SM0 31: Bridge Foundation Design

October 31, 2014

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1 Types of Foundations

The foundation types covered in this SMO are the six most commonly used foundation types for bridges in Tennessee and include:

- Spread footings supported on rock
- Pile caps supported on point bearing piles
- Pile caps supported on friction piles
- Prestressed concrete pile bents
- Pipe pile bents
- Drilled shafts

2 Spread Footings

This SMO does not address spread footings on soil. For spread footings on rock, the applicable articles from AASHTO LRFD (AASHTO 2014) and the Guide Specifications (AASHTO 2011) are summarized in this section. Hand calculations to establish compliance with some of the provisions will be necessary when the software used does not include the check.

2.1 LRFD Bridge Design Specifications (AASHTO LRFD)

Article 10.5.5.1 specifies that resistance factors at the Service Limit State, φ_b= 1.0. So, when comparing actual Service Limit State bearing pressures to “allowable” values in a geotechnical report, the “allowable” capacity should be multiplied by 1.0.

Article 10.5.5.2.2 specifies that resistance factors at the Strength Limit State, φ_b = 0.45. So, when comparing actual Strength Limit State bearing pressures to “ultimate” values in a geotechnical report, the “ultimate” capacity should be multiplied by 0.45.

Article 10.5.5.3.3 specifies a resistance factor, φ = 1.0, at the Extreme Event Limit State. So, when comparing actual Extreme Event Limit State bearing pressures to “ultimate” values in a geotechnical report, the “ultimate” capacity should be multiplied by 1.0.

Article 10.6.1.4 requires that a triangular or trapezoidal pressure distribution be assumed for determining bearing pressures for footings on rock.

Article 10.6.2.6 gives presumptive bearing capacity values at the Service Limit State. These values will typically not be required since our geotechnical reports specify bearing values.

Article 10.6.3.3 limits the eccentricity of load at the Strength Limit State to

$$ecc \leq 0.45B \tag{1}$$

Article 10.6.4.2 limits the eccentricity of load at the Extreme Event Limit state to:

$$ecc \leq \frac{1}{3}B, when\ \gamma_{EQ} = 0.0 \tag{2}$$

$$ecc \leq \frac{2}{5}B, when\ \gamma_{EQ} = 1.0 \tag{3}$$

When live load decreases the eccentricity, γ_{EQ}=0.0 is to be used.

Article 10.6.5 requires that a triangular or trapezoidal pressure distribution be assumed for structural design for footings on rock.

In the bearing resistance equations, q_n is the ultimate bearing resistance of the rock material. Our geotechnical reports typically give both the ultimate and the allowable (as some fraction of the ultimate). Use the ultimate bearing resistance in the equation at the Strength and Extreme Event Limit states, with the appropriate resistance factor. Use the allowable bearing resistance (from the geotechnical report) or the presumptive bearing resistance (from Article 10.6.2.6) at the Service Limit State.

2.2 Guide Specification for LRFD Seismic Bridge Design (Guide Specification)

Article 5.3.1 requires that foundation springs be incorporated in the structural modeling for Seismic Design Category 'D' when the Site Class is either 'C' or 'D'. For bridges in Seismic Design Category 'D', but in Site Classes 'A' or 'B', a rigid footing without foundation springs may be used. For Seismic Design Categories 'B' and 'C' on any Site Classes 'A' through 'D', a rigid foundation without foundation springs may be used. For any Seismic Design Category on a Class 'E' site, foundation springs are to be incorporated into the structural modeling.

Article 6.3.1 permits the lesser of (a) elastic seismic forces and (b) over-strength plastic hinging forces to be used for footing design at the Extreme Event Limit State for bridges in Seismic Design Category 'B'. Footings in Seismic Design Categories 'C' and 'D' are to be designed for the over-strength plastic hinging forces.

Article 6.3.2 gives dimensional requirements for footings to be classified as “rigid”. “Non-rigid” footings are outside the scope of the Specification and should be avoided. The requirement is given in Equation 4.

$$\frac{L - D_c}{2H_f} \leq 2.5 \tag{4}$$

Article 6.3.4 requires that overturning be checked in each principal direction using Equation 5.

$$M_{po} + V_{po}H_f \leq \phi P_u \left(\frac{L - a}{2}\right) \tag{5}$$

Article 6.3.6 limits the effective footing width to the value given in Equation 6. Primary flexural reinforcement is to be distributed uniformly with the effective width. Temperature and shrinkage reinforcement may be used outside the effective width.

$$b_{eff} = B_c + 2H_f \leq B \tag{6}$$

Article 6.3.7 requires that shear demand and capacity be taken at the face of the column at the Extreme Event Limit State. A minimum amount of shear reinforcing in footings is also recommended in the Commentary.

