes easier to replace. Attachment hardware consisting of a 3-inch long, ¾-inch diameter threaded stud, with double nuts, is recommended. The studs should be installed in a manner that ensures the best steel to steel connection and the best electrical connection. The weld area shall be ground to bare metal for this purpose. Only after the stud and anode are attached, shall the weld at the base on the stud be covered with curable polyamide epoxy coating.

E. Brackish and Saltwater Environments

Steel pile bents in brackish or salt water should not be hot-dipped galvanized with UV-resistant epoxy top coat. These pile bents should be coated with fusion bonded epoxy coating with a thickness of 18-20 mil.

Cathodic protection aluminum anodes should always be installed on pile pier bents in salt water when there is enough water.

F. Freshwater Environments

Steel pile bents in freshwater should be hot-dip galvanized with a UV-resistant topcoat system. The UV-resistant topcoat tends to fade where the upper part of the pile gets direct sunlight and reflected light from the water surface. Considerations should be given to topcoating with fluorocarbon paint, which is more UV resistant.

Cathodic protection aluminum anodes should be installed on pile pier bents in fresh water, with the exception of river crossings with very shallow water.

G. Coating Repairs

Pile coating "touch-up" per the manufacturer's recommendations is considered the best practice for dealing with damaged pile sections. The "touch-up" material on some jobs (in particular, Alna-Newcastle) seems to be performing well. The Bridge Program should determine the best "touch-up" method and specify it – not just specify *"touch-up per Manufacturer's recommendations*".

H. Pipe Pile Material

Steel pipe piles should be ASTM 252 and have straight butt-welded seams or be seamless. Spiral seams are not recommended because the magnitude of welded surfaces which are vulnerable to thin coatings, ice abrasion, and bumping during construction – all of which lead to damage in the coating. Welds should be ground down and blended smoothly with the pile material. The number of field and mill splices should be limited.

I. Damage during Construction

Specifications should include requirements for Contractors to line driving templates with fire hoses, carpets, etc., to prevent the scraping off of the coatings during pile driving. Contractors should be required to repair or replace any protective mats that fall off during driving, prior to commencing driving any more pile.

5.5.2.7 Additional Pile Bent Pier Design Criteria

Pile bents should consist of a concrete pile cap supported by a single row of piles, multiple rows of piles, or a braced group of piles.

A. Pile Design

Pile design should investigate resistance to axial loads, combined axial and bending, and buckling failure of the exposed pile lengths. Guidance for computing the unsupported pile length is provided in Section B, below. Stability of the pile bent pier under combined axial and lateral loads should be investigated with a dedicated soil-structure interaction analysis, using FB-Pier software.

B. Pile Length

The unsupported length, L_{us}, is defined by the following:

$$L_{us} = K \cdot (L_u + L_e)$$

where:

K = Effective Length Factor. Refer to LRFD Article 4.6.2.5 and Table C4.6.2.5-1.
L_u = Exposed pile length above ground.
L_e = Effective pile length from ground surface to the point of assumed fixity below ground, including scour effects. Refer to Figure 5-4 and Figure 5-5.

The depth to fixity shown in Figure 5-4 and Figure 5-5 was determined using the Davisson and Robinson procedure provided in LRFD Article 10.7.3.13.4 and assumes no lateral loading on the pile. Where piles used for pile bent piers are subjected to lateral loading or where the embedment length is less than $3L_e$, a detailed analysis by the Designer using actual loading and soil conditions is required.

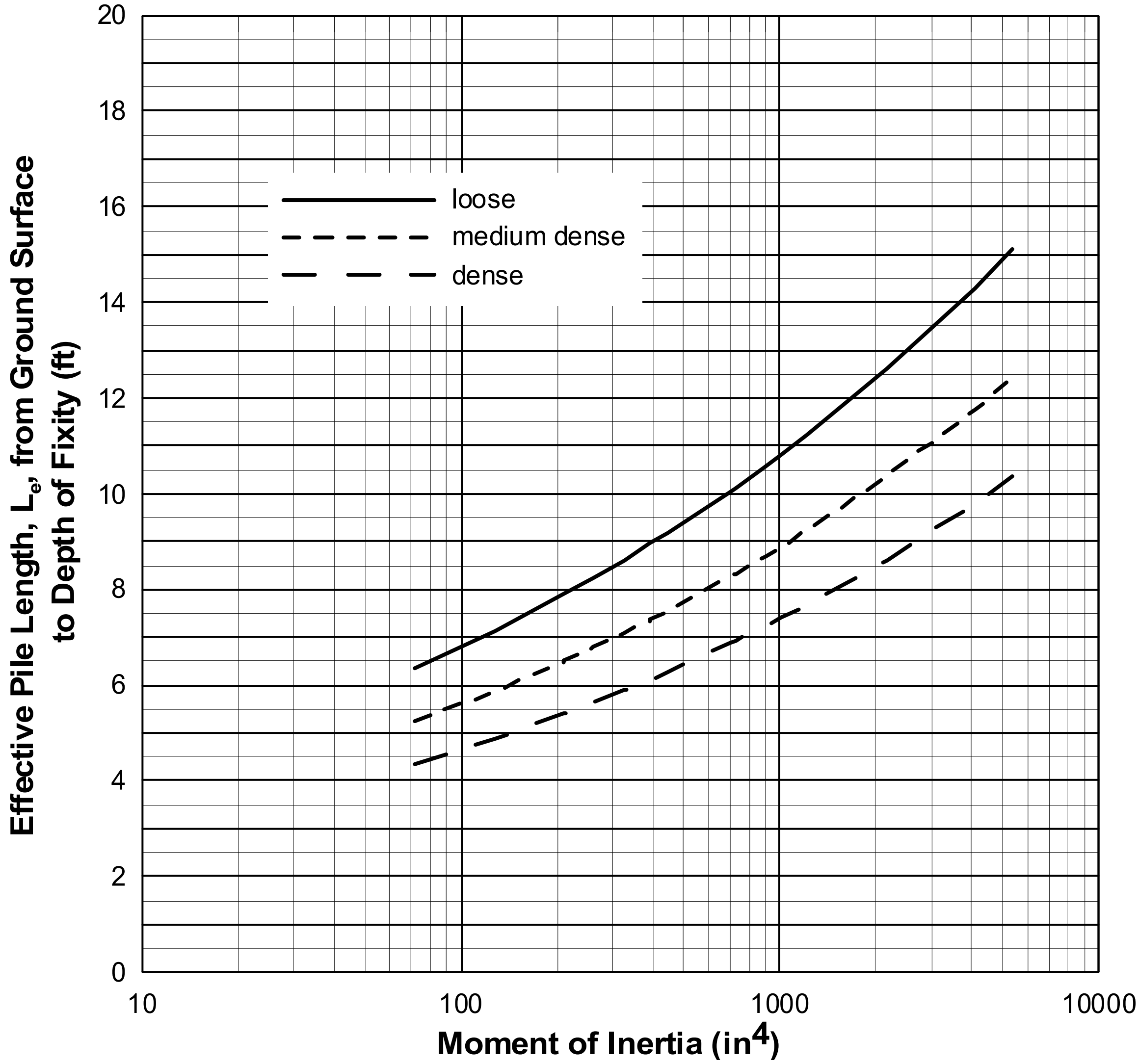


Figure 5-4 Effective Pile Length for Piles in Sand From Ground Surface to Depth of Fixity Axially Loaded

