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Bridge Design Guide, Chapter 5 : Substructures, 2003

Maine Department of Transportation

Chapter 5

SUBSTRUCTURES

Smith Bridge, Houlton

West Bridge, Fairfield-Benton

5 SUBSTRUCTURES

5.1 Terminology

B	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
FS_{SL}	sliding factor of safety
FS_{OT}	overturning factor of safety
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_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_A	horizontal sliding force
Q_t	horizontal soil shear resistance along footing base (force)
$q_{allowable}$	allowable bearing pressure
$Q_{applied}$	applied footing load
R	resultant force at base of footing
R_g	beam/girder rotation (radians)
S_p	section modulus of the pile
t	footing thickness
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 sea
α	batter angle from the horizontal plane
β	backfill slope
δ	friction angle between soil/bedrock and concrete
γ	soil weight
ϕ	soil internal angle of friction
σ_p	pile stress
σ_v	vertical bearing stress at base of footing
τ	horizontal superstructure forces transmitted through bearing at wall top

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

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

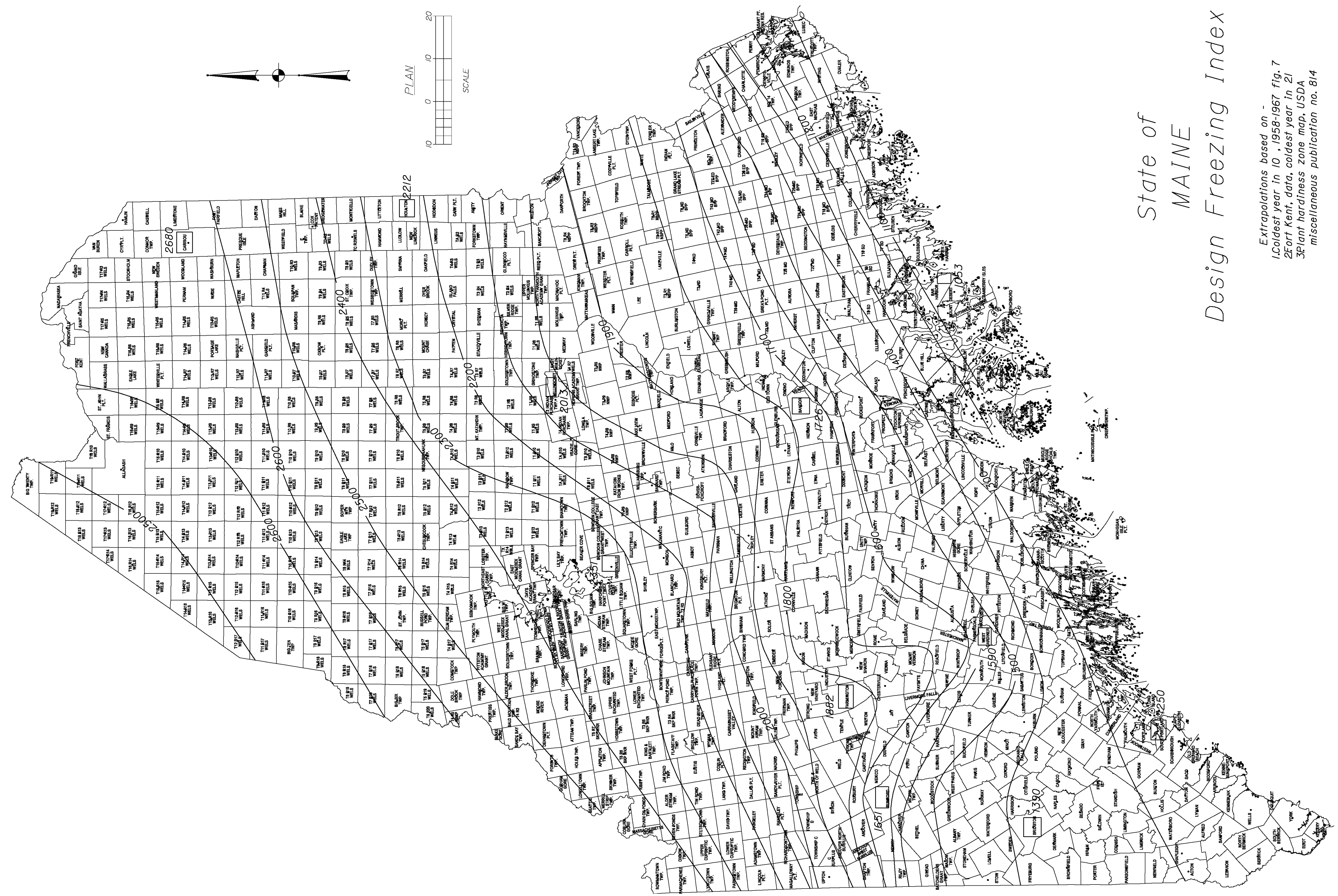


Figure 5-1 Maine Design Freezing Index Map

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 = 54 inches = 4.5 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. 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:

$$150 \cdot y = 62.4 \cdot z$$

where:

150 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

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. 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 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.

5.3 Spread Footings

Spread footings should be designed to support all live and dead loads and earth and water pressure loadings in accordance with the general principles specified by the <u>AASHTO Standard Specifications</u>. The geotechnical design should be made with reference to service loads and allowable stresses as provided in Service Load Design. Selection of foundation type is based on an assessment of the magnitude and direction of loading, depth to suitable bearing materials, evidence of previous flooding, potential for liquefaction, undermining or scour, frost depth, and ease and cost of construction.

Foundations should be designed to provide adequate structural capacity, adequate foundation bearing capacity with acceptable settlements, and acceptable overall stability of slopes adjacent to the foundations. The tolerable level of structural deformation (differential settlement) is controlled by the type and span of the superstructure.

Footings should be designed so that the pressure under the footing is as nearly uniform as practicable. The distribution of soil pressure should be consistent with properties of the soil or bedrock and the structure, and with established principles of soil and rock mechanics.

A footing should 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.

5.3.1 Footing Depth

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

5.3.1.1 Bearing Materials

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.

5.3.1.2 Footings on Bedrock

Footings should be founded on a single bearing material throughout the 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. For footings resting 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 6H/1V should be step-serrated or suitably benched.

5.3.1.3 Frost Protection

Footings will be placed below frost level 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.

5.3.1.4 Scour Protection

Footings at stream crossings should be founded at a depth at least 2 feet below the maximum calculated depth of scour. Refer to Section 2.3.11 Scour for information regarding scour depth.

5.3.2 Bearing Capacity

Spread footings should be designed to support design loads with adequate bearing and structural capacity, and with tolerable settlements. Bearing capacity of foundations may be estimated using procedures outlined in AASHTO Standard Specifications Article 4. The use of Terzaghi, Meyerhof, or Vesic methods for computation of ultimate bearing capacity is recommended. Consideration of shape factors, inclined loads, ground surface slope, and eccentric loading should be included in the calculation, if applicable. A minimum factor of safety for bearing capacity of 3.0 should be used for spread footings. Structures should be designed not to exceed the maximum soil or bedrock pressure under footings in accordance with the recommendations of the Geotechnical Designer.

5.3.3 Settlement

Settlement may be estimated using procedures described in AASHTO Standard Specifications Article 4 or other generally accepted methods. 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”.

Elastic settlement should be determined using the unfactored dead load, plus the unfactored component of live and impact loads assumed to extend to the footing level. 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.

Settlement of spread footings on sand can be predicted using calculation methods by Hough, Peck-Bazaraa, D'Appolonia, or Schmertmann, as 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. Tolerable movements are frequently described in terms of angular distortion between members. Per AASHTO Standard Specifications Article 4, angular distortion (δ'/ℓ) between adjacent footings should be limited to 0.005 for simple span bridges and 0.004 for continuous span bridges.

5.3.4 Stability

The overall stability of spread footings on or near a slope should be evaluated by limiting equilibrium methods of analysis, which employ the Modified Bishop, simplified Janbu, Spenser, or other generally accepted methods of slope stability analysis. Where soil and rock parameters and groundwater levels are based on in-situ and/or laboratory tests, the minimum factor of safety should be 1.3 (1.5 where abutments are supported above a slope). Otherwise, the minimum factor of safety should be 1.5 (1.8 where abutments are supported above a retaining wall).

Failure for 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 soil may be removed due to scour or during future construction. 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 50% of the maximum passive pressure resistance computed using Rankine Passive resistance.

Spread footings should be designed to achieve a factor of safety against sliding of at least 1.5. The coefficient of friction for sliding should be as shown in Table 3-3 for the soil type under the footing. For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding. If smooth bedrock is present at the bearing elevation, the bedrock should be stepped or doweled to improve stability. If sloping bedrock is present at the bearing elevation, the bedrock should be stepped or doweled to improve stability.

5.3.5 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.

5.3.6 Drainage Considerations

Adequate drainage of materials behind structures is of great importance and should be provided as described in Section 5.4.1.4 Drainage.

5.3.7 Seismic Considerations

Seismic hazards should be assessed as a part of the foundation type process. Per AASHTO Standard Specifications, seismic design and analysis is not required for single span bridges (classified as SPC A bridges) regardless of seismic zone. Refer to Section 3.7.2 Seismic Analysis for design considerations for other classified bridges and seismic zones.

5.4 Abutments

5.4.1 Conventional Abutments

5.4.1.1 General Design Requirements

When appropriate, 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, overturning, 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.

5.4.1.2 Loads

Abutments should be designed in accordance with either the AASTHO Standard Specifications, or the AASHTO LRFD Specifications, depending on the component being analyzed. Structural analyses and design of reinforced concrete for substructures will be computed using Load and

Resistance Factor Design (LRFD). For geotechnical evaluation and design of substructures, such as overturning, sliding, bearing pressure, global stability, and pile design, use the Service Load Design method (Allowable Stress Design(ASD)), except that the unfactored live load is calculated using LRFD loading. Loading combinations for the ASD methods are presented in Table 3.22.1A of the AASHTO Standard Specifications. Loads should be determined in accordance with AASHTO LRFD, and as outlined in Chapter 3 Loads. The following Service Load Cases will be evaluated:

- *Load Case I:* dead load plus earth pressure without superstructure

- *Load Case II*: dead load plus earth pressure, finished grade (including the vertical component of the dead load of the superstructure, approach slab, and the vertical component of the live load from superstructure)

- *Load Case III:* dead load plus earth load plus live load (same as Load Case II but also with live load effects of traffic on approach), finished grade

Anticipated construction loadings should also be investigated. For the abutment analysis, the typical construction loading conditions used look at the abutment partially backfilled without the superstructure in place. For the load condition 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.

Longitudinal forces for abutment design should include any live load longitudinal forces developed through bearings such as braking forces, or others as specified in AASHTO LRFD Section 3.0.

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. Surcharge Loads

Abutments without approach slabs should be designed with a live load surcharge when computing horizontal earth pressure. This additional lateral earth pressure is approximated by a surcharge equal to a height, H_{eq}, or earth fill. Refer to Section 3.6.8 Surcharge Loads for guidance in computing this additional lateral earth pressure.

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

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.

5.4.1.3 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.

5.4.1.4 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) through the walls. Refer to Standard Specification Section 512 – French Drains.

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

5.4.1.5 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.