Article 6.4.5 limits principal compressive and tensile stresses in footing joints, respectively, to Equations 7 and 8.

$$p_c \leq .25f_c' \tag{7}$$

$$p_t \leq 0.38\sqrt{f_c'} \tag{8}$$

Article 6.4.7 requires that column hoops be extended into the footing in Seismic Design Categories 'C” and 'D'. Shear stirrups are required in the footing as well, a minimum of #5 bars at 12 inches, within a horizontal distance from the face of column equal to D_{ftg}.

Provisions related to eccentricity and bearing pressure limits at the Extreme Event limit state may need to be checked by hand. Footing pressure and eccentricity with the over-strength plastic moment should be evaluated in each principal footing direction for bridges in Seismic Design Categories 'B', 'C', and 'D'.

3 Pile Cap Foundations

This SMO addresses the calculation of nominal and factored resistance (capacity) for point bearing piles, friction piles, and pile caps. Both geotechnical and structural resistance are addressed.

For both point bearing and friction pile applications, Article 10.7.1.2 of AASHTO LRFD requires:

- the minimum center-to-center spacing of piles is 30 inches or 2.5 times the pile diameter (or width), whichever is greater
- the minimum side distance from pile edge to pile cap edge is 9 inches
- the minimum pile embedment is 12 inches for piles not attached to the pile cap
- the minimum pile embedment is 6 inches if the pile is positively attached to the pile cap with V-bars or extended strands

The TDOT Standard Specification for Road and Bridge Construction defines requirements for pilot holes in Article 606.12. It is good practice to refer to this article on the Plans whenever the geotechnical report or other considerations require pilot holes for piling. When pilot holes are required by the geotechnical report, notes should be included in the plans stating that the cost of the pilot holes is to be included in the bid price for piling. In exceptional cases (for example, when large seams or voids are identified in the geotechnical report), a separate bid item may be required for the pilot holes.

3.1 Point Bearing Piles

Point bearing piles are common in Regions 1, 2, and 3 and are typically HP-piles. Pipe piles are sometimes used in point bearing applications. See STD-5-2 and STD-6-1 for additional details for steel piling.

3.1.1 Driving Criteria

Since we do not load test point bearing piles, the Plans should state that point bearing piles are to be driven to refusal. Article 606.08 of the TDOT Specifications for Road and Bridge Construction defines practical refusal as either:

- 15 blows per inch for 2 consecutive inches of driving
- when 2 times the minimum required bearing is achieved based on the last 6 inches of driving

Article 606.08 further specifies that the Contractor's pile driving equipment must be capable of driving to 1.5 times the Plans bearing value at 15 blows per inch. Thus, if the Specification is properly followed, the pile driven to refusal will have, at a minimum, 1.5 times the bearing capacity as calculated from the Specification. The bearing capacity equations in the Specification are the Engineering News Record formulas with the historically recommended factor of safety equal to 6 built in. AASHTO LRFD Article 10.5.5.2.3 states that the φ-factor when the ENR formula is used is to be 0.10. The resulting relationship between Strength Limit State pile capacity and the plans value of driving load is then given by Equations 9 and 10.

$$\phi R_n = (R_u)_{STRENGTH} = 0.10 \cdot 1.5 \cdot 6 \cdot R_{PLANS} = 0.90 \cdot R_{PLANS} \qquad (9)$$

$$\Longrightarrow \; R_{PLANS} = \frac{(R_u)_{STRENGTH}}{0.90} \qquad (10)$$

Alternatively, it may be desirable to specify driving criteria different from that in the TDOT Specifications. AASHTO LRFD Article 10.7.3.8.5 gives two acceptable dynamic pile formulas - The FHWA Gates Formula and the ENR Formula, both in a format which is intended to estimate ultimate geotechnical capacity of the driven pile. 'E_d' is the delivered hammer energy in ft-kips, 's' is the pile permanent set in inches per blow. 'N_b' is the number of hammer blows in the final condition in blows per inch. The ENR Formula as presented in

AASHTO LRFD Article 10.7.3.8.5 is 6 times the value obtained from Article 606.14 of the TDOT Specifications. Hence, the recommended value of 6.0 for the "safety factor" is built into the TDOT equation.

$$R_{ndr} = 1.75\sqrt{1000E_d}log_{10}(10N_b) - 100 \tag{11}$$

$$R_{ndr} = \frac{12E_d}{s + 0.1} \tag{12}$$

The appropriate φ-factors for each method are found in AASHTO LRFD Article 10.5.5.2.3 - 0.40 for the Gates Formula and, as previously noted, 0.10 for the ENR Formula. Comparisons between the two formulas are shown in Figures 1 through 3 for three different hammer specifications. As evident in the figures, the two formulas may give similar results for large hammer/stroke values while the Gates formula may give much higher ultimate capacities compared to the ENR formula for small hammer/stroke values. These are ultimate values and the φ-factor for the Gates formula is 4 times higher than that for the ENR formula. By specifying on the plans that the Modified Gates formula be used instead of the Specification (ENR) formula, it may be possible to reduce driving loads for a given, required Strength Limit State capacity. It depends on the characteristics of the Contractor's hammer, which is unknown at the design stage.