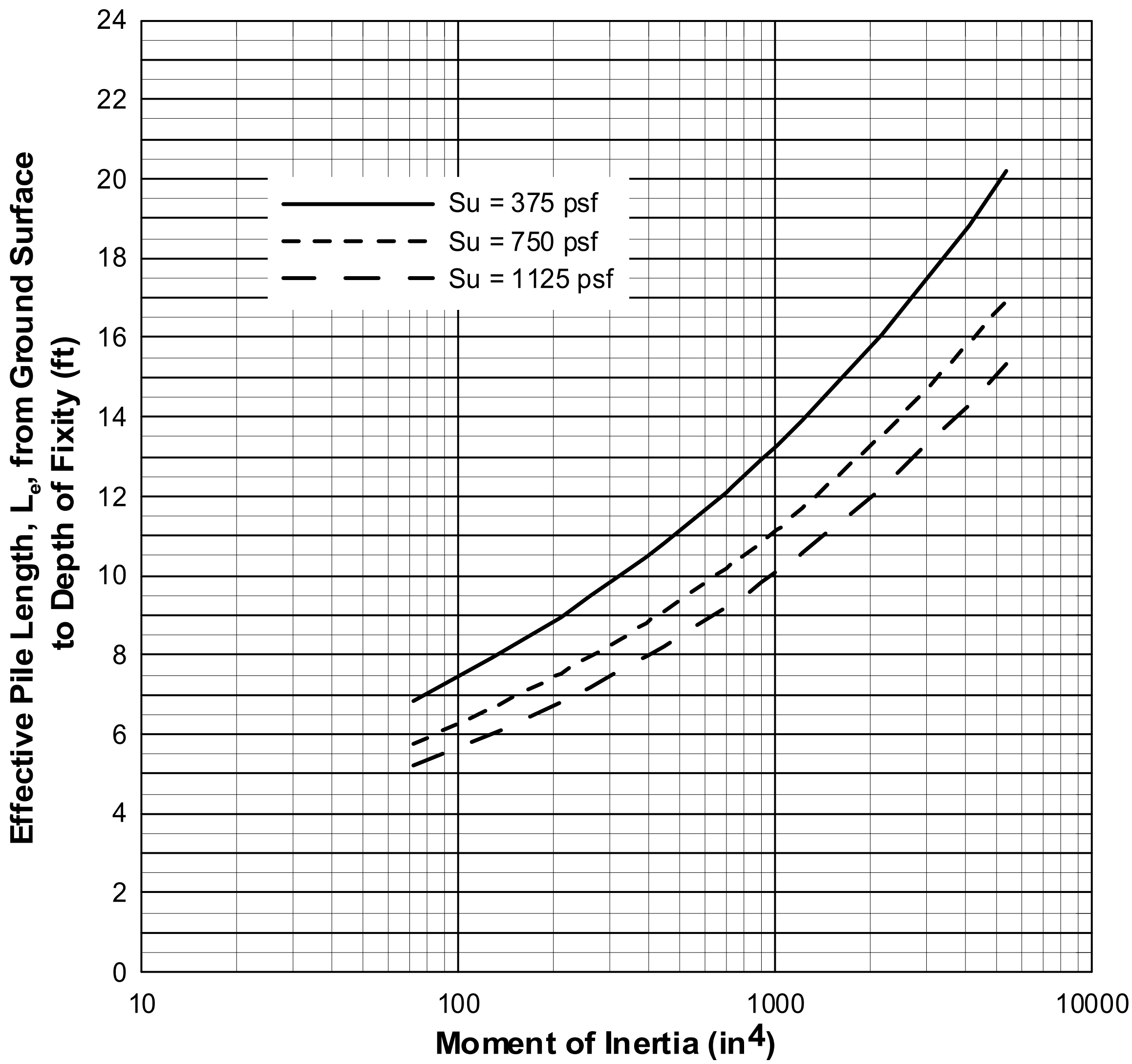


Figure 5-5 Effective Pile Length for Piles in Clay From Ground Surface to Depth of Fixity Axially Loaded

C. Nose Pile Batter

Where possible, the nose pile should be battered a minimum of 15° to take advantage of the allowance for ice load reduction due to nose inclination (refer to LRFD Article 3.9.2.2). When ice is applied to the pier cap or within 5 feet of the pier cap, no reduction should be taken.

D. Design Section

Encased H-piles and concrete-filled pipe piles should be designed assuming contribution from the concrete and a portion of the steel pipe pile shell, allowing for a minimum of 0.15 inch of sacrificial shell corrosion, based on a design corrosion rate of 0.05 mm per year. The pipe pile shell must have a minimum thickness of 1/2" to allow for proper driving of the pile and to resist corrosion.

MaineDOT Section 711.01 specifies ASTM 252 for Welded and Seamless Steel Pipe Piles. Designers should consider that ASTM 252 permits under-fabrication of the wall thicknesses up to 12.5% of the specified nominal wall thickness. Example: If the design calls for 5/8-inch wall, the design section should be reduced by a minimum 1/8-inch for sacrificial steel shell corrosion and an additional 1/16-inch to account for permissible fabrication variation.

5.6 Retaining Walls

5.6.1 General

Retaining walls typically used by the Bridge Program are gravity walls, cantilever-type walls, mechanically stabilized earth (MSE) walls, prefabricated proprietary walls and soil nail walls, each of which is discussed in detail in the following sections. The selection of the appropriate retaining wall should be based on an assessment of the magnitude and direction of loading, depth to suitable foundation support, potential for earthquake loading, presence of deleterious factors, proximity of physical constraints, wall site cross-section geometry, tolerable and differential settlements, facing appearance, and ease and cost of construction. A feasibility study should address which wall is most suited to the site and is simplest to construct. The study should address the approximate scope of the design for the most feasible walls, and provide cost comparison between alternatives.

5.6.1.1 Retaining Wall Type Selection

Due to construction techniques and base width requirements, some wall types are best suited for cut sections whereas others are best suited for fill situations. 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.

A. Walls in Cut Sections

Anchored walls and soil nail walls, which have soil reinforcements drilled into the in-situ soil/bedrock, and cantilever sheet pile walls, are generally used in cut situations. These walls are typically constructed from the top down.

B. Walls in Fill Sections

MSE walls are constructed by placing soil reinforcement between the layers of fill from the bottom up and are therefore best suited to fill situations. Additionally, the base width of MSE walls is typically on the order of 70% of the wall height, which would require considerable excavation in a cut section, making the use of this wall uneconomical.

C. Walls in Cut or Fill Sections

Gravity, cantilever-type, and prefabricated proprietary walls are freestanding structural systems built from the bottom up that do not rely on soil reinforcement techniques to provide stability. These types of walls have a narrower base width than MSE structures (on the order of 50% of the wall height) making this type of wall feasible in fill situations as well as many cut situations.

5.6.1.2 Service Life

Retaining walls should be designed for a service life based on consideration of the potential long-term effects of material deterioration, seepage, stray currents, and other potentially deleterious environmental factors on each of the material components comprising the wall. For most applications, permanent retaining walls should be designed for a minimum service life of 75 years. Retaining walls for temporary applications are typically designed for a service life of 36 months or less. Greater level of safety and/or longer service life (i.e., 100 years) may be appropriate for walls that support bridge abutments, for which the consequences of poor performance or failure would be severe.

The quality of in-service performance is an important consideration in the design of permanent retaining walls. Permanent walls should be designed to retain an aesthetically pleasing appearance, and be essentially maintenance free throughout their design service life.

5.6.1.3 Design Loads

Retaining walls should be designed in accordance with AASHTO LRFD Structural analyses, the design of reinforced concrete and geotechnical analyses of retaining walls will be computed using LRFD procedures using factored loads and factored resistances. The geotechnical design of conventional retaining walls typically follows the LRFD approach for the design of abutments on spread footings, presented in 5.3 Spread Footings and 5.4 Abutments. Where a wall is supported with piles or dilled shafts, the design will follow LRFD and 5.4.1.12 Abutments Supported on Pile Foundations and 5.7 Piles, as appropriate. Loads should be determined in accordance with AASHTO LRFD and as outlined in Chapter 3 and

5.4.1.2 Load Combinations and Load Factors. The following load conditions should be considered when applicable:

- Lateral earth pressure
- Weight of soil above the footing or within the wall system
- Self-weight of the wall
- Lateral loads due to live load impact on the parapets
- Surcharge loads, due to live load
- Surcharge load caused by earth, point, line or strip loads on the upper surface
- Railroad loading
- Hydrostatic pressure (if no drainage is provided)

Earth pressure due to compaction should be considered when static or dynamic compaction is used within a distance of one-half of the wall height. These loads will only apply to during construction phase; therefore a load factor of 1.0 is appropriate.

Walls that can tolerate little or no movement, or are restrained, should be designed for at-rest (K_o) earth pressure with a maximum load factor for at-rest earth pressure, γ_{EH}, of 1.35.

5.6.1.4 Limit States

Retaining walls should be designed to resist all applicable load combinations specified in LRFD 3.4.1 and 11.5.5.

Strength limit state checks of walls should assess external failure mechanisms:

- Sliding
- Eccentricity
- Bearing Resistance
- Structural Capacity

Service limit state check should assess overall stability, wall settlement and lateral displacement.

Walls should be evaluated for each of the applicable limit states:

- Strength I-construction. Strength Limit State I which models the basic load combination related to construction loads. Load factors for the dead load of other components shall not be less than 1.25. Live load surcharge is included to account for construction equipment live loading; a construction load factor of not less than 1.5 should be assumed.

- Strength I-a: Strength Limit State I, which models the basic load combination related to normal vehicle live load surcharge, dead load plus earth pressure, finished grade, including any point or strip loads on the wall backfill Minimum vertical permanent load factors and maximum horizontal load factors are selected to produce extreme force effects for wall sliding and eccentricity, and structural design of the wall stem.