AASHTO LRFD Section 5.10.8 does not apply in the design of conventional abutments, wingwalls, and retaining walls. Instead, #5 bars at 18 inches are used as temperature and shrinkage reinforcement in the stem walls of conventional abutments, wingwalls, and retaining walls.

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.

5.4.1.6 Factors of Safety

Factors of safety for abutments founded on spread footings and pile foundations should be as specified in Table 5-2.

Table 5-2 Minimum Factors of Safety

	Factor of Safety (minimum)	
	Spread Footing Foundation	**Pile Foundation**
Bearing capacity	3.0	NA
Sliding	1.5	NA
Overturning (eccentricity)	2.0	NA
Global stability (slope)	1.3	1.3

The ultimate capacity should be used in designing foundations for seismic loads. Consideration should be given to the amount of seismic settlement or translation the bridge can withstand.

5.4.1.7 Abutment Spread Footings

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

A. Spread Footings on Bedrock

Refer to Section 5.3.1.2 for guidance on the design of spread footings on bedrock.

B. Vertical and Horizontal Displacement

Vertical and horizontal movement criteria for abutments should be developed consistent with the function and type of structure and AASHTO Standard Specifications Figure 7.5.4A. Angular distortions and settlements should be designed per Section 5.3.3 Settlement.

C. 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.

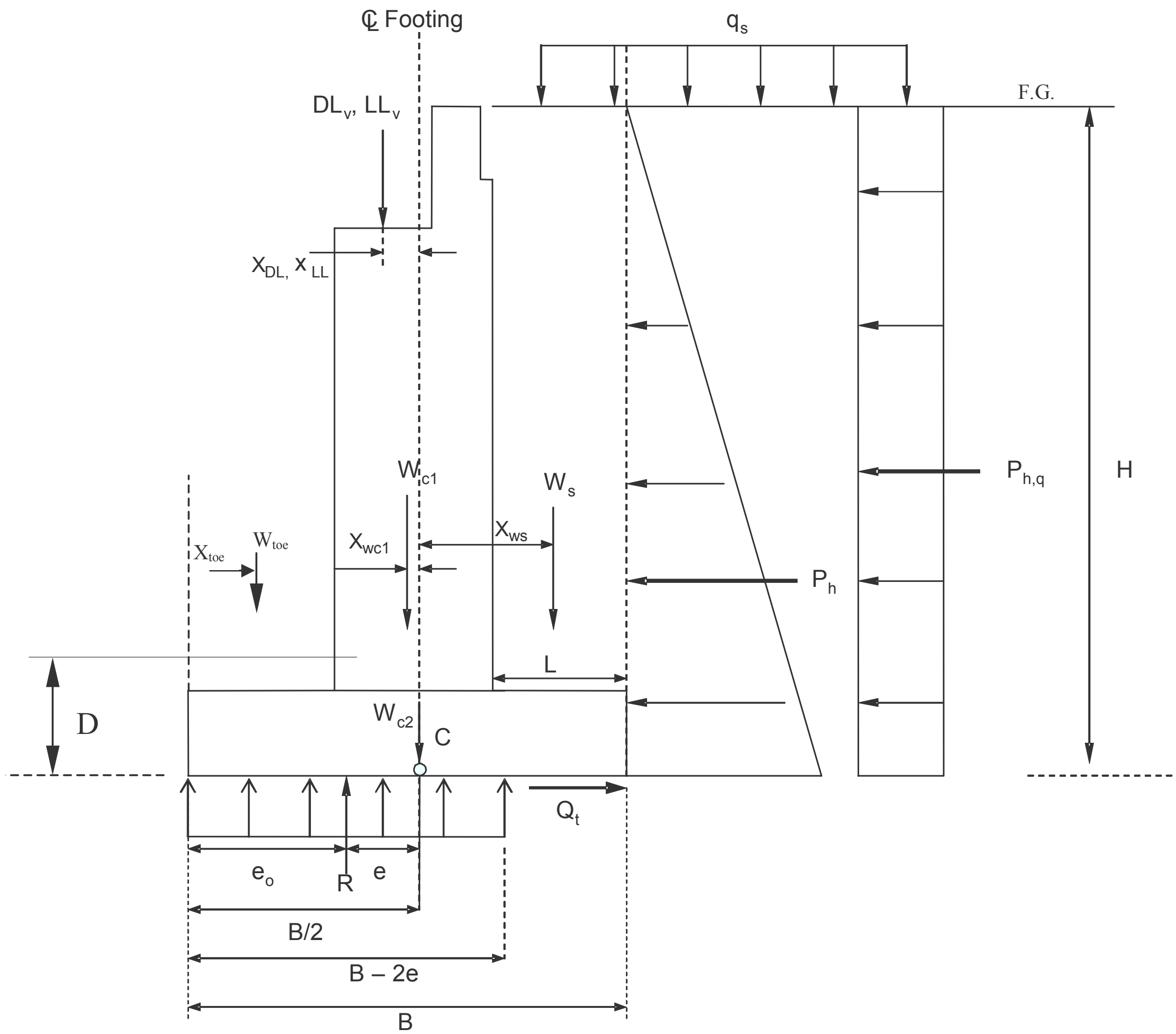
Global stability of walls and abutments should be investigated at the service limit state. Limit equilibrium methods that use Modified Bishop, Simplified Janbu, and Spencer methods are acceptable. A minimum factor of safety of 1.3 should be used for walls for static loads that do not support abutments. A factor of safety of 1.5 should be used for walls that support abutments.

D. Bearing Pressure

Maximum bearing pressure under footings at the design service loads should be determined per Section 5.3.2 Bearing Capacity. Structures should be designed to not exceed maximum soil or rock pressure under footings in accordance with recommendations of the Geotechnical Designer.

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 capacity. Refer to Procedure 5-1 and Procedure 5-2 for how to calculate bearing pressure.

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



Step 1. Calculate e, where:

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

$$M_o = P_h \cdot \frac{H}{3} + P_{h,q} \cdot \frac{H}{2} + W_{c1} \cdot X_{wc1}$$

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

$$M_r = W_s \cdot X_{ws} + q_s \cdot L \cdot X_{ws}$$

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

$$\sum V = W_s + W_{c1} + W_{c2} + W_{toe} + q_s \cdot L$$

and,

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

Step 2. The 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:

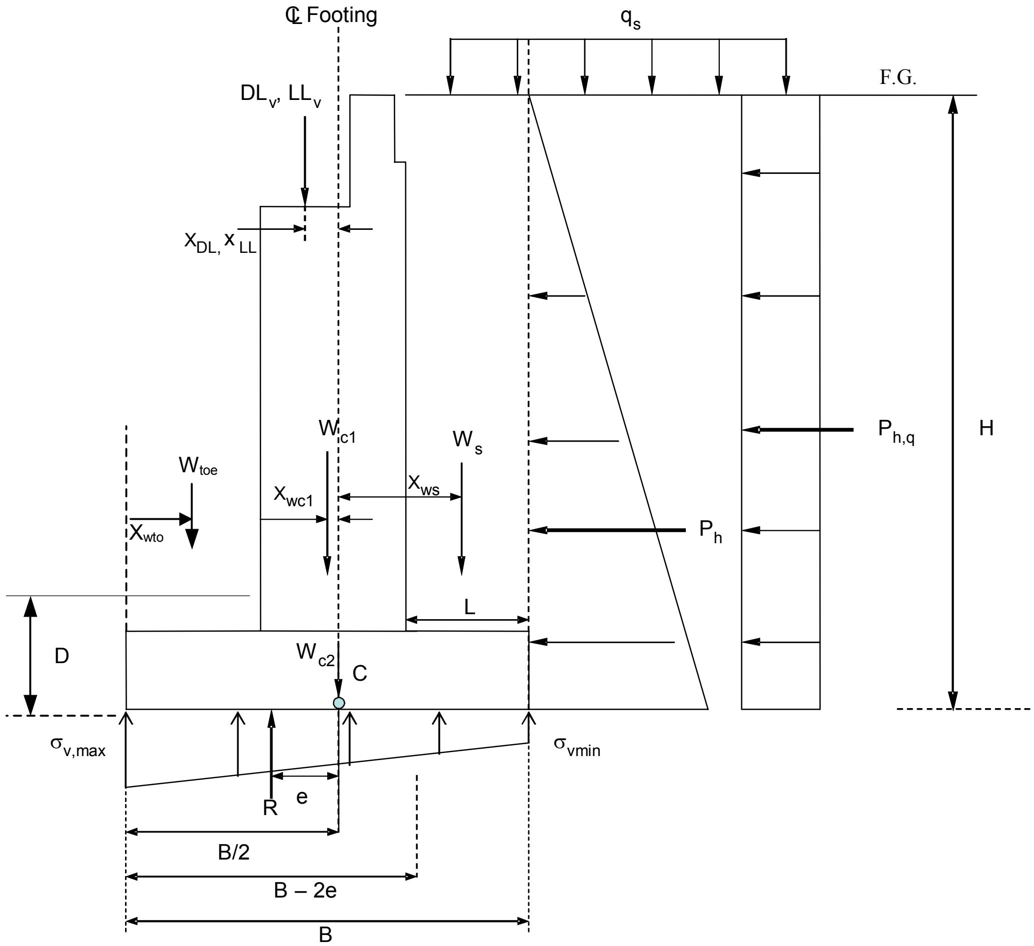
$$\sigma_v = \frac{\sum V}{B - 2e}$$

Step 3: Compare σ_{vmax} to the allowable bearing pressure provided in the Geotechnical Report. The maximum stress should be less that the allowable bearing stress.

$$\sigma_{v\max} \le q_{allowable}$$

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

Procedure 5-2 Bearing Pressure on Bedrock
For Conventional Abutment



Step 1: Calculate e, where:

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

$$M_o = P_h \cdot \frac{H}{3} + P_{h,q} \cdot \frac{H}{2} + W_{c1} \cdot X_{wc1}$$

M_r = sum of moments resisting forces about Point C:

$$M_r = W_s \cdot X_{ws} + q_s \cdot L \cdot X_{ws}$$

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

$$\sum V = W_s + W_{c1} + W_{c2} + W_{toe} + q_s \cdot L$$

and,

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

Step 2: The 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 vertical stress is calculated as follows:

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

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

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.