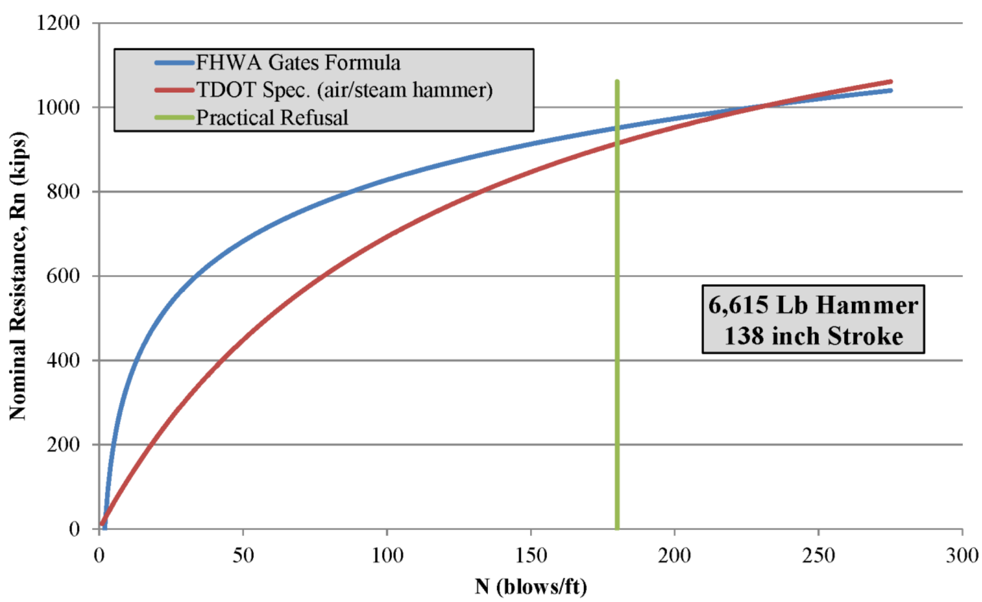


Figure 1. Gates vs. ENR - Case 1

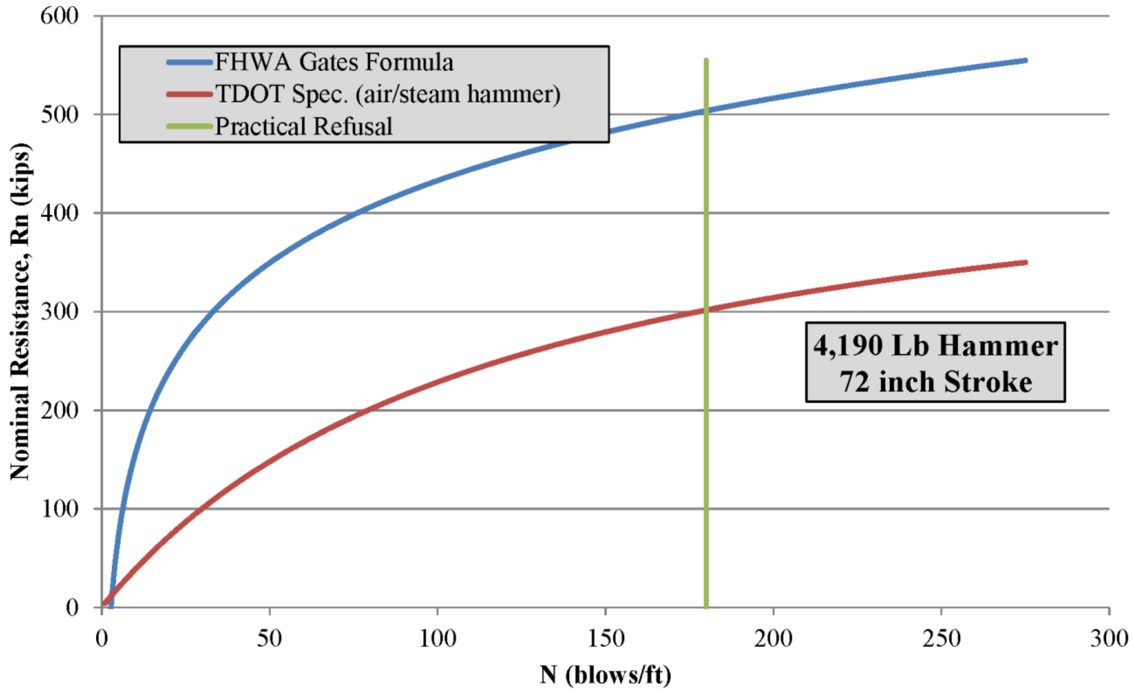


Figure 2. Gates vs. ENR - Case 2

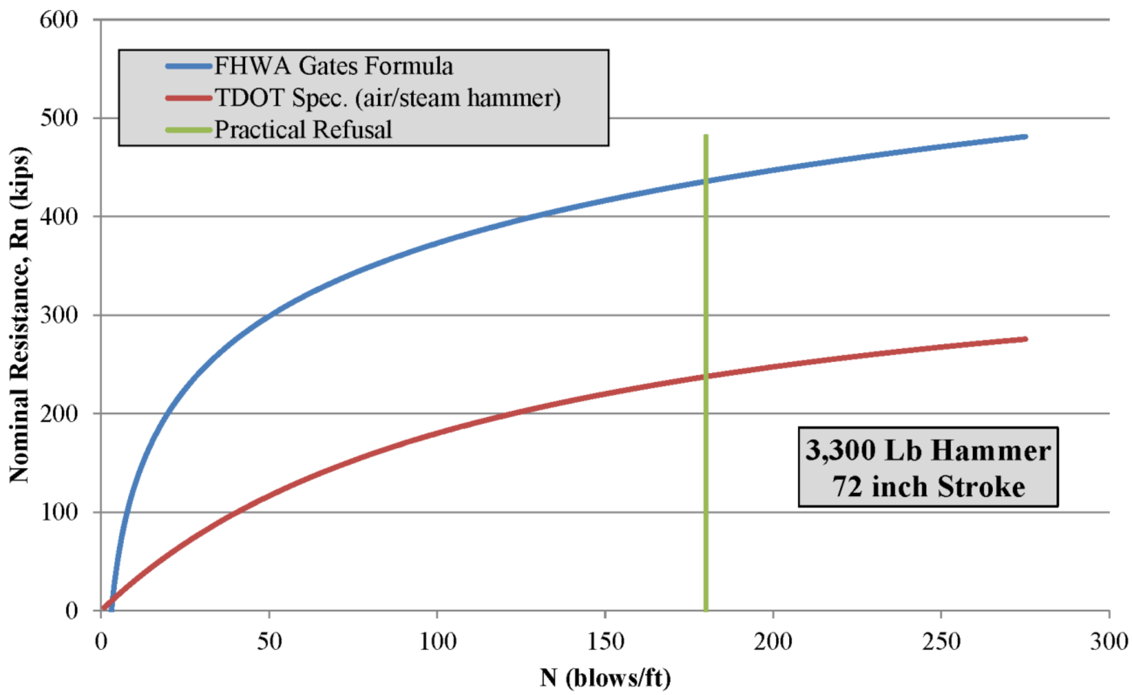


Figure 3. Gates vs. ENR - Case 3

3.1.2 Structural Capacity

Article 908.15 of the TDOT Standard Specifications for Road and Bridge Construction specify that structural steel piles shall meet the requirements for ASTM A 36. However, most material used in piling is now Grade 50. Even if Grade 36 is specified, there is a good chance that Grade 50 material will be provided by the fabricator. According to Article 105.04 of the Specifications, the order of precedence in interpreting contract documents is:

1. Special Provisions
2. Plans
3. Supplemental Specifications
4. TDOT Standard Specifications for Road and Bridge Construction

Since the likelihood is high that Grade 50 material will be the most readily available, it is acceptable practice to specify Grade 50 piles on the Plans and permit higher loads than have historically been permitted in point bearing piles. The most commonly used pile at TDOT has been the HP10X42 with a specified permissible service load stress of 0.25 X 36 = 9 ksi. This gives a load on the pile of 110 kips. If Grade 50 material for the piles is specified on the plans, then the service load used in design may be increased to 110 X (50/36) = 152 kips, or 76 tons. For cases in which abnormally high unbraced pile lengths may occur due to the presence of cavities or seams of very soft rock, consideration should be given to using piles with larger capacities (HP12X53, HP14X89, etc.) and accounting for second-order effects on axial capacity (Article 6.9.4 of AASHTO LRFD).

Pipe piles - either Spiralweld pipe or rolled pipe - come in several grades with yield strengths ranging from 30 ksi to 60 ksi or more. Specify the desired grade on the plans, based on the values used in design.