- Strength 1-b: Strength Limit State 1 as described above, except maximum vertical permanent load factors, including earth loads, are selected to produce an extreme force effect for bearing capacity analyses.

- Service I: Service Limit State I – Load combination relating to normal operational use of the wall with all loads taken at their unfactored values.

Wall foundations subject to scour should be designed at the strength and service limit states so that there is adequate foundation resistance, in conjunction with the depth of scour from the design flood, using appropriate strength and service limit state resistance factors.

The consequences of changes in wall foundation conditions due to scour from the check flood for scour should be assessed at the extreme event limit state with resistance factors of 1.0.

Where retaining walls are to be designed to resist earthquake forces, collisions by roadway or railway vehicles, or vessel collision, the structures should be evaluated for the following additional limit states:

- Extreme Event I – Load combination including earthquake forces

- Extreme Event II – Load combination relating to collision by vehicles, railways or vessels.

Each load for each limit state above is modified by the prescribed load factor, γ. Certain permanent loads, including earth loads, should be factored using the load factors γ_p. Load factors should be selected to produce the total extreme factored force effect. Applicable load factors, load combinations and the analyses for which they will govern, are provided in Table 5-6.

Table 5-6 Typical Load Groups and Load Factors

Load Group	γ_{DC}	γ_{EV}	γ_{LSV}	γ_{LSH}	γ_{EH} (active & passive)	γ_{EH} (at-rest)	γ_{ES}	Typical Geotechnical Analysis Governed
Strength I-a	0.90	1.0	1.75	1.75	1.5	1.35	1.5	– Sliding – Eccentricity – (overturning) – Structural design of wall stem – Anchor pullout
Strength I-b	1.25	1.35	1.75	1.75	1.5	1.35	1.5	– Bearing Capacity – Structural design of the wall footing
Service I	1.0	1.0	1.0	1.0	1.0	1.0	1.0	– Settlement – Lateral displacement – Global stability

5.6.1.5 Strength Limit State

A. Bearing resistance

The check for bearing resistance for wall spread footings on soil or rock is identical to the requirements for abutments described in 5.3.5 Bearing Resistance. Wall foundations subject to scour should be designed so that the nominal bearing resistance, in conjunction with the depth of scour determined for the check flood for scour, provides adequate resistance to support the unfactored Strength Limit State Loads with a resistance factor of 1.0.

B. Eccentricity

The overturning calculation used in ASD is replaced with the eccentricity check. Eccentricity of loading on walls founded on spread footings is identical to the requirements for abutments, should be calculated for each load group and checked to meet the following criteria:

- o E < B/3 for foundations on soil
- o E < 0.45B for foundations or bedrock

C. Sliding

Sliding calculations for walls on spread footings are identical to the requirements for abutments described in 5.3.8 Sliding. Passive pressures in front of the wall should be neglected. To maximize the effect of the live load surcharge, the horizontal component of the live load surcharge should be included, whereas the vertical component over the heel or base should be neglected.

D. Pile Resistance

The design of walls founded on deep foundations is similar to the design requirements described in 5.4.1.12 Abutments Supported on Pile Foundations and 5.7 Piles.

E. Overall Stability

The overall global stability of retaining walls should always be checked at Service I load combination with a resistance factor, φ, of 0.65.

5.6.1.6 Service Limit State Checks

Service limit state wall settlement should be checked with the following performance limits in mind:

- Total settlement can be estimated using the procedures and criteria described in 5.3.6 Settlement. The tolerable total settlement criterion generally considers its effect on serviceability.
- Settlement may be critical where the wall interacts with other structures, e.g. at the approach to a pile supported abutment.
- Distortion, i.e., the ratio of horizontal movement to vertical movement should be less that 1/500.
- Lateral deformations will usually take place during construction and be affected by wall batter, compaction effort and construction equipment next to the wall.
- Global stability.

5.6.1.7 Design Considerations

All retaining walls should be designed with consideration of frost protection (Section 5.2.1), scour protection (Section 2.3.11), bearing resistance (Section 5.3.5), settlement (Section 5.3.6), stability (Section 5.3.7), drainage considerations (Section 5.3.11), and seismic considerations (Section 5.2.5), as appropriate.

All retaining walls require a subsurface investigation of the underlying soil or bedrock that will support the structure or tie-back elements. Minimum requirements for number, spacing and depth of exploratory borings are provided in Section 2.10 Subsurface Exploration Programs.

5.6.1.8 Aesthetics

Retaining walls should have a pleasing appearance that is compatible with the surrounding terrain and other structures in the vicinity. Aesthetic requirements include consideration of the wall face material, the top profile, the terminals, and the surface finish (texture, color, and pattern). Where appropriate, provide planting areas and irrigation conduits. In higher walls, variation in treatment is recommended for a pleasing appearance. High, continuous walls are generally not desirable from an aesthetic standpoint. Consider stepping high or long retaining walls in areas of high visibility.

5.6.2 Gravity Retaining Walls

Gravity retaining walls are generally trapezoidal in section and derive their capacity to resist lateral soil loads through a combination of self- weight and sliding resistance. Gravity walls can be subdivided into rigid gravity walls, which will be discussed in this section, MSE walls discussed in Section 5.6.5.4, and prefabricated proprietary walls discussed in Section 5.6.5.

5.6.2.1 Design Section

Gravity wingwalls should have a thickness at the top of 1’-6” in a direction normal to the front neat line. Batters on the front and back faces of wingwalls should be related to the vertical plane, which is normal to the front neat line. The front neat line is a horizontal line, which is the intersection of the top of footing elevation and the front face of the wall. If there is no footing, a working elevation should be used. Gravity walls of any length should be constructed to work integrally with abutments.

5.6.2.2 Earth Loads

Rigid gravity walls should be designed as unrestrained, which means that they are free to rotate at the top in an active state of earth pressure. An active earth pressure coefficient, K_a, should be calculated using Coulomb Theory as described in Section 3.6.5.1.

5.6.3 Gravity Cantilever-type Retaining Walls

This section discusses gravity, cantilever-type retaining walls. This type of wall is differentiated from a non-gravity cantilever retaining wall by relying on the bending action of the wall stem, in addition to self-weight, to resist lateral earth pressures. The footing contributes to the wall stability in overturning and sliding. Non-gravity cantilever retaining walls (i.e., sheet pile walls) are discussed in Section 5.6.4.

5.6.3.1 Design Section Gravity Cantilever Retaining Walls

Cantilever walls should have the following limits for wall thicknesses (heights are measured from top of the wall footing):

- 1’-3” minimum thickness for walls up to 6 feet high at the highest point.
- 1’-6” minimum thickness for walls between 6 feet and 20 feet in height at the highest point.
- 1’-9” minimum thickness for walls over 20 feet in height at the highest point.
- Walls should be increased in thickness to accommodate recessed architectural treatment, as necessary.

Wingwalls that are 15 feet or more in height at the ends may be designed with butterfly wings, if economical to do so.

On wingwalls that are less than 15 feet in height at the ends, the footing may be reduced in length if it is not required for structural or geotechnical considerations. The wall should be detailed with the bottom of the wall at the elevation of the top of the footing.

Tops of parapets should not have elevations above the adjacent curbs or sidewalks.

Gravity cantilever wingwalls more than about 20 feet long should be designed to work independently from the abutment, except that footings should be integral. A vertical contraction or expansion joint with no shear key should be used near the corner between the abutment and the wingwall. The front face of the wingwalls should be recessed 2 inches back from the face of the wall on the abutment side of the contraction or expansion joint.

Gravity cantilever type wingwalls that are less than about 20 feet long should also be designed independently from the abutment; however, the wingwall should be restrained at the corner through an integral connection to the abutment. Soil pressure under the footing, sliding, and eccentricity should be evaluated as discussed in Section 5.3 Spread Footings. The restraining force at the corner is considered to be caused by at rest lateral earth pressure, as a minimum, because of the wingwall's inability to deflect at the corner. The corner should be designed to be restrained by concrete beam action with horizontal reinforcing steel anchored into the abutment section.

5.6.3.2 Earth Loads

For earth loads relative to cantilever walls refer to Section 3.6. Load factors for earth loads and surcharge loads are provided in Table 5-4. In the case of a long wall with a variable height, the wall should be divided into more than one design section. The design section should be at the highest third point of the wall. Refer to Figure 5-6 for further guidance.