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

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

Step 3: Compare σ_{vmax} to the allowable bearing pressure (q_{allow}) provided in the Geotechnical Report. The maximum stress should be less that the allowable bearing stress (q_{allow}).

$$\sigma_{v\max} \leq q_{allowable}$$

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

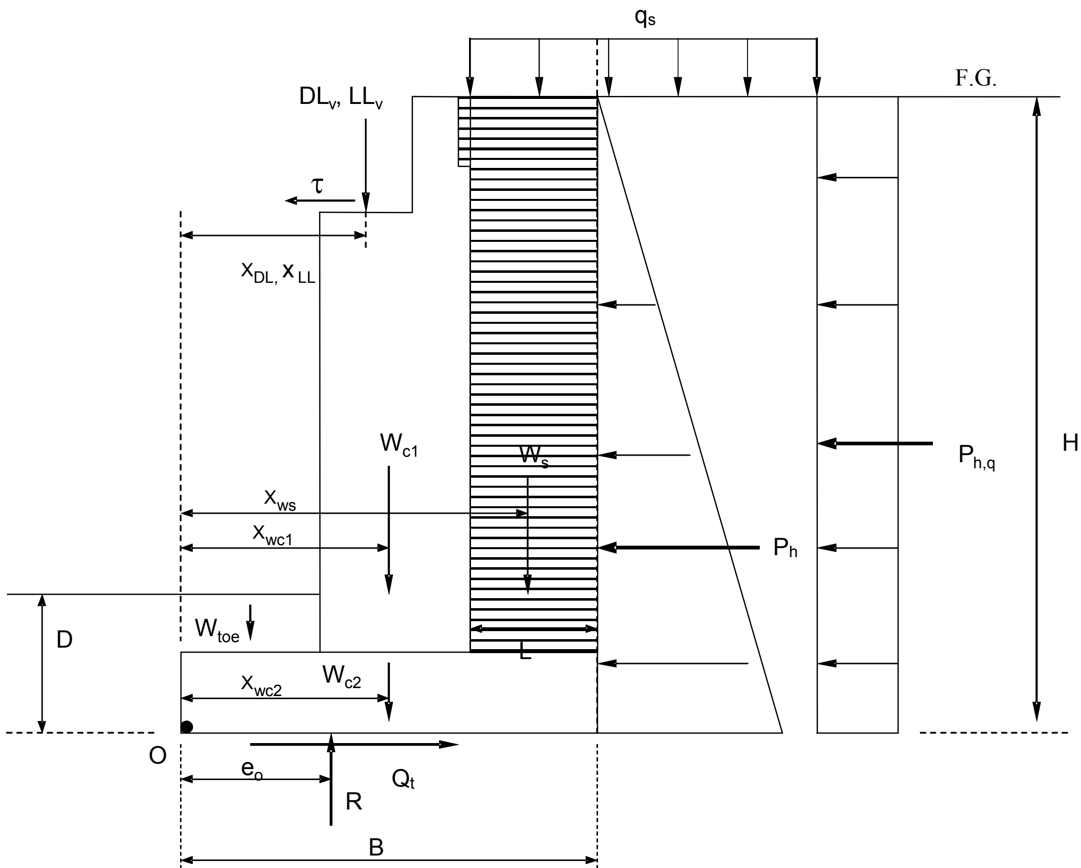
E. Sliding

Failure for 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 should be neglected in consideration that soil may be removed during future construction. Refer to Section 3.6.9 Passive Earth Pressure Loads for guidance. Abutments and walls on spread footings should be designed to achieve a factor of safety against sliding of at least 1.5. The factor of safety against sliding should be calculated as shown in Procedure 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 Overturning Stability and Sliding

For Conventional Abutment on Spread 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 positive.

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

$$M_o = P_h \cdot \frac{H}{3} + P_{h,q} \cdot \frac{H}{2} + W_{c1} \cdot X_{wc1} + \tau \cdot h$$

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

$$M_r = W_s \cdot X_{ws} + W_{c1} \cdot X_{wc1} + W_{c2} \cdot X_{wc2} + q_s \cdot L \cdot X_{ws}$$

$\sum V$ = sum of vertical forces action on footing and wall, as defined in the figure above.

$$\sum V = W_s + W_{c1} + W_{c2} + q_s \cdot L$$

and,

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

The resultant force at the base of the footing should be within the middle 1/2 if the footing dimensions for footings on soil and the middle 3/4 of the footing dimensions for footings on bedrock. For footings subjected to biaxial loading, these eccentricity requirements apply in both directions.

Step 2: Calculate the factor of safety against overturning.

$$FS_{OT} = \frac{M_r}{M_o}$$

Step 3: Calculate the factor of safety against sliding:

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

$$FS_{SL} = \frac{Q_t}{Q_{applied}}$$

where:
δ = friction angle between the footing base and the soil (refer to Table 3-3 or the Geotechnical Designer will provide a coefficient of friction for sliding.)
$Q_{applied}$ = applied footing load (obtained from the Structural Designer)

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

F. Overturning

Abutments and walls on spread footings should be designed such that the factor of safety against overturning is a minimum of 2.0. The factor of safety against overturning should be calculated as shown in Procedure 5-3.

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

5.4.1.8 Abutments Supported on Pile Foundations

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

For pile supported abutments, the applied service loads and moments (causing maximum and minimum compression in the piles) should be applied, and the resulting pile reactions and pile stresses determined. The maximum axial pile reaction should not exceed the allowable geotechnical capacity or allowable structural capacity, whichever controls. In accordance with AASHTO Standard Specifications Article 4.5.7.3, the resulting pile stresses should not exceed the allowable stress of $0.33F_y$, for non-integral structures. If greater stresses result, more piles, or larger piles, should be considered. The maximum lateral pile reactions should not exceed the allowable lateral load specified in Section 5.7.1.2 Lateral Capacity. Lateral loads that do exceed the allowable limits 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 HP piles should be evaluated by the Geotechnical Designer. Buckling analyses of piles should be performed by the Structural Designer.

Where abutments are required in water channels, the bottom of seal should be a minimum of 2 feet below the maximum calculated scour depth as described in the AASHTO Standard Specifications. 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 calculated 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.

5.4.1.9 Bridge Seat Dimensions

As a minimum, the bridge seat dimensions should meet the requirements of AASHTO Standard Specifications Division 1-A Section 3.10. Otherwise, for bridge seats supporting steel superstructures exceeding 100 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 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.

Bridge seats that are protected from roadway drainage by sealed bridge joints should be level and stepped to match bearing elevations, except where access to the space between end diaphragms and backwalls is difficult. In that case, the concrete pedestal type bridge seat may be used.

Bridge seats that are not protected from roadway drainage 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 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

Integral abutments 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.

5.4.2.2 Loads

Analysis and design of integral abutment substructures will be in accordance with either the AASHTO Standard Specifications or AASHTO LRFD, depending upon the component being analyzed. Structural analyses and design of reinforced concrete will be computed using LRFD. For geotechnical evaluations and design of substructures, such as global stability and pile design, use ASD, except that the unfactored live load is calculated using LRFD loading. Loading combinations for the ASD methods are presented in Table 3.22.1A of the AASHTO Standard Specifications. Loads should be determined in accordance with the AASHTO LRFD Specifications, and as outlined in Chapter 3. Refer to Procedure 5-4 and Example 5-2 for further guidance.

5.4.2.3 Maximum Bridge Length

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.

Four steel piles (listed in Table 5-3 and Table 5-4) most commonly used by MaineDOT were evaluated and maximum bridge length and maximum design pile load design guides were developed based upon the Greimann research. The piles 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 a 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, it was 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. The maximum bridge lengths are 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 current limit for maximum bridge length is 200 feet for steel and 330 feet for concrete, which cannot be exceeded without the approval of the Engineer of Design. 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).

5.4.2.4 Pile Capacity and Fixity

Pile structural capacity is governed by the axial and biaxial bending column action of the pile. Axial stresses result from vertical superstructure live and dead loads, abutment and pile dead load, and secondary thermal force (for multi-span structures only, refer to Figure 5-4).

The PΔ effect of the vertical pile load is the only moment considered. Thermal translation moments and live load girder rotation moments are assumed to be redistributed through inelastic rotation.

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. Minimum pile lengths are provided in Table 5-5; however soil conditions and loading conditions may require additional pile embedment to achieve fixity. Also, axial loads may govern and additional embedment length may be required in order to achieve the design axial load and a factor of safety of 4.0. If more accurate site-specific soil properties and loading conditions exist, an evaluation of minimum embedment length can be performed using the MassHighway method (MassHighway Bridge Manual, 1999) or the depth to fixity can be determined using the computer programs COM624P and Lpile, or the Davisson and Robinson equation in AASHTO LRFD Section 10.7.4.2. 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 Piles for additional design requirements.

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

5.4.2.5 Bridge Length for Pile Supported Abutments

Table 5-3 Maximum Bridge Length for Fixed Head Abutment (feet)

	$0^{o} \leq$ Skew < 20^{o}		$20^{o} \leq$ Skew $\leq 25^{o}$	
Pile Size	**Steel**	**Concrete**	**Steel**	**Concrete**
HP 10 X 42	200	330	140	230
HP 12 X 53	130	215	75	125
HP 14 X 73	120	200	70	115
HP 14 X 89	200	330	200	330

Table 5-4 Maximum Bridge Length for Pinned Head Abutment (feet)

	$0^{o} \leq$ Skew < 20^{o}		$20^{o} \leq$ Skew $\leq 25^{o}$	
Pile Size	**Steel**	**Concrete**	**Steel**	**Concrete**
HP 10 X 42	200	330	200	330
HP 12 X 53	200	330	200	330
HP 14 X 73	200	330	200	330
HP 14 X 89	200	330	200	330

The above bridge length criteria is 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 $\leq$12 feet and pile spacing is $\leq$ 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).

- Allowable stress design for piles is per the AASHTO Standard Specifications

- Yield strength, F_y, of the pile equal to 36 ksi.

Bridge lengths in excess of the above limitations may be used with the approval of the Engineer of Design when special design features are provided. However, in no case should steel bridge lengths exceed 300 feet or concrete bridge lengths exceed 500 feet.

5.4.2.6 Abutment Details

Typical 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.