Point bearing piles are driven to refusal. From the LRFD Spec, Articles 6.5.4.2 and 6.15.2:

- φ_c= 0.50 for H-piles when tips are required (compression only)
- φ_c = 0.60 for H-piles when tips are not required (compression only)
- φ_c = 0.70 for H-piles (combined compression and flexure)
- φ_f = 1.00 for H-piles (combined compression and flexure)
- φ_c= 0.60 for pipe piles when tips are required (compression only)

- φ_c = 0.70 for pile piles when tips are not required (compression only)
- φ_c = 0.80 for pipe piles (combined compression and flexure)
- φ_f = 1.00 for pipe piles (combined compression and flexure)

Assume an un-braced pile length of 10 feet. From LRFD Spec Article 6.9.2.1, assuming λ is less than 2.25:

$$\lambda = \left(\frac{KL}{\pi r}\right)^2 \cdot \frac{F_y}{E} \tag{13}$$

$$P_n = 0.66^{\lambda} F_y A_g \tag{14}$$

Axial load – HP10X42 pile: A = 12.4 in^2, r_{min} = 2.41 inches: (Case 1, tips required; Case 2, tips not required)

$$\lambda = \left(\frac{2 \cdot 120}{2.41\pi}\right)^2 \cdot \frac{50}{29000} = 1.73 \ < 2.25 \tag{15}$$

$$P_n = 0.66^{1.73} \cdot 50 \cdot 12.4 = 302 \ kips \tag{16}$$

$$\varphi P_n = 0.5 \cdot 302 = 151 \ kips, Case\ 1 \tag{17}$$

$$\varphi P_n = 0.6 \cdot 302 = 181 \ kips, Case\ 2 \tag{18}$$

Axial load – HP12X53 pile: A = 15.5 in^2, r_{min} = 2.86 inches:

$$\lambda = \left(\frac{2 \cdot 120}{2.86\pi}\right)^2 \cdot \frac{50}{29000} = 1.23 \ < 2.25 \tag{19}$$

$$P_n = 0.66^{1.23} \cdot 50 \cdot 15.5 = 465 \ kips \tag{20}$$

$$\varphi P_n = 0.5 \cdot 465 = 232 \ kips, Case\ 1 \tag{21}$$

$$\varphi P_n = 0.6 \cdot 465 = 279 \ kips, Case\ 2 \tag{22}$$

Strength Limit State pile compression loads should be limited to these values, with any adjustment applied as needed for other unbraced lengths. Pile uplift loads at the Strength Limit State should be limited to 20 kips. This value is somewhat arbitrary and higher loads may be possible if a detailed analysis is done. See the LRFD Spec Articles 10.5.5.2.3 and 10.7.3.8.6 for more detailed information on estimating uplift resistance of point bearing piles.

For the Extreme Event Limit State, use φ-factors of 1.0 in accordance with LRFD Spec Article 10.5.5.3.3.

Axial load – HP10X42 pile:

$$P_n = 0.66^{1.73} \cdot 50 \cdot 12.4 = 302\ kips \tag{23}$$

Axial load – HP12X53 pile:

$$P_n = 0.66^{1.23} \cdot 50 \cdot 15.5 = 465\ kips \tag{24}$$

For bridges in Seismic Zones 3 and 4, a lateral load analysis of the piles should be performed to determine load-deflection characteristics of the piles. With φ-factors again taken as 1.0, determine the permissible lateral pile load in each direction using the interaction equation from the LRFD Spec Article 6.9.2.2.

The axial load-moment interaction equation is then:

$$\frac{P_u}{P_n} + \frac{8}{9} \cdot \frac{M_u}{M_n} \leq 1.00 \tag{25}$$

This interaction equation may also be used at the Strength Limit State – with appropriate φ-factors - for bridges where lateral loads from sources other than seismic are very high.

3.1.3 Geotechnical Capacity

For point bearing piles driven to refusal on hard rock, Article 10.7.3.2.3 of AASHTO LRFD stipulates that the nominal resistance, R_n is controlled by the structural limit state (the structural capacity of the pile at the Strength Limit State). Occasionally, geotechnical reports will specify a limiting value for end nearing on the pile based on geotechnical considerations. These values are typically very low and cannot be met with reasonable pile arrangements. In such cases, consult with the geotechnical engineer and verify that driving to refusal precludes the necessity of specifying these geotechnical bearing capacities for point bearing piles.

3.2 Friction Piles

Friction piles are common in Region 4 and are typically 14 inch, 16 inch, or 18 inch prestressed piles. See STD-5-1 and STD-6-1 for additional details of concrete piling. All friction piles are verified for nominal bearing resistance using either (a) static load tests (AASHTO LRFD Article 10.7.3.8.2) or (b) dynamic testing (PDA - AASHTO LRFD Article

10.7.3.8.3). So the problem associated with bearing piles and the use of the ENR formula does not exist with friction pile applications. While the ENR formula is used in the field, load testing or dynamic analysis is used to correct the values indicated by the ENR formula.