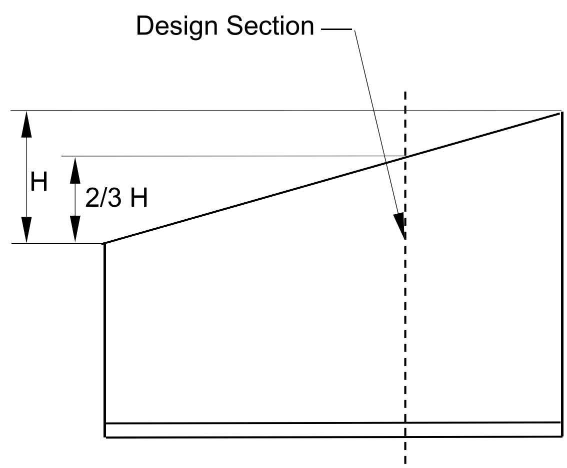


Figure 5-6 Retaining Wall Design Section

Gravity cantilever walls should be designed as unrestrained, which means that they are free to rotate at the top in an active state of earth pressure. An active earth pressure coefficient, $K_{a,}$ should be as described in Section 3.6.4 and factored as specified in Table 5-6.

5.6.4 Non-Gravity Cantilever and Anchored Retaining Walls

This section discusses non-gravity cantilever retaining walls. Non-gravity cantilever retaining walls derive lateral resistance through embedment of vertical wall elements, sometimes in combination with anchors or tie-backs. These vertical elements may consist of sheet piles, soldier piles, caissons, or drilled shafts. The vertical elements may form the entire wall face or they may be spanned structurally using timber lagging or other materials to form the wall face.

The design of cantilever and anchored walls include additional checks for the geotechnical resistance of anchors in pullout, bearing resistance of vertical elements, the passive resistance of vertical elements and the structural capacity of anchors, vertical wall elements and wall facing. Resistance factors specific to cantilever and anchored walls can be found in LRFD Table 11.5.6-1

5.6.4.1 Soil Nail Walls

Soil nail walls are technically anchored walls that employ a reinforced soil mass serving as a gravity retaining structure. The reinforced soil mass of a soil nail wall is created by drilling and grouting steel anchors into an in-situ soil mass. The anchored soil mass is then covered with shotcrete. The temporary shotcrete face is then covered with a permanent facing system, typically cast-in-place concrete, precast concrete, or timber lagging. Soil nail walls are suited to cut situations only.

Soil nail walls are relatively low cost and can be used in areas of restricted overhead or lateral clearance. Soil nail walls are built from the top down and are only suitable if the site soils have adequate “stand-up” time of 1 to 2 days in a 5 foot vertical cut. Soil nail walls are not applicable to sites with bouldery soils, which could interfere with nail installation. This wall type is not recommended in uniform or water bearing sands or where there is a potential deep failure surface. Maximum wall heights of 30 feet are allowed.

These walls can be designed by the Designer or specified as a design-build item. The PS&E package should include the plan development information discussed in Section 5.6.5.5. Special Provisions have been developed for soil nail walls. Check with the Geotechnical Designer for the current Special Provision.

5.6.5 Prefabricated Proprietary Walls

Prefabricated proprietary walls are any prefabricated wall system approved by MaineDOT and produced by a manufacturer licensed by the wall vendor. Prefabricated proprietary walls are typically designed by the vendor, but may be designed by the Geotechnical Designer. In design, the vendor should consider external stability with respect to sliding and overturning (at every module level) and internal stability with respect to pullout, as specified in LRFD 11.10 and Chapter 3, Loads. The Geotechnical Designer is required to verify acceptable global stability of the wall using a resistance factor of 0.65 prior to advertisement. The factored bearing resistance of the wall foundation soil or bedrock must be shown on the plans.

5.6.5.1 Proprietary Retaining Walls

Retaining walls available for a given project include standard walls, where the responsibility of the design is the Structural Designer, and proprietary walls, which are designed by a wall manufacturer. There are MaineDOT preapproved proprietary wall systems and non-approved proprietary wall systems. Preapproved wall systems have been extensively reviewed by MaineDOT and are listed on the MaineDOT Qualified Products List (QPL) webpage for the particular wall type. MaineDOT has developed a review process for the pre-approval of non-approved proprietary walls systems (MaineDOT, 2010), available on the MaineDOT QPL website. Non-approved proprietary walls must go through the pre-approval review process prior to use of the wall system.

5.6.5.2 Prefabricated Concrete Modular Gravity Walls

Prefabricated concrete modular gravity (PCMG) walls covered under Special Provision 635 should consist of either "T-Wall®" as provided by a licensed manufacturer of the Neel Company, Springfield, Virginia, or "DoubleWal®" as provided by a licensed manufacturer of DoubleWal Corp., Plainville, Connecticut.

PCMG walls should be designed in accordance with Special Provision 635 and LRFD Article 11.11. In general, the design requirements are similar to the requirements for conventional retaining walls and abutments, with the exception of pullout resistance requirements and dedicated analyses at each level of modular units.

PCMG walls should be considered on all projects where metal bin, gabion, MSE, and cast-in-place walls are considered. PCMG walls should be limited to a maximum height of 27.5 feet and a maximum batter of 1/6 (2 inches per foot). Refer to Section 5.6.5.5 PS&E for Project with Proprietary Walls for plan development requirements.

Whenever possible, a battered wall will be used in preference to a vertical wall. The use of a vertical wall design may be necessary where the wall is located on a horizontal curve that may result in construction conflicts, or where property costs or other right-of-way considerations dictate.

PCMG walls should be designed with adequate embedment for frost protection. Refer to Section 5.2.1 Frost for guidance.

PCMG walls should not be used in locations where there is scour potential, unless suitable scour protection can be economically provided. Refer to Section 2.3.11 Scour for guidance.

Where special drainage problems are encountered, such as seepage of water in the excavated backslope, underdrain will be provided behind the wall. Refer to Section 5.3.11 Drainage Considerations for further guidance.

Where PCMG walls will come in contact with salt water, all rebar should be epoxy coated and the concrete should be class LP. The appropriate note from Appendix D Standard Notes Prefabricated Concrete Modular Gravity Wall should be on the contract drawings.

Where PCMG walls are to be located in water, consideration should be given to drainage behind the wall. As a minimum, the Designer should consider a 12 inch thick layer of crushed stone extending vertically along the inside wall face. Crushed stone should be separated from surrounding soils with an erosion control geotextile. When drainage features are used for PCMG walls, payment should be considered incidental.

PCMG walls may be considered to retain soil supporting bridge substructures, with the exception of bridges over waterways. Their use is subject to the approval of the Assistant Bridge Program Manager at the PDR stage. These types of walls shall be designed for a service life of 100 years. The PCMG concrete shall contain a minimum of 5.5 gal/yd^3 of corrosion inhibitor and use corrosion resistant reinforcing. PCMG walls which retaining abutments and are within 30 feet of the edge of a roadway or 50 feet of the centerline of a railway track should be designed for collision forces or protected with a crashworthy barrier (see 5.4.1.7.E). Additional design criteria for abutments retained by PCMG walls are similar to those for MSE walls described in 5.6.5.4.

Cofferdams required for PCMG wall construction should be considered incidental to wall construction. The appropriate notes from Appendix D Standard Notes Prefabricated Concrete Modular Gravity Wall should be on the contract drawings.

PCMG walls are measured and paid for by the area of wall face, as determined from the plan dimensions. The PCMG pay item includes compensation for excavation, excavation support foundation material, backfill material, and wall design. Consult Special Provision 635 for current measurement and payment information.

5.6.5.3 Precast Concrete Block Gravity Walls

Precast concrete block gravity walls consist of walls where precast concrete units are stacked vertically, function either as a gravity retaining wall or as a facing with geosynthetic-reinforced soil backfill, as covered in Special Provision 635. The connection between adjacent courses of modular blocks may be mechanical (cast knobs) or frictional. A preference is for mechanical connections. These wall systems are generally limited to a maximum height of 4.5 feet when the precast concrete units function as a gravity wall without reinforced backfill and no surcharge load is applied. When wall height is in excess of 4.5 feet or a surcharge is applied, geosynthetic reinforcement may be added to the modular blocks to create a geosynthetic-reinforced soil (GRS) wall.

Precast Concrete Block Gravity Walls without reinforced backfill should meet the design requirements of LRFD 11.11. If the backfill is reinforced, walls should meet the design requirements of LRFD 11.10 and BDG 5.6.5.4.B. Geosynthetic-Reinforced Soil Walls.

Blocks for modular block walls are made from wet cast concrete. Wall systems comprised of dry cast concrete are susceptible to degradation caused by freeze-thaw and are not an approved wall type. Precast concrete block gravity walls are not permitted in waterways.