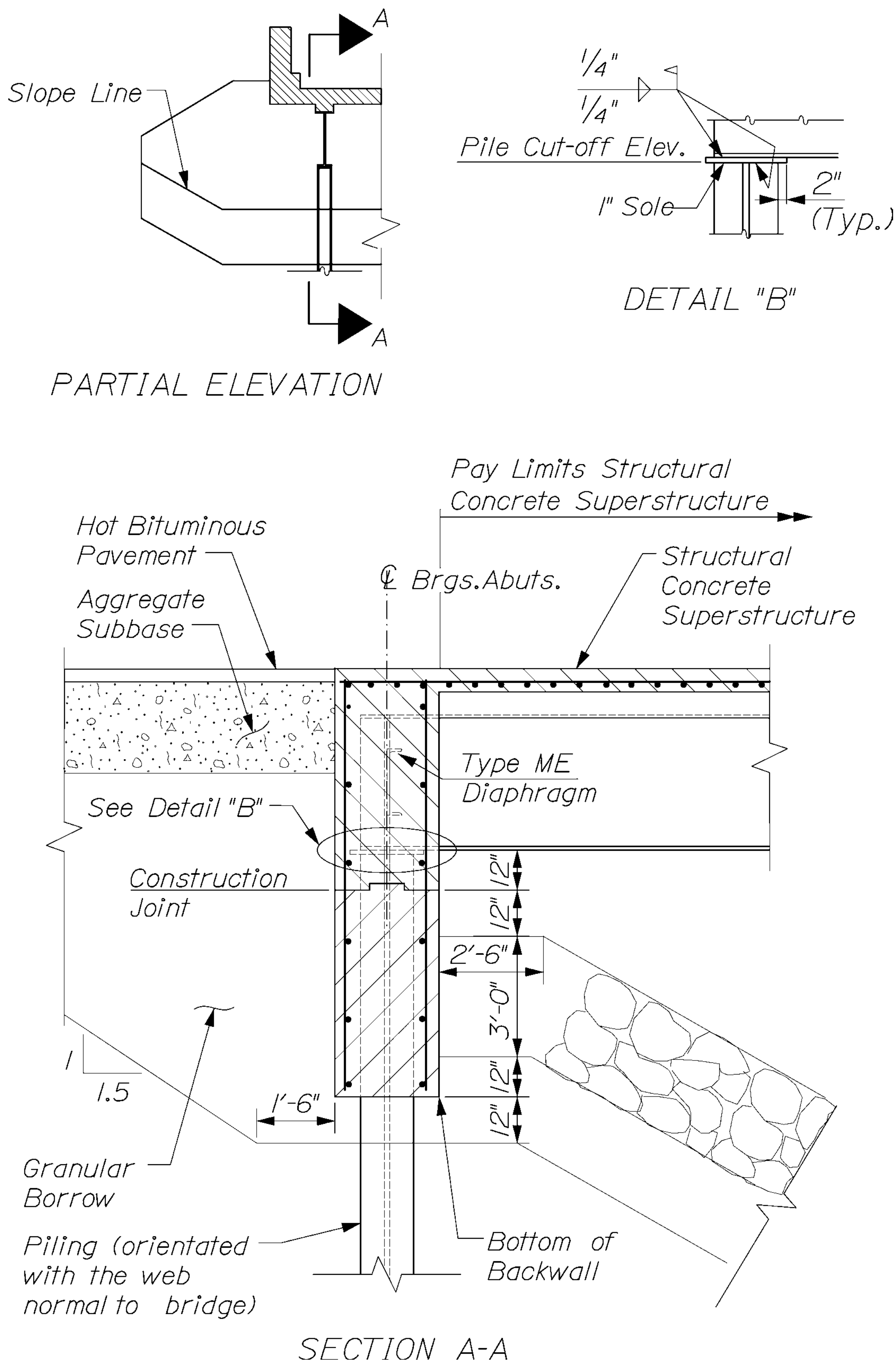


Figure 5-2 Fixed Head Integral Abutment Details-Steel Superstructures

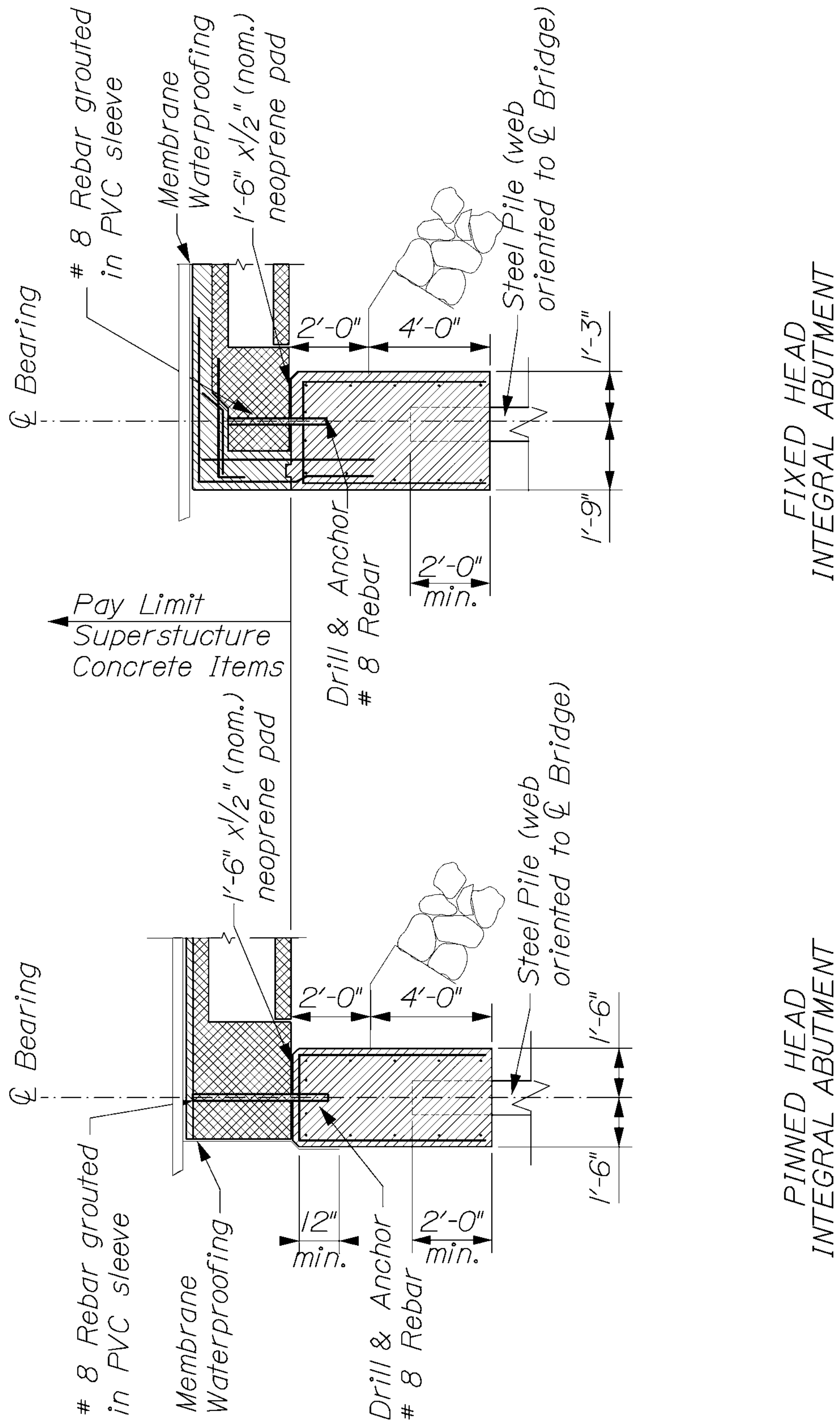


Figure 5-3 Integral Abutment Details – Precast Superstructures

5.4.2.7 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%.

5.4.2.8 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.

5.4.2.9 Abutment and Wingwall Design

Wingwalls should preferably be straight extension wings not to exceed 10 feet in length. Abutment and wingwall reinforcement should be sized assuming passive earth pressure (K_p) on the back face of the wall. Refer to Section 3.6.6 Coulomb Passive Lateral Earth Pressure Coefficient. Consult the Geotechnical Designer or Report for further guidance.

5.4.2.10 Approach Slabs

In addition to the requirements of Section 5.4.4, approach slabs should be used when 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. For approach slabs below grade, consideration should be given to attaching the approach slab 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.

5.4.2.11 Drainage

The area behind integral abutments should be backfilled with granular borrow for underwater backfill. A proper drainage system as described in Section 5.4.1.4 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.

Procedure 5-4 describes the process for designing pile supported integral abutments. Refer to Section 5.4.2.13 for a design example.

Procedure 5-4 Integral Abutment Design Process

Step 1. Calculate Maximum Vertical Pile Loads

Dead Load Superstructure Reaction
Unfactored Live Load Plus Impact Superstructure Reaction
Abutment Dead Load
Pile Dead Load
Secondary Thermal Effects, multi-span bridges only (Figure 5-7 or Figure 5-8)

Step 2. Select Pile Size as a Column

Select pile size to meet the allowable load from Figure 5-5 or Figure 5-6.

Step 3. Piles must be capable of transferring loads to the ground by either end bearing or friction. End bearing piles should be checked for the provisions of Section 5.7.1 H-Piles, with safety factor of 4.0.

Step 4. Check Live Load Rotation Demand. This pile stresses resulting from the superstructure live loads should not exceed 0.55 F_y.

a. Beam or Girder end rotation:

The moment at the pile head can be calculated from the following approximate stringer end rotation:

$$R_g = \frac{W \cdot L_S^2}{24 \cdot E_g \cdot I_g}$$

Where:
R_g = Beam/girder rotation (radians)
W = Total beam/girder live load, end span
L_S = Length of end span
E_g = Modulus of elasticity of end span beam/girder
I_g = Moment of inertia of end span stringer (composite I for composite beams)

b. Rotation induced moment

$$M = \frac{4 \cdot E_p \cdot I_p \cdot R}{L_e}$$

Where:
M = Pile head moment
E_p = Modulus of elasticity of pile
I_p = Moment of inertia of pile
R = Beam/girder rotation (radians)
L_e = Effective pile length (use minimum embedment length in Table 5-5, or refer to Section 5.4.2.4 Pile Capacity)

c. Pile Stress

$$\sigma_p = \frac{M}{S_p}$$

Where:
σ_p = Pile stress
S_p = Section modulus of the pile
M = Pile head moment

Step 5. End bearing piles should have the following minimum embedment lengths:

Table 5-5 Minimum Embedment Lengths

Pile	Minimum Embedment Length (ft)
HP 10 X 42	10
HP 12 X 53	12
HP 14 X 73	13
HP 14 X 89	15

Additional embedment length may be required for the use of friction piles and end bearing piles.

5.4.2.12 Integral Abutment on Spread Footing Design

Spread footing abutments may be used within the following limitations:

- Steel structure length: $\leq$ 80 feet
- Concrete structure length: $\leq$ 140 feet
- Abutment heights: $\leq$ 8 feet
- Skews: $\leq$ 25°