3.2.1 Driving Criteria

AASHTO LRFD Article 10.7.3.8 permits the use of the Davisson failure criteria along with static load testing for piles 24 inches and less in diameter/width. This is the typical method of friction pile capacity verification. AASHTO LRFD Article 10.5.5.2.3 defines the φ-factor to be used for various testing conditions.

- φ = 0.75, static load test of at least 1 pile per site condition, no dynamic testing
- φ = 0.75, dynamic testing on 100% of the production piles
- φ = 0.65, dynamic testing of at least 2 piles per site condition, but not less than 2% of the production piles
- φ = 0.80, static load test of at least 1 pile per site condition, with dynamic testing of at least 2 piles per site condition, but not less than 2% of the production piles

The required nominal resistance should be listed on the plans. So, for example, if static load testing is specified in the plans, the Strength Limit State pile load should be limited to:

$$(R_u)_{STRENGTH} = \phi R_n = \phi R_{PLANS} \tag{26}$$

$$R_{PLANS} \geq \frac{(R_u)_{STRENGTH}}{0.75} \tag{27}$$

Suppose the maximum Strength Limit State pile compression is 180 kips and that the static load test method is to be used in the field (φ = 0.75). Then the plans designation should be to drive piles to (180 kips / 0.75) = 240 kips (120 Tons). In other words, in order to get a reliable capacity of 180 kips at the Strength Limit State the piles need to be driven to 120 Tons. Note that for the same project, but with PDA and no static load test in the field (φ = 0.65), the piles would have to be driven to (180/0.65) = 277 kips (139 Tons).

So when load test data arrives from the field, an evaluation should be made without incorporating the φ-factor. Suppose, again, that the Strength Limit State pile compression is 180 kips, with static load tests in the field. Then the required plans information would state that the piles need to be driven to 120 Tons. Suppose further that the test pile data indicated that the pile driven to 120 Tons by the driving equation actually indicated a failure load of

149 Tons using the Davisson criteria. Then the reliable Strength Limit State capacity of the test pile is determined by equation 28.

$$\phi R_n = 0.75 \cdot 149 = 112\, Tons = 224\, kips \tag{28}$$

The appropriate K-factor is then K_{LRFD}=149/120=1.24 and the appropriate instruction to the field would be to drive production piles to: (120 Tons)/1.24 = 97 Tons. Piles driven to 97 Tons by the driving equation in the Construction Specifications will reach a nominal bearing of 120 Tons. K_{LRFD} should always be limited to values no larger than 1.5.

From AASHTO LRFD, Article 10.5.5.3.3, the φ-factors for the Extreme Event Limit State are φ=1.0 for axial compression and φ=0.8 for uplift. The permissible pile loads at the Extreme Event Limit State are thus:

- times the required plans capacity in compression
- 0.80/0.35 = 2.28 times the uplift capacity from Step 3 above in uplift.

3.2.2 Structural Capacity

Pile uplift loads at the Strength and Extreme Event Limit States are limited to the lesser of:

- the geo-technical capacity
- the structural capacity of the pile in tension the structural capacity of the seismic attachment (see STD-6-1)

The structural steel capacity of the seismic attachment should be determined using a φ-factor of 0.9 at the Strength Limit State and a φ-factor of 1.0 at the Extreme Event Limit State. From STD-6-1, the seismic attachment for use with concrete piles consists of 4 C6-bars grouted 8 feet into the pile and embedded 1'-3" above the top of the pile into the footing, terminating with a hook.

- φT_n = 0.9×4×0.44×60 = 95 kips, Strength Limit State
- φT_n = 1.0×4×0.44×60 = 106 kips, Extreme Event Limit State

When evaluating pile patterns at the Extreme Event Limit State, it is permissible to apply the plastic moment of the column, not the over-strength plastic moment, to the footing when hinging is the basis of design. See Guide Spec Article 6.4.2 for this allowance.

For in-ground hinges and top of pile hinges in prestressed concrete piles, the confinement reinforcing should be designed rather than relying on the details in the Standard Drawings. It is impossible to know when a pile will reach the required bearing. Guide Spec Article C4.9 may be used in conjunction with equations from the literature (Sritharan, et al. 2008) to determine the required reinforcement ratio for a given displacement demand and geometry.

$$\mu_D = 1 + 3\left(\mu_\varphi - 1\right)\frac{L_p}{L}\left(1 - 0.5\frac{L_p}{L}\right) \tag{29}$$

$$\rho_s = \frac{4A_t}{d's} \geq 0.06\frac{f_c'}{f_{yh}} \cdot \frac{\mu_\varphi}{18}\left(2.8 + \frac{1.25P}{0.53f_c'A_g}\right) \tag{30}$$

Substitute a displacement ductility, μ_D=4, in Equation 29. Solve for μ_φ and substitute into the Equation 30 to find the required reinforcement ratio.