5.6.5.4 MSE Walls

A. MSE Walls with Steel Reinforcement

This type of MSE wall uses galvanized strips or mats of steel to reinforce soil and create a reinforced soil block behind the wall face. The reinforced soil mass acts as a unit and resists the lateral loads through the dead weight of the reinforced mass. MSE walls are constructed from the bottom up and are therefore best suited for fill situations.

With a few exceptions, the procedure for the design of MSE walls using LRFD is identical to that followed using ASD. External stability evaluations include bearing resistance, sliding, and eccentricity. Internal stability calculations include pullout and rupture of reinforcements, capacity of reinforcement connections to the wall face, and structural capacity of the wall facing. MSE walls are typically designed by the wall manufacturer for internal and external stability. All MSE walls should be designed in accordance with:

1. LRFD Article 11.10

2. Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volumes I and II, November 2009, FHWA-NHI-10-024 and FHWA-NHI-10-025

3. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, November 2009, FHWA-NHI-09-087

4. Standard Specification Section 636 – Mechanically Stabilized Earth Retaining Wall

It is the responsibility of the Geotechnical Designer to assess the wall for bearing resistance, settlement, and global slope stability.

The calculation of lateral earth pressure on MSE walls should be as specified in AASHTO LRFD 3.11.5.8.

MSE walls with steel reinforcement and precast panels are relatively low in cost. These walls do require a high quality backfill with strict electrochemical requirements, as defined in the Standard Specifications Section 636 - Mechanically Stabilized Earth Retaining Wall. The base width of MSE walls is typically 70% of the wall height, which requires considerable excavation in a cut situation. Therefore, in a cut situation, base width requirements usually make MSE structures uneconomical and difficult to construct. It is best to limit the height to approximately 35 feet for routine projects.

Facing options depend on the aesthetic and structural needs of the wall system. Facing options typically include precast modular panels with various shapes and texturing options. The facing type used can affect the ability of the wall to tolerate settlement, depending on whether continuous vertical joints between adjacent panels are specified. Refer to Section 5.6.1.8 Aesthetics for further guidance.

MSE walls are inherently flexible and can tolerate moderate settlements without suffering structural damage, depending upon the MSE wall panel shape and alignment.

MSE walls are not appropriate if very weak soils are present that will not support the wall and that are too deep to be over excavated, or if a deep failure surface is present that result in slope instability. In these cases, a deep foundation or soil modification may be considered.

MSE walls may be used to retain soil supporting bridge substructures. The substructure units may be either spread footings or pile supported, with the following additional design criteria:

- The MSE wall shall be designed to provide a service life of not less than 100 years.

- For the analysis of spread footings on top of the reinforced soil zone, a factored bearing resistance of 7 ksf should be used for the strength limit state, and a factored bearing resistance of 4 ksf should be used to limit settlements to less than approximately 0.5 inch.

- A minimum distance of 4 feet should be provided between the bottom of the superstructure and the berm in front of the abutment breastwall or pile cap and behind the MSE top panel, for future bridge inspection and maintenance purposes.

- The minimum distance from the centerline of the bearing on the bearing on the abutment to the outer edge of the MSE wall facing should be 3.5 feet.

- A minimum distance of 2 feet should be provided between the back of MSE wall panels and the front face of abutment or pile cap.

- If the abutment is supported on piles or piles installed in sleeves, a minimum distance of 2 feet should be provided to allow compaction equipment to be used between piles or sleeves and the back face of panels, and to allow a 15° reinforcing strap skew to clear a typical 2-ft diameter pile sleeve.

- The top of the MSE panel in front of footings or pile caps should be set 1 foot above the berm elevation.

- If embedding spread footings for frost protection within the reinforced mass is impractical, provide at least 2 feet of soil cover and place the footing on a minimum 3-foot thick bed of compacted coarse aggregate.

- An impervious geomembrane consisting of low-permeability, 2-sided textured HDPE a minimum of 60 mils thick should be installed near the top of the reinforced soil zone to reduce the chance of water and salt-laden water infiltration into the reinforced backfill. The membrane should be bonded to the back face of the abutment, and sloped to shed water that infiltrates from the road surface.

- The need for fencing along the top of the wall should be investigated on a project by project basis.

Prior to selection of MSE walls for a project, consideration should be given to the location of any utility behind or within the reinforced soil backfill zone. It is best not to place utilities within the reinforced backfill zone because it would be impossible to access the utility from the ground surface without cutting through the soil reinforcement layers, thereby compromising the integrity of the wall. Coordination of the wall with project elements (such as drainage, utilities, luminaries, guardrail, or bridge elements) is critical to avoid costly change orders during construction. Moreover, failure of a sewer or water main located within an MSE wall mass could result in failure of the wall. As a result, MSE walls must not be used in areas where water and/or sewer utilities are present. It is also best to locate drainage features and signal or sign foundations outside of the MSE reinforced backfill zone.

Since MSE walls are proprietary and the wall vendor performs the design, it is imperative that the design requirements be clearly stated on the plans. If there are any unusual aesthetic requirements, design acceptance requirements, or loading conditions for which the wall needs to be designed, they should be clearly shown on the plans. Refer to Section 5.6.5.5 PS&E for Project with Proprietary Walls for plan development requirements.

MSE walls are measured and paid for by the area of wall face, as determined from the approved shop drawings. The high quality backfill and wall design are included in the MSE wall pay item. The Designer should consider this when comparing the cost of MSE walls with other wall systems, which typically pay for backfill as a separate pay item. Excavation is also paid for separately as common excavation. The Designer should consult the current Special Provision for measurement and payment information.

B. Geosynthetic-Reinforced Soil Walls

Geosynthetic-reinforced soil (GRS) walls are MSE or Precast Gravity Block walls with geosynthetic (polymeric) soil reinforcement. GRS walls are designed to create a reinforced soil volume behind a wall facing. Facing options include precast concrete modular panels or modular concrete blocks. Geosynthetic facings, although available, are not acceptable for permanent facing due to potential facing degradation when exposed to sunlight. Facings consisting of dry-cast concrete are susceptible to degradation caused by freeze-thaw and are not allowed. GRS walls are not permitted in waterways.

GRS walls are constructed from the bottom up and are therefore best suited for fill situations. The base width of GRS walls is typically 70% of the wall height, which requires considerable excavation in a cut situation.

It is best to limit the height of GRS walls to 20 feet or less for routine projects.

GRS walls have a low cost and can handle significant settlement. Compared to steel-reinforced systems, internal wall deformations may be greater and electrochemical backfill requirements less strict, but a high quality backfill is still required. Only geosynthetic products for which long-term product durability is well defined per LRFD 11.10.6.4 will be allowed.

GRS walls are proprietary and are designed by a wall manufacturer for internal and external stability. GRS walls shall be designed with a service life of not less than 75 years. The walls shall be designed in accordance with the following:

1. LRFD Article 11.10

2. Design of Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Volumes I and II, November 2009, FHWA-NHI-10-024 and FHWA-NHI-10-025

3. Corrosion/Degradation of Soil Reinforcements for Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, November 2009, FHWA-NHI-00-087

It is the responsibility of the Geotechnical Designer to assess the wall for bearing capacity, settlement, and global slope stability.

Since these preapproved walls are proprietary and the wall vendor performs the design, it is imperative that the design requirements for GRS wall be clearly stated on the plans. If there are any unusual aesthetic requirements, design acceptance requirements, or loading conditions or pressures for which the wall needs to be designed, they should be clearly shown on the plans. Refer to Section 5.6.5.5 PS&E for Project with Proprietary Walls for plan development requirements.

Coordination of the wall with project elements (such as drainage, utilities, luminaries, guardrail, or bridge elements) is critical to avoid costly change orders during construction. It is best to locate drainage structures and signal or sign foundations outside of the reinforced backfill zone.

5.6.5.5 PS&E for Project with Proprietary Walls

The PS&E package for a bridge project including proprietary wall item will include the following:

- General wall plan
- Wall profile, showing neat line top and bottom of the wall and final ground line in front of and in back of the wall
- Profiles showing the existing and final grades
- Typical wall cross section with generic details including batter
- Factored bearing resistance
- Foundation embedment criteria
- Leveling pad details
- General details for any desired appurtenances, such as coping or drainage requirements
- Project specific loads for other design acceptance requirements (examples: seismic loads, earth loads due to thermal movement of abutments)
- Special facing treatment (shape, texturing, color)
- Project-specific construction requirements (example: crushed stone)
- Highway approach cross sections showing only the face of the wall and footing

5.6.6 Geosynthetic Reinforced Soil Integrated Bridge Systems

GRS walls associated with Geosynthetic Reinforced Soil Integrated Bridge Systems (GRS-IBS) designed in accordance GRS-IBS Interim Implementation Guide, FHWA-HR-11-026, January 2011, may be considered for some bridges over waterways, with the approval of the Assistant Bridge Program Manager.