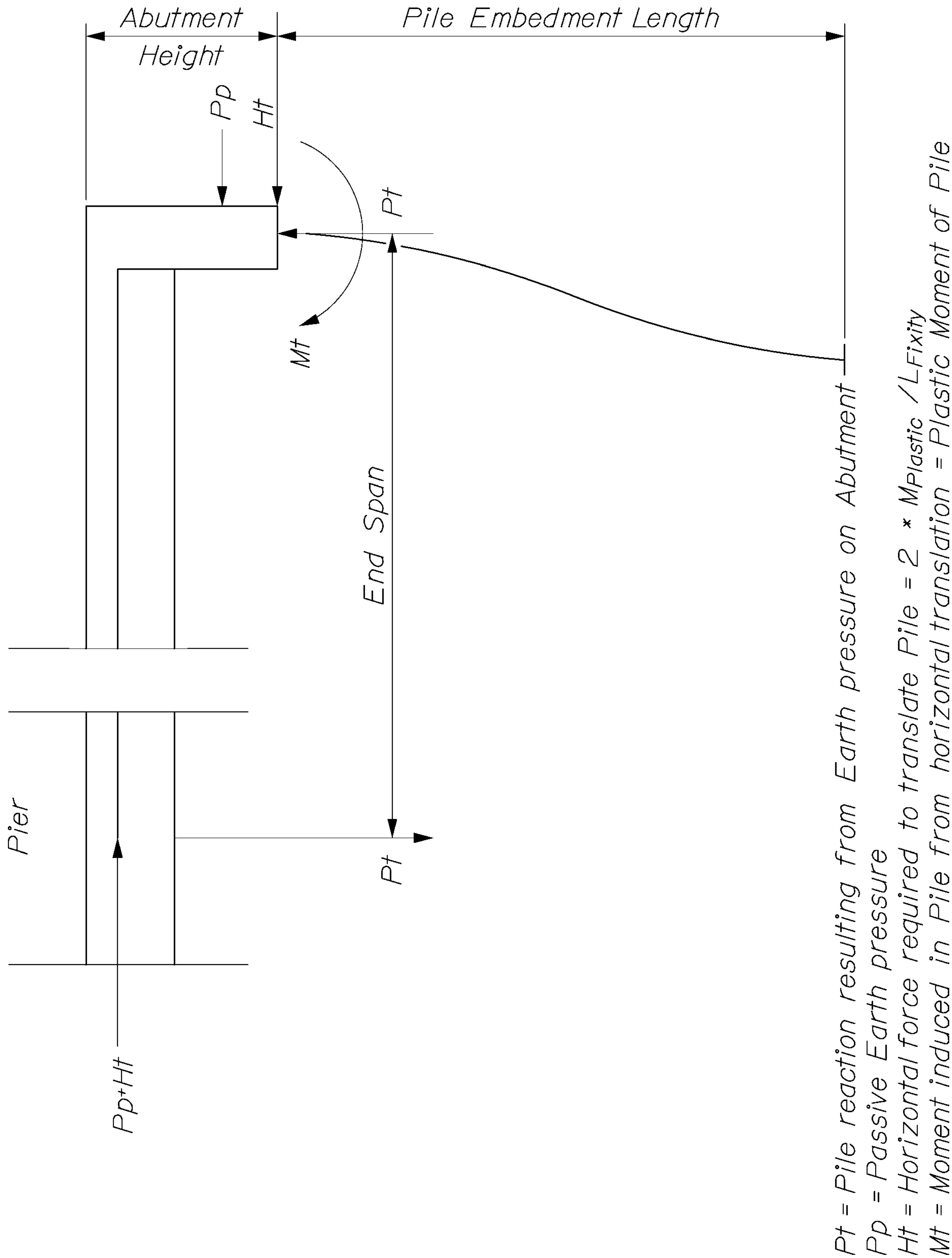


Figure 5-4 Thermally Induced Secondary Pile Forces

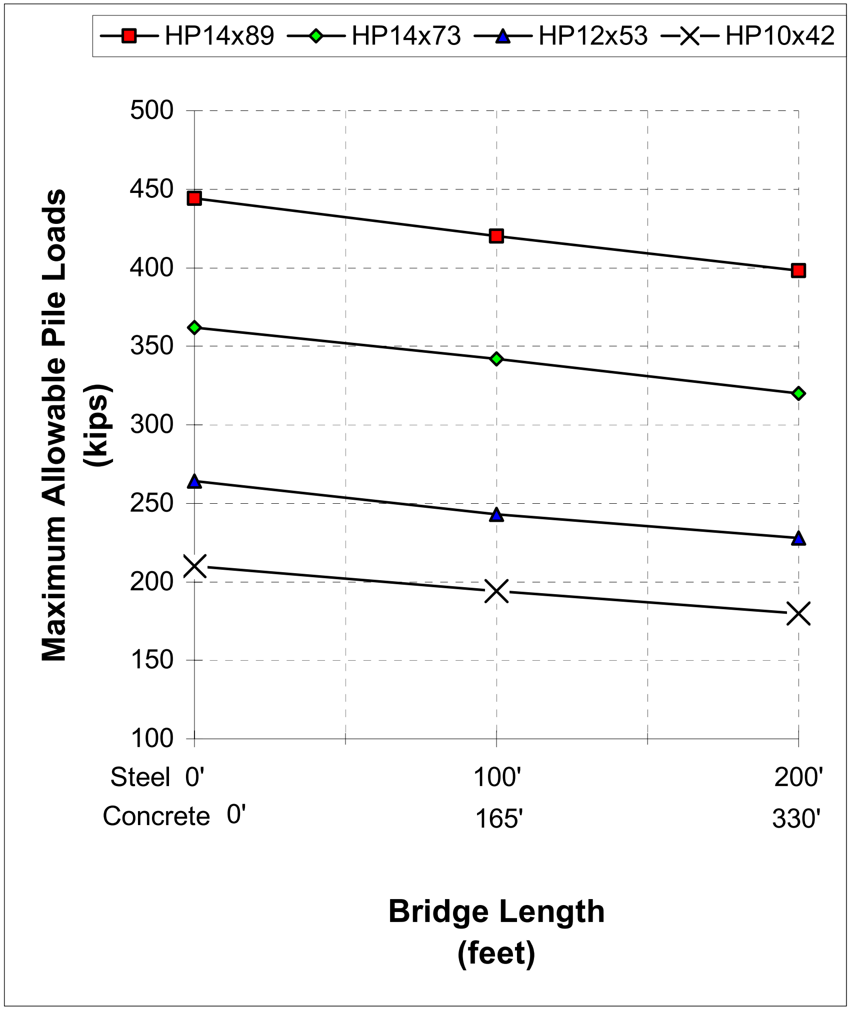


Figure 5-5 Maximum Allowable Pile Load Fixed Head Piles

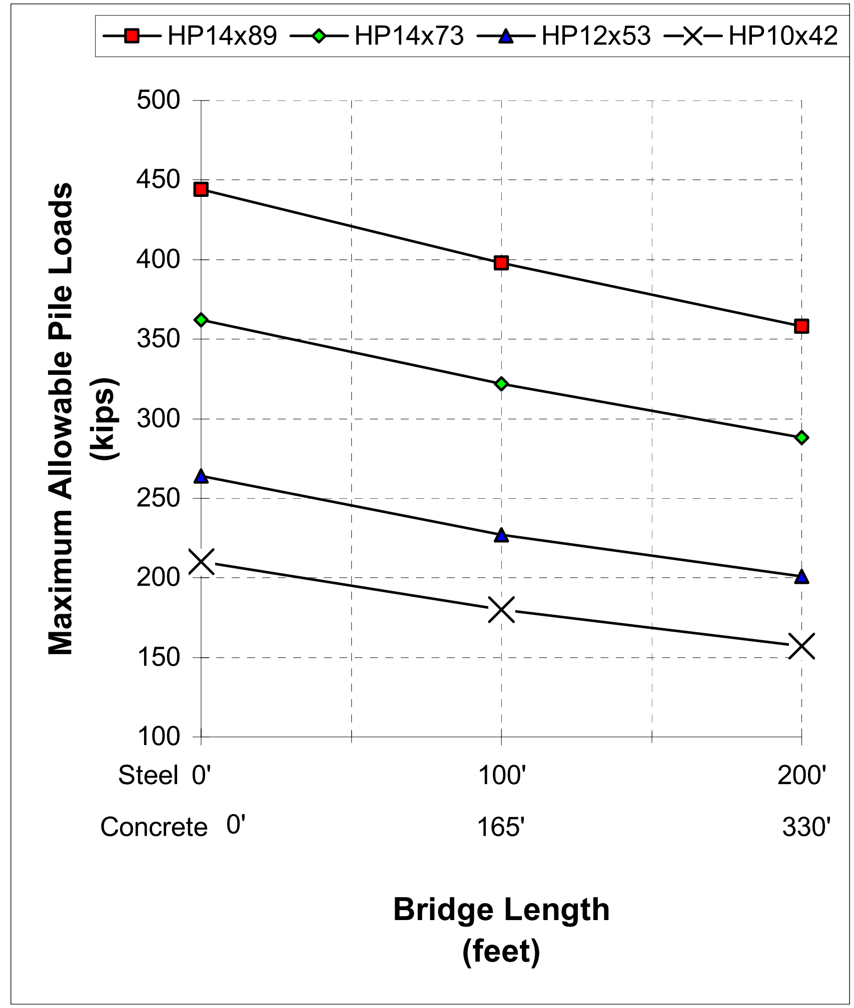


Figure 5-6 Maximum Allowable Pile Load Pinned Head Piles

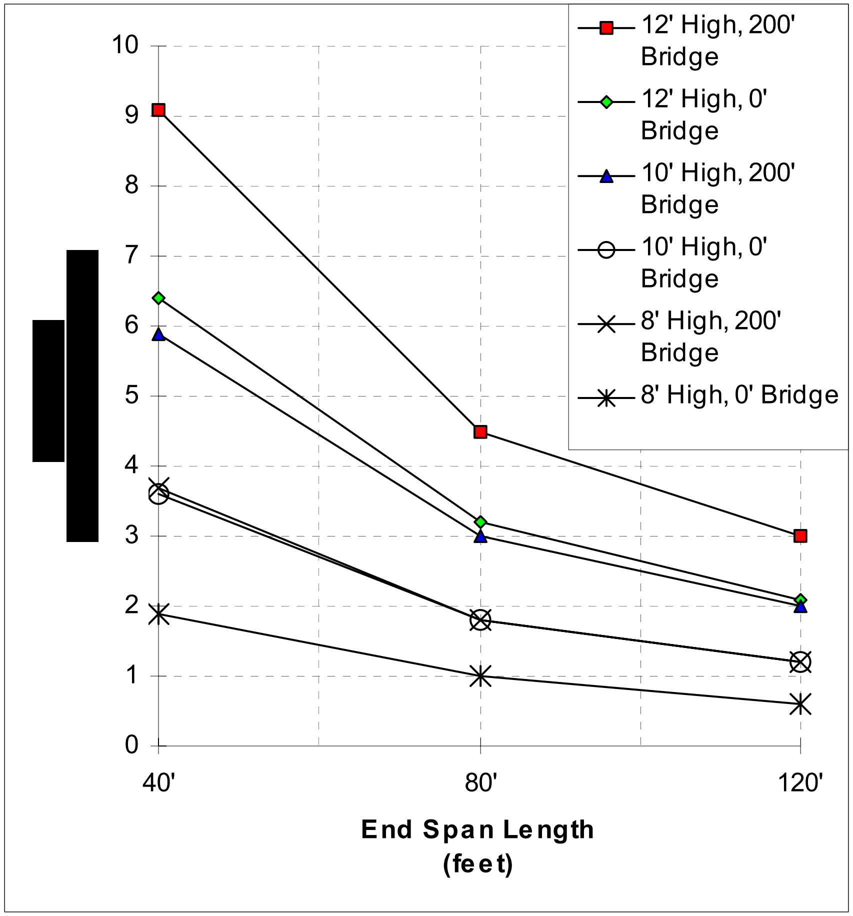


Figure 5-7 Fixed Head Pile Thermally Induced Secondary Pile Forces

for Multi-Span Bridges

(Kips per Longitudinal Foot of Backwall)

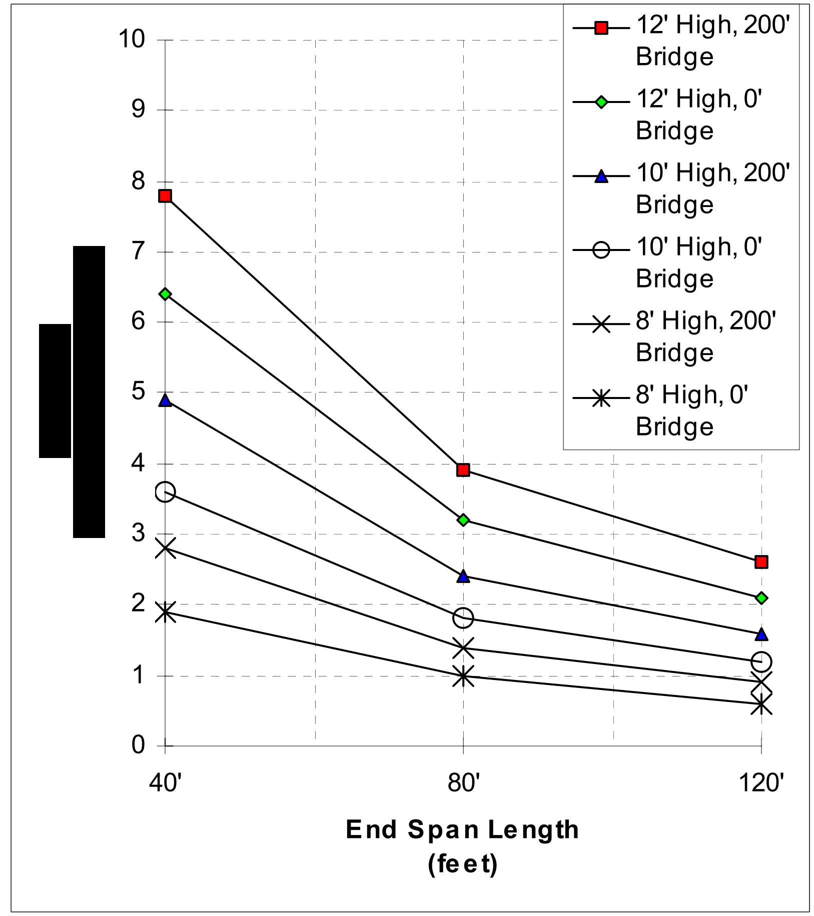


Figure 5-8 Pinned Head Pile Thermally Induced Secondary Pile Forces

for Multi-Span Bridges

(Kips per Longitudinal Foot of Backwall)

5.4.2.13 Example Problem

The following problem provides an example integral abutment design.