5.6.7 Anchored Wall Systems

5.6.7.1 CON/SPAN® Wingwall

CON/SPAN® wingwall systems may only be used in conjunction with CON/SPAN® precast drainage structures. The system consists of a precast face panel with a precast concrete soil anchor located near the base of the face panel. The wingwall system is connected to the CON/SPAN® drainage structure. The wall should be backfilled with granular borrow material suitable for underwater backfill and compacted per the Standard Specifications. The maximum wall height available is 16.5 feet, and should only be used with a level backfill surface and seismic loads less than a = 0.1g when a seismic analysis is required for design (ASCE, 2001). Refer to Section 3.7.2 Seismic Analysis for guidance.

The CON/SPAN® wingwall system should be designed in accordance with LRFD 11.9 Anchored Walls. The design requirements for the CON/SPAN® wingwall system should be included with the contract documents in Special Provision 534.

CON/SPAN® wingwall system should be placed on a footing, which serves both as a leveling slab and a structural foundation. This may include, but is not limited to a cast-in-place concrete footing, cast-in-place stub wall with footing, or a precast concrete footing meeting the requirements of Section 5.2.1 Frost, Section 5.3 Spread Footings, and Section 2.3.11 Scour. The footing should be sized to support the weight of the wall panels and weight of soil in and above the anchor system (ASCE, 2001).

The CON/SPAN® wingwall system should be equipped with a drainage system, consisting of a perforated drainage pipe installed in the backfill behind the wall, which outlets through a 4 inch diameter weep hole cast in the facing panel, per the manufacturer's requirements (ASCE, 2001).

5.6.7.2 Metal Structural Plate Headwall/Wingwall

Metal structural plate headwall/wingwall may only be used in conjunction with metal structural plate box culverts. However, preference should be given to the use of a PCMG wall system for increased durability. The headwall system consists of a metal structural plate face, which is connected to the top of the metal structural plate box with an anchor rod. The wingwall system consists of a metal structural plate face with a deadman connected to the face with a tie rod and whale system. The maximum wall height available is 14.25 feet.

The metal structural plate headwall/wingwall system should be designed in accordance with the most recent version of AASHTO LRFD. The design requirements for the metal structural plate headwall/wingwall system should be included with the contract documents.

5.6.8 Gabions

Gabion walls consist of stacked 3 feet cubed wire baskets, which are filled with stone. Groups of filled gabion baskets are stacked to construct a gravity wall. Gabion walls should be designed as specified in Section 3.6.7.2 Prefabricated Modular Walls. In designing gabion walls, a unit weight, γ, of 100 lb/ft^3 should be used for the weight of stone inside the baskets. Gabion walls should be backfilled with granular or gravel borrow. An angle of wall friction, δ, of 24° should be used for design. Wire for gabion baskets should be either PVC-coated or galvanized. A PVC coating is preferred as it does not flake off.

MaineDOT experience has shown that constructing gabion walls correctly can be costly and time-consuming. Disadvantages in the use of gabions include subjection to corrosion when placed in water and occurrence of vandalism by the cutting of the basket wires. Gabion walls should be used only in non-critical situations, in dry environments, and in rural areas, where the probability of corrosion and vandalism are less (MaineDOT, 2002). Gabion wall heights in excess of 6 feet are not recommended.

5.7 Piles

5.7.1 General

Piles should be considered when spread footings cannot be founded on bedrock or on competent soils at a reasonable cost. Piles should also be considered where soil conditions permit use of spread footings, but where the soils are susceptible to scour, liquefaction or lateral spreading.

Pile foundations should be designed so that the available factored geotechnical and drivability resistance is greater than the factored loads applied to the pile at the strength limit state. Service limit state design of driven pile foundations includes an evaluation of settlement, overall stability, lateral squeeze and lateral movement.

5.7.2 H-Piles

H-Piles used for bridge foundations should be comprised of rolled-steel sections of ASTM A572, Grade 50 steel, with a minimum yield stress of 50 ksi. Refer to Section 7.2.1 Structural Steel for H-pile material requirements.

5.7.2.1 Axial Resistance

The maximum factored axial design load applied to H-pile sections should not exceed the lesser of the factored structural pile resistance, the factored geotechnical pile resistance and the factored drivability resistance. The factored structural resistance of H-pile sections should be determined using a resistance factor, φ, listed below:

- Φ_c = 0.50 for axial resistance of piles in compression and subject to damage due to severe driving where use of a pile tip is necessary.

- Φ_c = 0.60 for axial resistance of piles in compression under good driving conditions, where use of a driving tip is not necessary.

For combined axial and flexural resistance of undamaged pile, the resistance factors are listed below:

- Φ_c = 0.70 for axial resistance of H-piles in compression.

- Φ_f = 1.00 for flexural resistance of H-piles.

The resistance factors, Φ_c and Φ_f, are to be used in interaction equations in LRFD 6.9.2.2.

The factored axial structural axial resistances of selected H-Pile sections are presented in Table 5-7. For the purposes of Table 5-7, the H-piles were assumed fully braced, and an effective length factor (K) of 1.0 was used. The Structural Designer should recalculate structural resistances for the upper and lower portions of the H-pile based on unbraced lengths and K-values from project specific LPILE® analyses and recalculate structural resistances. For preliminary design purposes, however, the resistances provided in Table 5-7 may be used to estimate the factored structural axial resistance of that portion of the pile which is theoretically in pure compression, i.e., that portion below the point of fixity.

Commentary: Experience in using 50 ksi steel for H-Pile foundations has shown that the factored axial geotechnical resistance frequently governs design. This is particularly apparent for end-bearing piles on poor-quality and/or soft bedrock and for friction piles.

Table 5-7 Factored Axial Structural Resistance of Selected H-Pile Sections

F_y = 50 ksi and fully braced

Pile Section	Factored Axial Structural Resistance	
	Good driving conditions $\Phi = 0.60$ (kips)	Severe driving conditions $\Phi = 0.50$ (kips)
HP 10x42+	372	310
HP 10x57	504	420
HP 12x53+	465	388
HP 12x63	552	460
HP 12x74	654	545
HP 12x84	738	615
HP 14x73+	642	535
HP 14x89+	783	653
HP 14x102	900	750
HP 14x117	1032	860

Note: Those marked + are preferred sections

The factored geotechnical and drivability resistances should be determined for site-specific conditions by the Geotechnical Designer. Consideration should be given to downdrag, soil relaxation, soil setup, lateral spreading and any other site-specific factors, which may affect the pile capacity during and after construction. The factored geotechnical resistance should be determined by applying a resistance, factor which is dependent on the design method.

5.7.2.2 Lateral Pile Resistance for the Service Limit State

Horizontal movement of pile groups induced by lateral loads shall be evaluated for Service Limit State Design. The lateral resistance of a pile is governed by the loading condition, pile stiffness, stiffness of the soil, and the degree of fixity. The lateral resistance (P_L) and depth to fixity (D_f), for service limit state design for selected H-Pile sections in sand and clay are presented in Table 5-8 and Table 5-9, respectively. The factored lateral resistances presented in Tables 5-8 and 5-9 assume a resistance factor of 1.0 and a maximum lateral deflection of 1/8 inch.

Commentary: The lateral resistance and depth to fixity presented in Tables 5-8 and Table 5-9 were determined using the computer program LPILE® Plus Version 4, the soil properties stated, a fixed condition at the pile head, an infinitely long pile, an applied axial load equal to As x 0.25 x F_y and a deflection of 1/8".

Table 5-8 Factored Lateral Resistance and Depth to Fixity for Strength Limit State Design for H-Pile Sections in Sand, φ=1.0

Pile Section	Loose		Medium Dense		Dense	
	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)
HP 10x42+	6.2	24	9.9	20	11.7	18
HP 10x57	7.1	26	11.4	22	13.6	19
HP 12x53+	8.1	28	13.3	24	16.1	20
HP 12x63	8.9	30	14.4	25	17.4	21
HP 12x74	9.4	31	15.6	25	18.9	22
HP 13x60	9.0	31	15.0	25	18.2	21
HP 13x73	9.8	32	16.4	26	20.0	22
HP 13x87	10.6	32	17.7	26	21.7	23
HP 14x73+	10.5	32	17.8	26	21.9	23
HP 14x89+	11.4	33	19.5	27	24.1	24
HP 14x102	12.3	35	20.9	28	25.9	25
HP 14x117	13.1	36	22.3	29	27.0	25

Note: Those marked + are preferred sections. P_L and D_f are determined assuming a friction angle, ϕ, of 32°.

Where the applied lateral load from the Service Limit State Load Combination exceeds that presented in Tables 5-8 and 5-9, or the pile length is less than the depth to fixity shown in the table, a more thorough analysis is recommended, using actual loading and soil conditions. Where soils differ from the conditions assumed in the tables, the Designer should complete a more thorough analysis.

Tables 5-8 and 5-9 present the lateral resistance and depth to fixity for a lateral load applied perpendicular to the pile flange. For conventional abutments and mass piers, H-piles should be oriented with the flange perpendicular to the substructure axis in the direction of the maximum applied lateral load. For conventional abutments and mass piers, where H-piles are oriented with the web perpendicular to the maximum applied lateral load, a thorough analysis of the foundation is recommended, using actual loading and soil conditions (Tables 5-8 and 5-9 do not apply). For integral abutments where the web is oriented perpendicular to the principal axis, the design should be in accordance with Section 5.4.2 Integral Abutments.

Table 5-9 Factored Lateral Resistance and Depth to Fixity for Service Limit State Design for H-Pile Sections in Clay, φ=1.0, Load Perpendicular to Flange

Pile Section	Soft[1]		Medium Stiff[2]		Stiff[3]	
	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)	P_L (kips)	D_f (ft)
HP 10x42+	5.1	22	9.2	18	13.1	16
HP 10x57	5.5	24	10.2	20	14.5	18
HP 12x53+	6.3	26	11.7	21	16.6	19
HP 12x63	6.7	27	12.4	22	17.6	19
HP 12x74	7.1	27	13.1	22	18.7	20
HP 13x60	7.0	27	12.8	22	18.2	19
HP 13x73	7.5	28	13.8	23	19.5	21
HP 13x87	7.9	29	15.6	25	20.7	21
HP 14x73+	8.1	29	14.8	24	21.0	21
HP 14x89+	8.7	31	15.9	25	22.5	22
HP 14x102	9.1	31	16.7	26	23.6	22
HP 14x117	9.5	32	17.5	26	24.8	24

Note: Those marked + are preferred sections.
[1]S_u = 375 psf, [2]S_u = 750 psf, [3]S_u = 1125 psf

5.7.3 Layout and Construction

The pile spacing should not be larger than is reasonable or practical. The center-to-center pile spacing should not be less than 30 inches or 2.5 to 3 times the pile diameter. A reasonable maximum spacing for piles in the back row of abutments is 12 feet.

Care should 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.

The distance from the side of any pile to the nearest edge of the pile cap shall not be less than 9.0 inches. The tops of piles should project at least 18 inches into the pile cap after all damaged pile material has been removed.

All piles should be equipped with a driving shoe. Refer to Standard Specification Section 501 – Foundation Piles for further guidance.

5.7.4 Concrete Piles

Concrete piles are used as displacement piles provided they can be driven without damage, that is, there are no boulders or hard driving dense soils. Two types of concrete piles are precast conventionally reinforced and precast prestressed piles. Both types are of constant cross section, though they may have tapered tips. Pile shapes include square, octagonal, and round sections and may be either solid or hollow. Typical pile cross sections used range from 10 inches to 16 inches, but sizes above and below this range are also produced. Refer to LRFD Article 5.13.4, Concrete Piles, and FHWA, 1998 for detailed information regarding concrete piles.

Precast concrete piles are suitable for use as friction piles when driven in sand, gravel, or clays. Precast concrete piles are capable of high capacities when used as end bearing piles. In boulder conditions, a short piece of structural H-pile section or “stinger” may be cast into or attached to the pile for penetration through the zone of cobbles and boulders.

Conventionally reinforced concrete piles (concrete with reinforcing steel bars and spiral reinforcing steel cages) are susceptible to damage by mishandling or driving. Prestressed concrete piles are more vulnerable to damage from striking hard layers of soil or obstructions during driving than conventionally reinforced concrete piles. Piles should be equipped with a metal driving shoe for hard driving conditions. High stresses during driving can cause cracking in all concrete piles.

Precast piles are difficult to splice, particularly prestressed piles. Accurate knowledge of pile lengths is required when using concrete piles, as they are also difficult to shorten. Special precautions should be taken when placing concrete piles during cold weather. Temperature gradients can cause concrete to crack due to non-uniform shrinkage and expansion.

A concrete pile foundation design should consider that deterioration of concrete piles can occur due to sulfates in soil, ground water, or sea water; chlorides in soils and chemical wastes; or acidic ground water and organic acids. 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 ground water samples is recommended when chemical wastes are suspected.

5.7.5 Steel Pipe Piles

5.7.5.1 Design - General

The maximum factored applied axial load on any pipe pile shall not exceed the lesser of the factored structural compressive resistance, the factored axial geotechnical resistance and the factored drivability pile resistance. For the strength limit state, the factored axial compressive structural resistance of pipe piles ($P_{r)}$ shall be estimated using the following resistance factors (Φ_c):

- Φ_c = 0.60 for piles subject to damage in severe driving conditions where use of a pile tip is necessary
- Φ_c = 0.70 for piles under good driving conditions where use of a pile tip is not necessary

The nominal compressive structural resistance (P_n) for pipe piles loaded in compression should be estimated as specified in LRFD 6.9.5.1 using the column slenderness factor, λ.

At the strength limit state an axial resistance factor, Φ_c, of 0.80, and a flexural resistance factor, Φ_f, of 1.0 should be applied to combined nominal axial and flexural resistance in the interaction equation in LRFD 6.9.2.2.

5.7.5.2 Material and Design Section

Pipe piles consist of seamless, straight butt-welded or spiral 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. Typical wall thicknesses are limited to the range of

1/2” to 1 inch. MaineDOT practice has commonly limited their use to 24 to 32 inch diameters when used in pier bents. All pipe piles are filled with Class A concrete after driving. Additionally, pipe piles employed as pier bents are internally reinforced with a reinforcing cage.

Concrete filled pipe piles have a high load-carrying capacity and provide high bending resistance where an unsupported length is subject to lateral loads. For design criteria and corrosion protection of pipe piles in pier bents, refer to Section 5.5.2.5 Pile Protection and 5.5.2.6 Pipe Pile Coatings and Cathodic Protection.

Pipe piles may be driven open or closed ended. If the capacity from the full pile toe is required, the pile should be driven closed ended, with a flat plate or conical tip. Closed ended types are preferred, except if the pile is designed as a friction displacement pile.

If obstructions are expected, the pile should 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 and can significantly increase the pile toe resistance. Piles driven open-ended should be cleaned, leaving a length of soil plug ranging from two to three pile diameters, and filled with concrete after driving.

Steel pile material should conform to ASTM A252 Grade 3. Open-ended piles should be reinforced with steel cutting shoes to provide protection against damage. When pipe piles are driven to weathered bedrock or though boulders, an end plate or conical point with a rounded nose is often used to prevent distortion of the pile nose. End closures should 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 socked into bedrock. Anchoring pipe piles with rock dowels or anchors is not recommended and should only be considered when the preceding alternatives are found to be not feasible.

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

5.7.6 Downdrag

Where the soil deposit in which piles are installed is subject to settlement, downdrag forces may be induced on piles. As little as 1/2” of differential settlement may induce downdrag forces. Downdrag loads reduce the usable pile capacity. Possible development of downdrag loads on piles should be considered when:

- Sites are underlain by compressible clays, silts, or peats
- Fill has been recently placed on the surface
- The groundwater has been substantially lowered

Downdrag loads should be considered as permanent additional axial loads when the nominal bearing capacity of the pile foundation is evaluated, and when settlement of the pile foundation is evaluated.

To calculate downdrag loads on piles, the traditional approach is the total stress α-method, which is used for computing downdrag in cohesive soils. Newer methods are based on the relationship between pile movement and negative shaft resistance, and described in Briaud and Tucker (1993). The downdrag loads should be factored by the appropriate load factor for downdrag, γ_p, and added to the factored vertical dead load applied to the pile.

If downdrag forces are significant, they can be reduced by applying a thin coat of bitumen of the pile surface (Dixon, et. al., May 1998). Battered piles should be avoided where downdrag loads are expected due to induced bending moments in response to settlement. These bending moments can result in pile deformation. In situations where downdrag forces cannot be reduced by applying bitumen coating, the Designer should consider:

- Forcing soil settlement prior to driving piles by preloading and consolidation the soils
- Using lightweight fills
- Increasing the pile size
- Sleeve piles

5.7.7 Pile Installation Quality Control and Nominal Pile Resistance

The nominal resistance a pile is driven to in the field is a function of the level of quality assurance/control provided during construction operations. The resistance factors for nominal pile resistances are presented in Table 5-10. These resistance factors are based upon construction quality control beyond the standard subsurface exploration and static pile capacity analysis.