Example 5-2 Pile Supported Integral Abutment

Given:
Two span continuous steel beam bridge with fixed head integral abutment
All girders parallel
Vertical grade on bridges: 1% grade
Spans: 85 feet - 85 feet
Skew: 10°
Girder and pile spacing: 7.0 feet
Girder I: 29,750 in^4
Abutment Height: 12 feet
Pile Length: 15 feet
Abutment wall thickness: 2.5 feet
Dead Load Reaction: 45.7 k
Live Load + Impact Reaction: 31.3 k

Step 1. Select pile based upon bridge length.

Total bridge length = 170 feet

From Table 5-3 for fixed head piles, 0° to 20° Skew
HP 10 x 42 and HP 14 x 89 piles are acceptable

Step 2. Check alignment.

Beams are parallel to each other, all substructure units are parallel and grade between abutments does not exceed 5%.

Step 3. Check superstructure design.

Span ratios are equal, providing equal movement at each abutment.

Step 4. Wingwalls

Make wings straight extensions less than 10 feet.

Step 5. Approach slabs

Bridge length exceeds 80 feet: Approach slabs are required.

Bridge length exceeds 140 feet: Provisions for movement between the approach slabs and approach pavement are required.

Step 6. Pile design: Calculate maximum vertical pile loads.

a. Dead load superstructure reaction	=	45.7 k
b. LivelLoad superstructure reaction	=	31.3 k

c. Abutment dead load
12 feet x 7.0 feet x 0.15 k/ft^3 x 2.5 feet = 31.5 k
d. Pile dead load = 0.089 k/ft x 15 feet = 13.4 k
e. Secondary thermal effects = 4.0 k/feet of
abutment from Figure 5-6 = 4.0 k/feet x 7.0 feet = <u>28.0 k</u>
TOTAL 149.9 k

Step 7. Pile design: Check pile capacity as a column.

From Figure 5-5:

HP 10 x 42 = 185 k allowable > 138.3 k <u>OK</u>
HP 14 x 89 = 405 k allowable > 138.3 k <u>OK</u>

Both piles are acceptable.

Step 8. Pile design: Piles must be capable of transferring loads to the ground.

Pile capacity for 12,500 psi (FS = 4), refer to also Table 5-6.

Capacity HP10 x 42 = 155 k (from Table 5-6) > 149.9 k <u>OK</u>

Step 9. Pile design: Check live load rotation demand.

The pile stress from girder live load rotation $\leq$ 0.55 Fy

0.55 Fy = 0.55 (50 ksi) = 27.5 ksi

a. Beam end rotation:

$$R = \frac{W \cdot L_S^2}{24 \cdot E_S \cdot I_S}$$

W = 31.3 k x 2 = 62.6 k
L_S = 85 feet x 12 in/ft = 1020 inches
E_S = 29,000 ksi
I_S = 29,750 in^4

$$R(radians) = \frac{62.6 \cdot kips \cdot (1020 \cdot in)^2}{24 \cdot 29000 \cdot ksi \cdot 29750 \cdot in^4} = 0.0032 \cdot rad$$

b. Rotation induced moment for HP 10 x 42:

$$M = \frac{4 \cdot E_p \cdot I_p \cdot R}{L_e}$$

E_p = 29,000 ksi
I_p = 71.7 in^4
R = 0.0032 radians

L_e = 10 feet (from Table 5-5) x 12 in/ft = 120 inches

$$M = \frac{4 \cdot 29000 \cdot ksi \cdot 71.7 \cdot in^4 \cdot 0.0032 \cdot rad}{120 \cdot in} = 222 \cdot in \cdot k$$

c. Pile stress:

$$\sigma = \frac{M}{S}$$

$$\sigma = \frac{222 \cdot in \cdot k}{14.2 \cdot in^3} = 6 \cdot ksi \leq 27.5 \cdot ksi$$ Therefore, OK.

USE HP 10 x 42

Step 10. Pile design: Provide minimum embedment length in accordance with Table 5-5 and check with the Geotechnical Designer for the allowable geotechnical capacity of the pile and the appropriate depth to fixity (L_e).

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). In these bridges, the abutment foundations behave conventionally, while the backwall (end diaphragm) moves along a horizontal joint below ground. This serves to eliminate the roadway joint, and therefore should reduce maintenance requirements. 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.

Commentary: An example of a semi-integral bridge is the Gouldsville Bridge in Presque Isle, constructed in 2002. This structure incorporates elastomeric bearings, a small airspace to prohibit bearing at the end diaphragm, and buried joints of compressible joint filler between the end diaphragms and the approach slabs. However, the bearings were modified to allow expansion to occur conventionally, using both fixed and expansion bearings. As the behaviors of integral and semi-integral bridges are evaluated further, future designs should provide improvements and more consistency in design.

In general, semi-integral bridges resist excessive translation in the longitudinal directions via full-depth end diaphragms and passive earth pressure. Western states, such as Washington, have taken the lead in designing, building, and evaluating this structural type. Maximum structure lengths in this research are relatively long, usually over 200 feet for steel, which produces large thermal expansion movements certain to generate more passive soil pressure than many of the relatively short integral bridges built in Maine.

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

- o Utilization of attached approach slabs and return wingwalls to lock the superstructure into the backfill
- o 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 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 5.4.2.10. 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.

Approach slabs should be used on collectors and arterials, where the design hour volume (DHV) is greater than 200, or where abutment heights (bottom of footing to finish grade) are greater than 20 feet, or where poor soil conditions are encountered and settlement is anticipated in the vicinity of the abutment.

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 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:

- o Loading conditions
- o Skew
- o 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 Loads

Mass piers should be designed in accordance with both the AASHTO Standard Specifications and the AASHTO LRFD Specifications. Structural analysis and design of reinforced concrete should be completed using LRFD. Geotechnical analysis and design, such as bearing capacity, sliding, and overturning should be completed using ASD. Loads should be determined in accordance with the AASHTO LRFD Specifications, and as outlined in Chapter 3. The following service load design groups should be considered as a minimum for geotechnical analysis:

- *Group I:* dead load; live load plus impact; centrifugal force; earth pressure, if applicable; buoyancy; and stream flow pressure.

- *Group III:* dead load; live load plus impact; centrifugal force; earth pressure, if applicable; buoyancy; stream flow pressure; wind; wind on live load; and longitudinal force from live load.

- *Group VIII:* dead load; live load plus impact; centrifugal force; earth pressure, if applicable; buoyancy; stream flow pressure; and ice pressure.

- *Group IX:* dead load; earth pressure, if applicable; buoyancy; stream flow pressure; wind; and ice pressure.

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 AASHTO Standard Specifications Section 3.18.1.3.

5.5.1.3 Structural Design

Piers should be designed in accordance with AASHTO LRFD Section 5.7 to 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 determined in accordance with AASHTO LRFD Section 5.13.3.6.

5.5.1.4 Geotechnical Design

A. Overall Stability

The effect of forces tending to overturn mass piers should be considered, as specified in AASHTO Standard Specifications Article 3.15.3.

B. Spread Footing

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

C. Deep Foundation

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 Standard Specifications, 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.

D. 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 maximum calculated scour depth, or 2) designing the deep foundation elements for an unsupported length. The exposed pile length should be the vertical distance from the bottom of the seal to the calculated scour depth.

5.5.1.5 Pier Protection

A. Collision

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.

B. Collision Walls

When piers or abutments are located within 25 feet of the centerline of the railroad tracks collision walls extending 6 feet above top of rail are required between columns for railroad overpasses, and similar walls extended 2.35 feet above ground should be considered for grade separation structures, unless other protection is provided.

C. Scour

Refer to Section 2.3.11 Scour for guidance.

D. 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
- Where the Q50 velocity is greater than 5 ft/s
- 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

5.5.2.2 Loads

Pile bent piers should be designed in accordance with either the AASHTO Standard Specifications or AASHTO LRFD, depending upon the component being analyzed. Structural analysis and design of reinforced concrete should be completed using LRFD. Geotechnical analysis and design, such as global stability and pile design, should be completed using ASD, except that the unfactored live load is calculated using LRFD loading. Loads should be determined in accordance with AASHTO LRFD, and as outlined in Chapter 3 Loads. The following service load design groups should be considered as a minimum:

- *Group I:* dead load; live load plus impact; centrifugal force; earth pressure, if applicable; buoyancy; and stream flow pressure

- *Group III:* dead load; live load plus impact; centrifugal force; earth pressure, if applicable; buoyancy; stream flow pressure; wind; wind on live load; and longitudinal force from live load

- *Group VIII:* dead load; live load plus impact; centrifugal force; earth pressure, if applicable; buoyancy; stream flow pressure; and ice pressure

- *Group IX*: dead load; earth pressure, if applicable; buoyancy; stream flow pressure; wind; and ice pressure

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 AASHTO Standard Specifications Section 3.18.1.3.

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 only to 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

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.

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.

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.

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:

- Pile cap shrinkage and temperature actions are applied to the longitudinal axis of the pier.
- 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.

5.5.2.3 Pile Type Selection Criteria

Concrete filled pipe piles, precast concrete piles, and combination H-piles encased with pipe piles filled with concrete 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: the AASHTO Standard Specifications and AASHTO LRFD are 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.)

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

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

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

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.

5.5.2.4 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 coated with fusion-bonded epoxy paint.

In corrosive environments, cathodic protection should be used and applied on the downstream side of the piles within 5 feet of the streambed.

B. Pipe Piles

A fusion-bonded epoxy protective coating should be applied to a minimum of 10 feet below streambed or 2 feet below the total scour depth.