Table 5-10 Resistance Factors for Driven Piles

Construction Control Method	Resistance Factor, φ_{dyn}
Static load test of at least one pile, with dynamic testing of at least 2% to 5% of the production piles.	0.80
Dynamic testing with signal matching of at least 1 pile per substructure, but no less than 2 dynamic pile tests from opposite corners for substructures longer than 40 feet or with more than 15 piles, but no less than 2% of the production piles at any one site, and up to 5% of the production piles for sites with moderate to highly variable subsurface conditions.	0.65
Wave equation analysis without dynamic measurements or load test	0.40

A pile group is classified as nonredundant if there are less than five (5) piles in the group. If a pile group is nonredundant, past LRFD practice dictated a 20 percent reduction of the pile resistance factors, φ_{dyn}, provided in Table 5-10, and should be considered to provide a uniform level of safety.

Pile testing programs should include, at a minimum, wave equation analyses. Wave equation analyses confirm that the design pile section can be installed to the desired depth and ultimate capacity, without exceeding allowable pile driving stresses, with an appropriate driving system and criteria.

In addition to wave equation analyses, pile testing programs should also include dynamic load tests or, rarely, static load tests. Dynamic testing with signal matching should be considered in order to:

- Field-verify the nominal pile axial resistance
- Establish driving criteria
- Monitor piles installed in difficult subsurface conditions, such as soils with obstructions and boulders, or a steeply sloping bedrock surface
- Verify consistent hammer operation during extended pile installation operations
- Justify higher resistance factors

In general, the pile testing program should be commensurate with the design assumptions; for example, at least 1 pile per bearing stratum will be tested.

Pile testing programs should specify the number, location, and time of all dynamic tests and/or static pile tests. When a dynamic load test program is specified, the following requirements shall apply:

- For large pile groups with more than 20 piles, the first and second pile tests shall be conducted at opposite corners of the substructure, and at least one additional dynamic test shall be conducted mid-production, after approximately one half of the production piles have been installed.

- Post-driving analyses (CAPWAP) are required.

- Provisions for 24 to 72 hour pile restrikes shall be included, for substructures where setup or relaxation effects are expected.

- Provisions for 24 to 72 hour dynamic restrike tests are mandatory for friction piles or piles designed to end bear in any strata other than bedrock.

- Provisions should be provided for the conduct of additional dynamic load tests during production, for field verification that the driving criteria are consistently achieving the required nominal pile resistances.

A minimum of 2% of the piles shall be tested when dynamic (or static) testing is specified. It may be necessary to test 5% or more piles, when there are more than 20 piles in a substructure, when difficult driving is expected, when variable or inconsistent soil conditions are expected, or when additional tests during production are necessary to verify hammer performance and geotechnical resistances.

The establishment of the driving criteria should include limiting driving stresses to the following thresholds:

- For steel piles in compression and tension, driving stresses should not exceed 90% of the yield strength of the pile material. For 50 ksi steel, this results in a maximum driving stress of 45 ksi.

- For concrete filled pipe piles, if unfilled when driven, driving stresses should not exceed 90% of the yield strength of the steel shell material.

- For concrete piles, driving compressive stresses should not exceed 0.85 times the concrete compressive strength. Tensile stresses during driving should not exceed 0.70 times the yield strength of the steel reinforcement.

- For prestressed concrete piles, driving compressive stresses should not exceed 0.85 times the concrete compressive strength minus the effective prestress. Tensile stresses during driving should be limited to 0.095 times the square root of the compressive strength (ksi) plus the effective prestress.

5.8 Drilled Shafts

Drilled shafts may be an economical alternative to spread footings or pile foundations. 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 and rock-socketed deep foundations are needed.
- 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 elimination of waterline footings is advantageous and possible by extending drilled shafts as a column up to the pier cap.
- The foundation is required to resist high lateral loads or uplift loads.
- There is little tolerance for deformation.
- The cost and constructability of seals and caps for pile supported structures is high.

Although there are many references for the design and analysis of drilled shafts, MaineDOT follows the procedures found in FHWA, 2010 and LRFD Article 10.8.

The structural design of drilled shafts is similar to the LRFD method for a column with axial load and bending, and shear. Interaction diagrams should be developed to assess resistance to combined axial and bending.

The Bridge Program has developed a Special Provision to govern the construction of drilled shafts. Consult the Geotechnical Designer for the current version.

5.9 Embankment Issues

Embankment design considerations include settlement, slope stability, and bearing capacity at the base. Special design requirements for embankments will be presented in the Geotechnical Report. The Geotechnical Designer should review plans to determine any special design requirements with regard to an embankment.

5.9.1 Embankment Settlement

The embankment settlement should be evaluated using the methods discussed in Section 5.3.6 Settlement and must be within tolerable limits. Differential settlement is more of a concern than total settlement and should be evaluated by the Geotechnical Designer. Tolerable settlement also depends upon the structural integrity of the bridge or culvert and should be coordinated with the Structural Designer.

If settlement exceeds the tolerable limits, or the time needed to allow for settlement is excessive, several methods to address this are available to the Designer:

- Compressible materials can be removed and replaced to limit settlements.
- Preloads alone or in combination with surcharge can be used to complete settlements prior to construction.
- Prefabricated vertical drains can be used in conjunction with preloads to accelerate settlements.
- Lightweight fill materials such as tire shreds, geofoam or light weight concrete fill can be used.

The use of a preload, surcharge, or prefabricated vertical drains should be accompanied by the use of instrumentation (settlement platforms, piezometers, inclinometers) to assist in determining that an acceptable level of consolidation has taken place.

5.9.2 Embankment Stability

Embankment stability problems most often occur where embankments are to be built over soft weak soils such as low strength clays, silt, or peats. There are three major types of instability that should be considered in the design of embankments over weak foundation soils: circular arc failure, sliding block failure, and lateral squeeze. These stability problems are defined as “external” stability problems. “Internal” stability problems generally result from the selection of poor quality materials and/or improper placement requirements. Refer to Section 5.3.7 Overall Stability for methods of analysis.

Once the soil profile, soil strengths, and depth of water table have been determined by both field explorations and field and laboratory testing, the stability of the embankment can be analyzed. The evaluation of slope stability of earth slopes with or without a foundation unit should be investigated at the Service I Load Combination and an appropriate resistance factor. The resistance factor, φ, may be taken as:

- 0.75 - where the geotechnical parameters are well defined, and the slope does not support or contain a structural element

- 0.65 – where the geotechnical parameters are based on limited information or the slope contains or supports a structural element.

Available slope stability programs produce a single factor of safety. In light of this, the past practice of checking overall slope stability using ASD methods may be continued to insure that slopes and slopes with footings have a factor of safety equivalent to 1.3 and 1.5, respectively.

If the load and resistance balance cannot be met, several methods to improve stability can be undertaken:

- Removal and replacement of the weak material

- Use of a mid-slope berm or other variations of berms

- Soil reinforcement with steel, geogrid, or geotextile

- Installation of prefabricated vertical (wick) drains, sand drains, or stone columns

- Instrumentation and control of embankment construction

- Installation of a structural support such as a retaining wall

Lateral squeeze can occur when the lateral movement (consolidation) of soft soils transmits an excessive lateral thrust, which may bend or push an adjacent substructure. The best way to minimize lateral squeeze is to complete embankment settlements prior to construction of adjacent substructures.

5.9.3 Embankment Bearing Capacity

The embankment bearing resistance should be evaluated using the methods discussed in Section 5.3.5 Bearing Resistance. The factored bearing resistance should equal or exceed the factor applied loads.

5.9.4 Embankment Seismic Considerations

Currently, there are no LRFD codes for embankment seismic design. Therefore, using allowable stress design methods, a minimum seismic factor of safety of 1.0 is acceptable for slope stability and liquefaction. Refer to Section 3.7.4 Embankment & Embankments Supporting Substructure Units. Should poor seismic performance of an embankment impact the overall serviceability or performance of a critical structure the Department may specify a higher level of seismic performance or specify appropriate seismic provisions.

If the seismic slope stability factor of safety falls below 1.0 using the seismic coefficient-factor of safety method, a permanent seismic deformation analysis should be conducted using the Newmark Method (Newmark, 1965). This method approximates the cumulative vertical deformation or settlement at the back of the slope for a given earthquake ground motion. The failure mass is modeled as a block on a plane. A maximum allowable seismic settlement of 6 inches at a bridge approach, resulting from the design earthquake event, is considered acceptable. Refer to Section 3.7 Seismic for loading considerations.

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