Cathodic protection should be used in addition to the fusion-bonded epoxy coating in corrosive environments such as salt water.

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.

5.5.2.5 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 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 <u>AASHTO LRFD</u> Section 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 5-9 and Figure 5-10.

The depth to fixity shown in Figure 5-9 and Figure 5-10 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. Refer to Davisson and Robinson procedure (National Corporate Highway Research Program 1991).

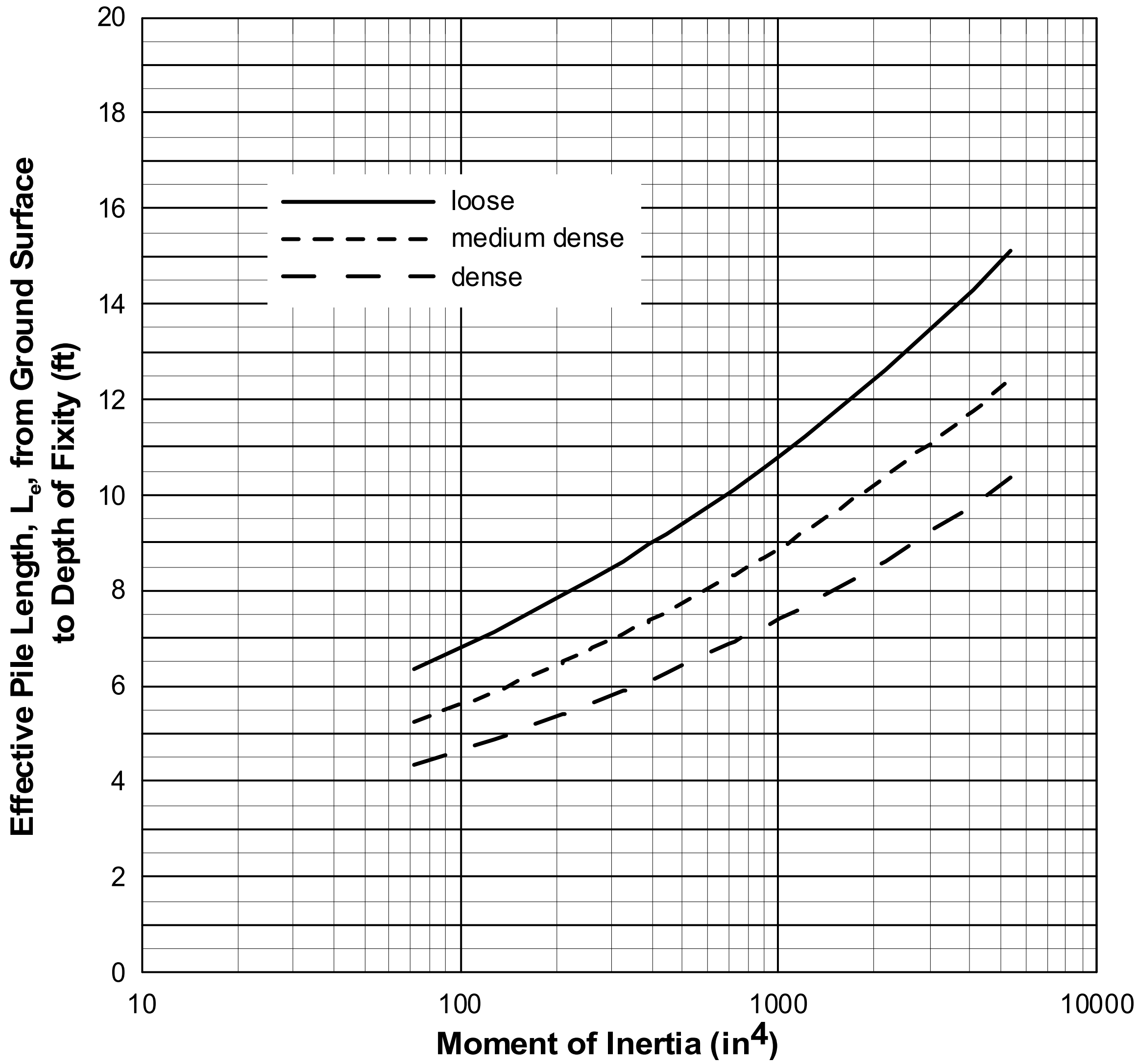


Figure 5-9 Effective Pile Length for Piles in Sand

From Ground Surface to Depth of Fixity

Axially Loaded

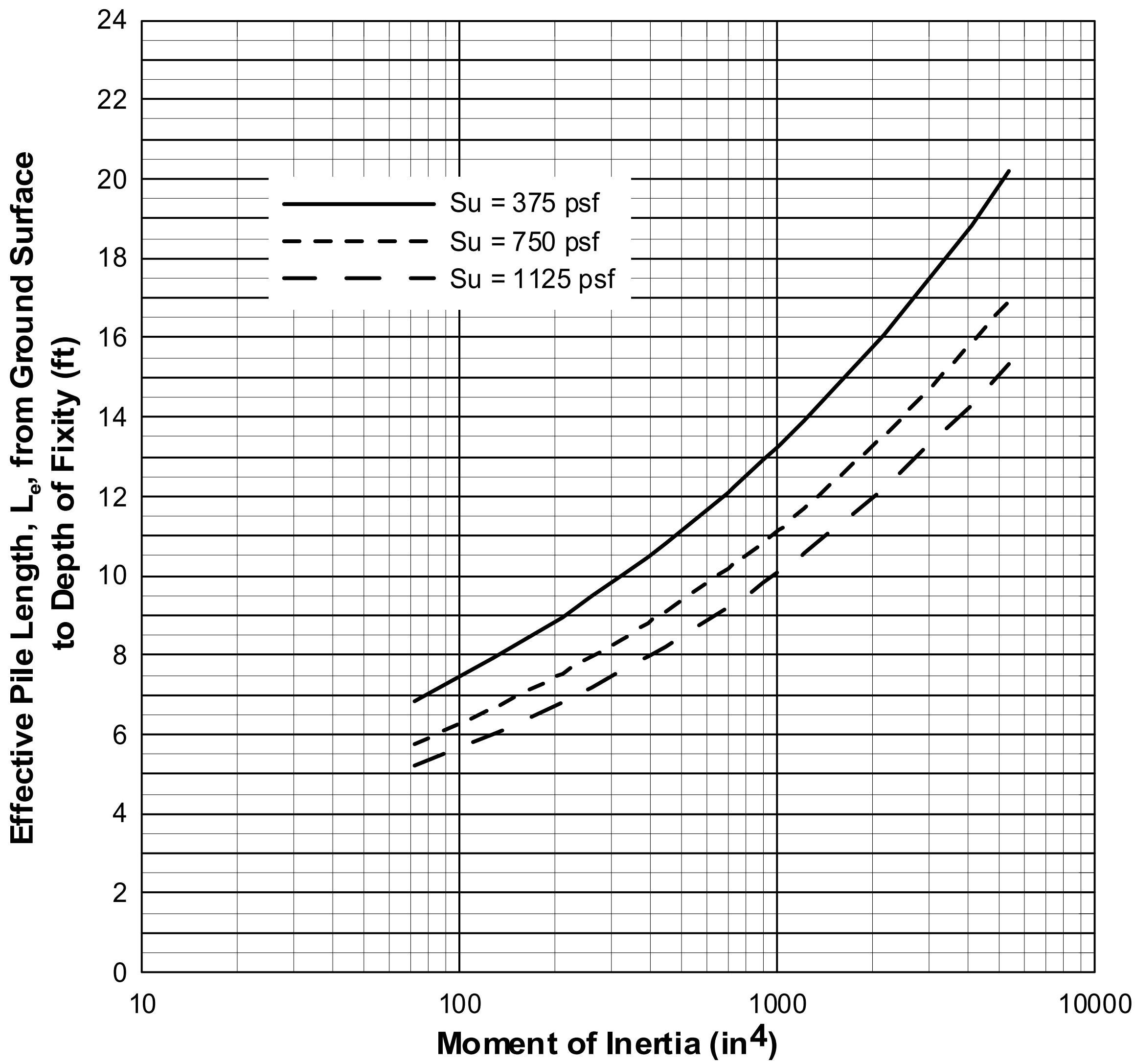


Figure 5-10 Effective Pile Length for Piles in Clay

From Ground Surface to Depth of Fixity

Axially Loaded

B. 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 AASHTO LRFD Section 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.

C. 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 1/8” of sacrificial shell corrosion. The pipe pile shell must have a minimum thickness of 1/2" to allow for proper driving of the pile and to resist corrosion.

5.6 Retaining Walls

5.6.1 General

Retaining walls typically used by the Bridge Program are gravity walls, cantilever 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 have soil reinforcements drilled into the in-situ soil/bedrock and, and therefore 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, 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 either the AASTHO Standard Specifications, or AASHTO LRFD, depending upon the component being analyzed. Structural analyses and design of reinforced concrete for retaining walls will be computed using LRFD. For geotechnical evaluation and design of retaining walls, such as overturning, sliding, bearing pressure, and global stability, use the ASD, except that unfactored live load is calculated using LRFD loading. Loading combinations for the ASD methods are presented in AASHTO Standard Specifications Table 3.22.1A. Loads should be determined in accordance with AASHTO LRFD and as outlined in Chapter 3. The following load conditions should be considered when applicable:

- Lateral earth pressure, including any live and dead load surcharge
- Self weight of the wall
- Lateral loads due to live load impact on the parapets

- Soil surcharge, due to live load
- Railroad loading
- Hydrostatic pressure

Walls that can tolerate little or no movement should be designed for at-rest (K_o) earth pressure.

5.6.1.4 Design Considerations

All retaining walls should be designed with consideration of frost protection (Section 5.2.1), scour protection (Section 2.3.11), bearing capacity (Section 5.3.2), settlement (Section 5.3.3), stability (Section 5.3.4), drainage considerations (Section 5.3.6), and seismic considerations (Section 5.3.7), as appropriate. All retaining walls require a geotechnical investigation of the underlying soil/bedrock that will support the structure.

5.6.1.5 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 derive their capacity to resist lateral soil loads through a combination of dead 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. Since deflections of gravity walls are consistent with deflections of gravity abutments, only extra #6 bars at 8 feet long placed at the corners are required.

5.6.2.2 Earth Loads

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 Retaining Walls

This section discusses gravity cantilever retaining walls. This type of wall is differentiated from a non-gravity cantilever retaining wall by the presence of a footing. 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 overturning 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. 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 5-11 for further guidance.

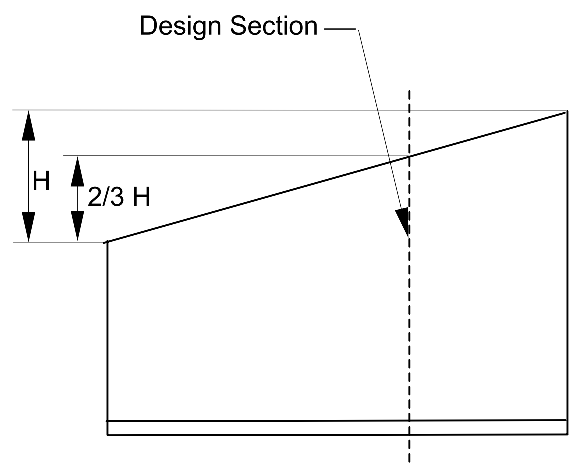


Figure 5-11 Retaining Wall Design Section

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.

5.6.4 Non-Gravity Cantilever Retaining Walls

This section discusses non-gravity cantilever retaining walls. Non-gravity cantilever retaining walls derive lateral resistance through embedment of vertical wall elements. These vertical elements may consist of piles (soldier or sheet), 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. Gravity cantilever retaining walls (i.e., cantilever walls with a footing) are discussed in Section 5.6.3.

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 Articles 5.8 and 5.9 of the AASHTO Standard Specifications and Chapter 3, Loads. The Geotechnical Designer is required to verify acceptable factors of safety for global stability of the wall prior to construction. The allowable bearing pressure of the wall 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 nonapproved proprietary wall systems. Preapproved wall systems have been extensively reviewed by MaineDOT and are listed in the Special Provision for the particular wall type. MaineDOT has developed a policy for the review of nonapproved proprietary walls systems (MaineDOT, January 2, 2003), available from the Engineer of Design or the Transportation Research Division. Nonapproved proprietary walls must go through the process outlined in this policy prior to use of the wall system.

5.6.5.2 Prefabricated Bin Type Retaining Walls

A. Prefabricated Concrete Modular Gravity Wall

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 Section 3.6.7.2 Prefabricated Modular Walls.

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 5.3.6 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.

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.

B. Metal Bin Walls

Metal bin walls are a gravity-type retaining wall with corrugated steel sides, built into a box shape and filled with compacted granular soil. The bins form a system of adjoining close-faced bins, each about 10 feet long. Galvanizing or a fiberglass or carbon graphite fiber coating protects the metal. To improve the service life of metal bin walls, consideration should be given towards increasing the galvanizing requirements and establishing electrochemical requirements for the confined backfill. The base width of bin walls is typically limited to 60% of the wall height. They are flexible and adjust to minor ground movement without significant distortion. Observed corrosion of some galvanized metal bin walls indicate a service life shorter than 75 years, and preference should be given to the use of a PCMG wall system.

5.6.5.3 Modular Block Walls

Modular block walls consist of walls where modular blocks, stacked vertically, function as a gravity retaining wall, as covered in Special Provision 611. The connection between adjacent courses of modular blocks may be mechanical (pins) or frictional (tongue-and-groove configuration). These wall systems are generally limited to a maximum height of 4 feet when no surcharge load is applied. When wall height is in excess of 4 feet or a surcharge is applied, geosynthetic reinforcement may be added to the modular blocks to create a geosynthetic-reinforced soil (GRS) wall. This particular application is discussed in Section 5.6.5.4B Geosynthetic-Reinforced Soil Walls.

Blocks for modular block walls are dry-cast, and they are susceptible to degradation caused by freeze-thaw. At the time of publication of this guide, suppliers have not been able to meet MaineDOT's freeze-thaw requirements specified in Special Provision 611. Modular block wall use should be restricted to areas where exposure to road salts is limited, due to this degradation. Modular block 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.

MSE walls are designed by the wall manufacturer for internal and external stability. All MSE walls should be designed in accordance with AASHTO Standard Specifications Article 5.8 as required in Standard Specification Section 636 – Mechanically Stabilized Earth Retaining Wall and Chapter 3 Loads. It is the responsibility of the Geotechnical Designer to assess the wall for bearing capacity, settlement, and global slope stability.

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 5.6.1.5 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 substructure units. The substructure units may be either spread footings or pile supported.

Prior to selection of MSE walls for a project, consideration should be give 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 walls with geosynthetic (polymeric) soil reinforcement. GRS walls are designed to create a reinforced soil block behind a wall facing. Facing options include precast 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.

Blocks for GRS walls are dry-cast and they are susceptible to degradation caused by freeze-thaw. At the time of publication of this guide, suppliers have not been able to meet MaineDOT's freeze-thaw requirements. GRS wall use should be restricted to areas where exposure to road salts is limited. 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. Therefore, in a cut situation, GRS wall structures are uneconomical and difficult to construct. 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 AASHTO Standard Specifications Article 5.8 will be allowed.

GRS walls are proprietary and are designed by a wall manufacturer for internal and external stability in accordance with AASHTO Standard Specifications Article 5.8 and Chapter 3 Loads. 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.

C. Soil Nail Walls

Soil nail walls are technically MSE walls in that they 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.

5.6.5.5 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
- Allowable bearing capacity
- Foundation embedment criteria
- General details for any desired apprentices, such as coping or drainage requirements
- Project specific loads for other design acceptance requirements (example: seismic loads)
- 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 Anchored Walls

5.6.6.1 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 the most recent version of the AASHTO Standard Specifications. 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).

5.6.6.2 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 the AASHTO Standard Specifications. The design requirements for the metal structural plate headwall/wingwall system should be included with the contract documents.

5.6.7 Gabions

Gabion walls consist of stacked 3 feet cubed wire baskets, which are filled with stone. Groups of filled gabion baskets are staked 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 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.

5.7.1.1 Axial Capacity

The axial design load applied to H-pile sections should not exceed the lesser of the allowable structural capacity and the allowable geotechnical capacity. The allowable structural capacity should be determined using a factor of safety of 3.0 or 4.0, defined as follows:

SF = 3.0: For axial loads on long piles (30 feet or greater) when driving to bedrock, where pile damage is unlikely, and the ultimate capacity is verified using dynamic load testing.

SF = 4.0: For other end bearing piles, for all friction piles, for all piles less than 30 feet long, for all integral abutment piles, and all other piles that may have unusual induced moments and that are not specifically designed for bending.

The allowable structural axial capacity of selected H-Pile sections is presented in Table 5-6.

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

Table 5-6 Allowable Structural Axial Capacity of Selected H-Pile Sections

F_y = 50 ksi.

Pile Section	Allowable Structural Axial Capacity	
	SF = 4 12.5 ksi (kips)	SF = 3 16.7 ksi (kips)
HP 10x42+	155	207
HP 10x57	210	280
HP 12x53+	194	258
HP 12x63	230	307
HP 12x74	273	363
HP 13x60	219	292
HP 13x73	270	360
HP 13x87	319	425
HP 14x73+	268	357
HP 14x89+	326	435
HP 14x102	375	500
HP 14x117	430	573

Note: Those marked + are preferred sections

The geotechnical capacity should be determined for site-specific conditions by the Geotechnical Designer. Consideration should be given to downdrag, soil relaxation, soil setup, and any other site-specific factors, which may affect the pile capacity during and after construction. The allowable geotechnical capacity should be determined by applying a factor of safety, which is dependent on the design method and the magnitude of quality assurance/control provided during construction operations. The factors of safety for H-pile geotechnical axial capacity are presented in Table 5-7. These factors of safety are based upon construction quality control beyond the standard subsurface exploration and static capacity evaluation or analysis.

Table 5-7 Factors of Safety for Allowable Geotechnical Axial H-Pile Capacity

Construction Control Method	Factor of Safety
Static load test with wave equation analysis	2.00
Dynamic testing with wave equation analysis	2.25
Indicator piles with wave equation analysis	2.50
Wave equation analysis	2.75

5.7.1.2 Lateral Capacity

The lateral capacity of a pile is governed by the loading condition, pile stiffness, stiffness of the soil, and the degree of fixity. The lateral capacity (P_L) and depth to fixity (D_f), for selected H-Pile sections in sand and clay are presented in Table 5-8 and Table 5-9, respectively.

Commentary: The lateral capacity and depth to fixity presented in Table 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 that presented in Table 5-6 (SF = 4), and a deflection of 1/8".

Table 5-8 Lateral Capacity and Depth to Fixity for H-Pile Sections in Sand

Load Perpendicular to Flange

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 exceeds that presented in Table 5-8 and Table 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.

Table 5-8 and Table 5-9 present the lateral capacity 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 (Table 5-8 and Table 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 Lateral Capacity and Depth to Fixity for H-Pile Sections in Clay

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

5.7.1.3 Layout and Construction

The pile spacing should not be larger than is reasonable or practical. The minimum spacing of piles is 2.5 to 3 times the diameter of the pile. 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.

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

5.7.2 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 AASHTO Standard Specifications Article 4.5.16 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.

5.7.3 Pipe Piles

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

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 2 or 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.

5.7.4 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 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 loads when the ultimate 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 added to the vertical dead load applied to the pile. A factor of safety of 1.0 against downdrag forces is required.

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

- o Increasing the pile size
- o Sleeve piles

5.7.5 Pile Testing Programs

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 monitoring should be considered in order to:

- o Verify the pile geotechnical capacity
- o Monitor piles installed in difficult subsurface conditions, such as soils with obstructions and boulders, or a steeply sloping bedrock surface
- o Verify consistent hammer operation during extended pile installation operations
- o Lower factors of safety.

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 should apply:

- o Prior to production piles being installed, dynamic load tests will be conducted at selected representative foundation locations for the purposes of verifying design.
- o Post-driving analyses (CAPWAP) are required.
- o Provisions for restriking piles should be included, for the case that setup or relaxation effects are significant.
- o Provisions should be provided for the conduct of additional dynamic load tests during production, for verification that the driving criteria are consistency achieving the design capacities.

A minimum of 2% of the piles should be tested when dynamic (or static) testing is specified. It may be necessary to test a higher percentage, say 5% of the piles, when difficult driving is expected, variable or inconsistent soil conditions are expected, or when additional tests during production are necessary to verity hammer performance and geotechnical capacities.

Driving stresses in steel piles, in compression and tension, should not exceed 90% of the yield strength of the pile material. For A-50 steel, this results in a maximum driving stress of 45 ksi. Driving stresses in concrete filled pipe piles, if unfilled when driven, should not exceed 90% of the yield strength of the steel shell material. Driving compressive stresses in precast, prestressed concrete piles should not exceed 0.85 times the concrete compressive strength, minus the effective prestress after losses. Tensile stresses are limited to 0.095 times the square root of the compressive strength (ksi) plus the effective prestress (ksi). The tension and compression driving stress limits are on the gross concrete area. Driving stresses in conventionally reinforced concrete piles should be limited to 0.85 f_c in compression and 0.70 f_y of the steel in tension.

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.
- Scour depth is large.
- Foundations are required in stream channels. Drilled shafts will avoid expensive construction of cofferdams. Advantages are the reduction of the quantities and cost of excavating, dewatering, and sheeting, and in limiting environmental impact.
- The foundation is required to resist high lateral loads or uplift loads.
- There is little tolerance for deformation.
- The cost 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, 1988.

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 5.3.3 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 5.3.4 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, and the factor of safety determined. A minimum factor of safety of 1.25 is required for embankments. This factor of safety should be increased to a minimum of 1.3 for embankments whose failure would cause significant damage, such as end slopes supporting bridge abutments and retaining walls.

If the factor of safety 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 capacity should be evaluated using the methods discussed in Section 5.3.2 Bearing Capacity. A minimum factor of safety of 3.0 should be used.

5.9.4 Embankment Seismic Considerations

A minimum seismic factor of safety of 1.0 is acceptable for slope stability and liquefaction. For bearing capacity of retaining walls and abutments, a minimum factor of safety of 1.5 is acceptable.

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 Newark 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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