Figures 4 through 6 may be helpful in evaluating the structural capacity of friction piles.

Unless a detailed lateral load analysis of the pile-soil system is performed, lateral loads on piles should be limited to values which result in no appreciable decrease in moment capacity. The permissible pile shear, with a φ-factor of 0.90 incorporated, is estimated to be:

- 14” Pre-stressed pile – φV_n = 38 kips
- 16” Pre-stressed pile – φV_n = 56 kips
- 18” Pre-stressed pile – φV_n = 60 kips

These values were obtained from a section analysis of the piles, the results of which are shown in Figures 7 through 9. The moment-shear interaction diagrams are based on several assumptions:

- Axial loads are 100 kips (14” pile), 110 kips (16” pile), and 120 kips (18” pile);
- An effective pre-strain of 0.0059 inches/inch;
- An ultimate concrete strain equal to 0.01. This is based on the confining effects of soil and is in accordance with information from page 576 of Priestley's book (Priestley, Seible and Calvi 1996).
- The effective pre-strain is based on total estimated losses of 35 ksi: ε_{pe} = (0.75×270-35)/28,500 = 0.0059

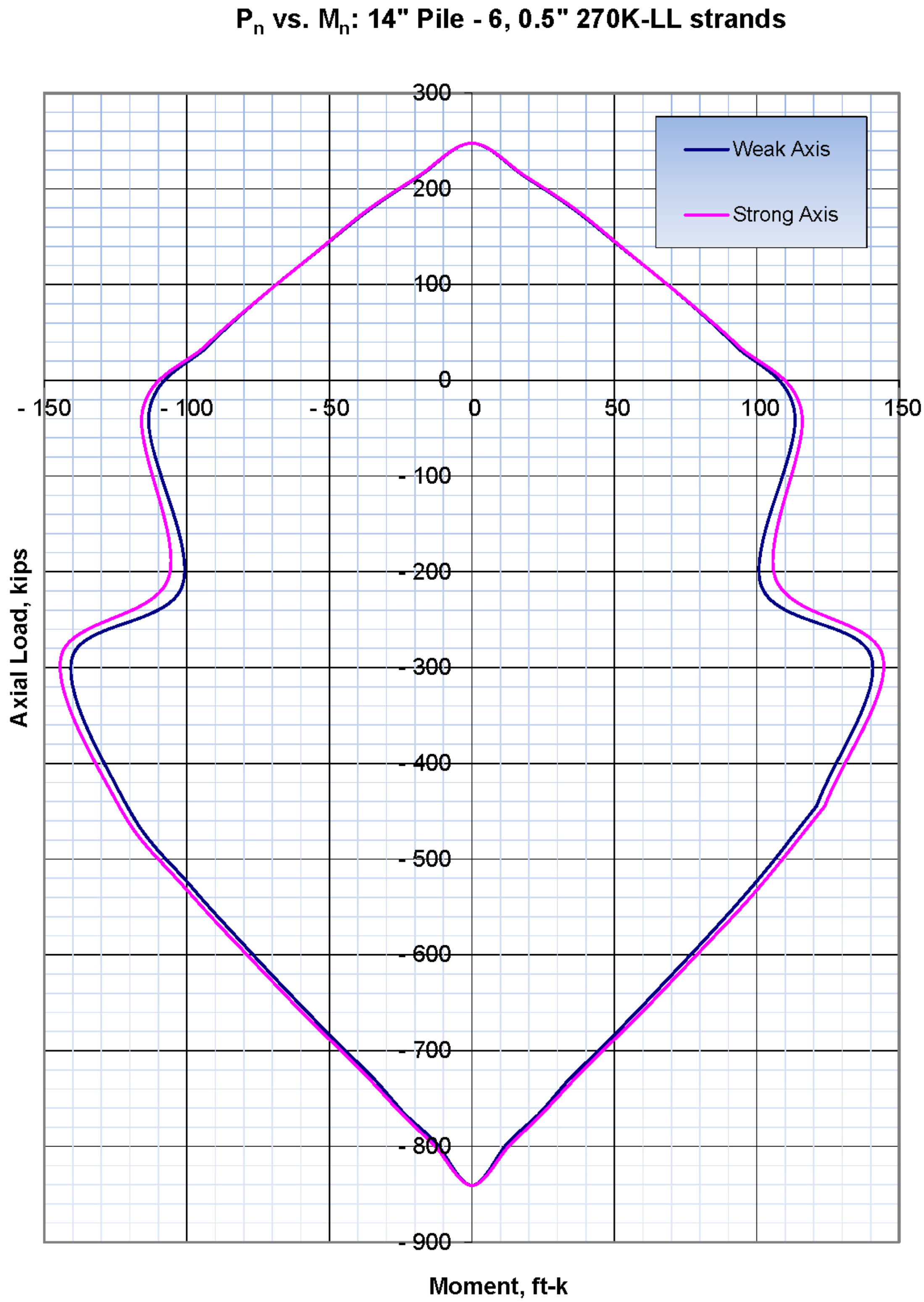


Figure 4. 14-inch Pile Interaction Diagram

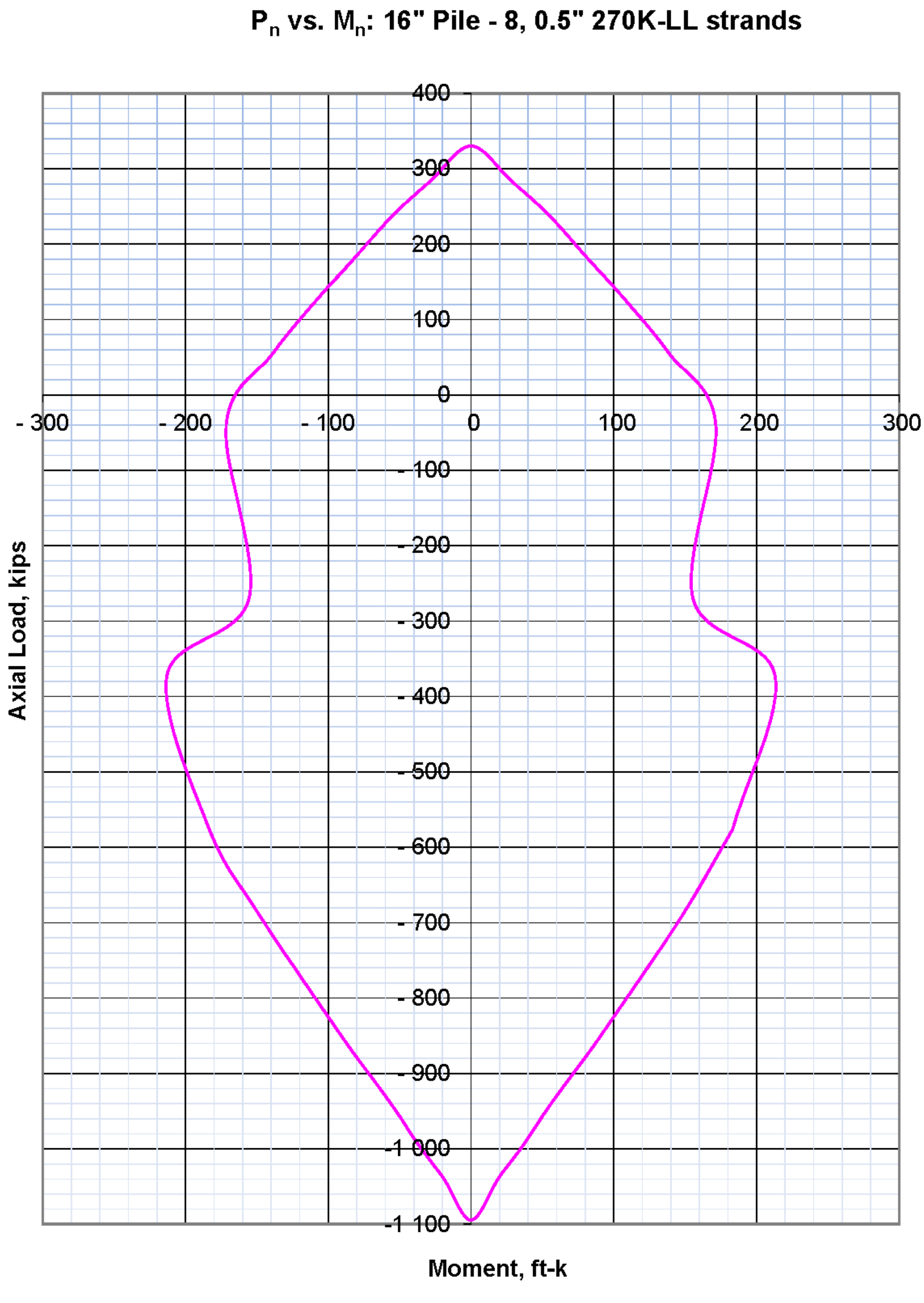


Figure 5. 16-inch Pile Interaction Diagram

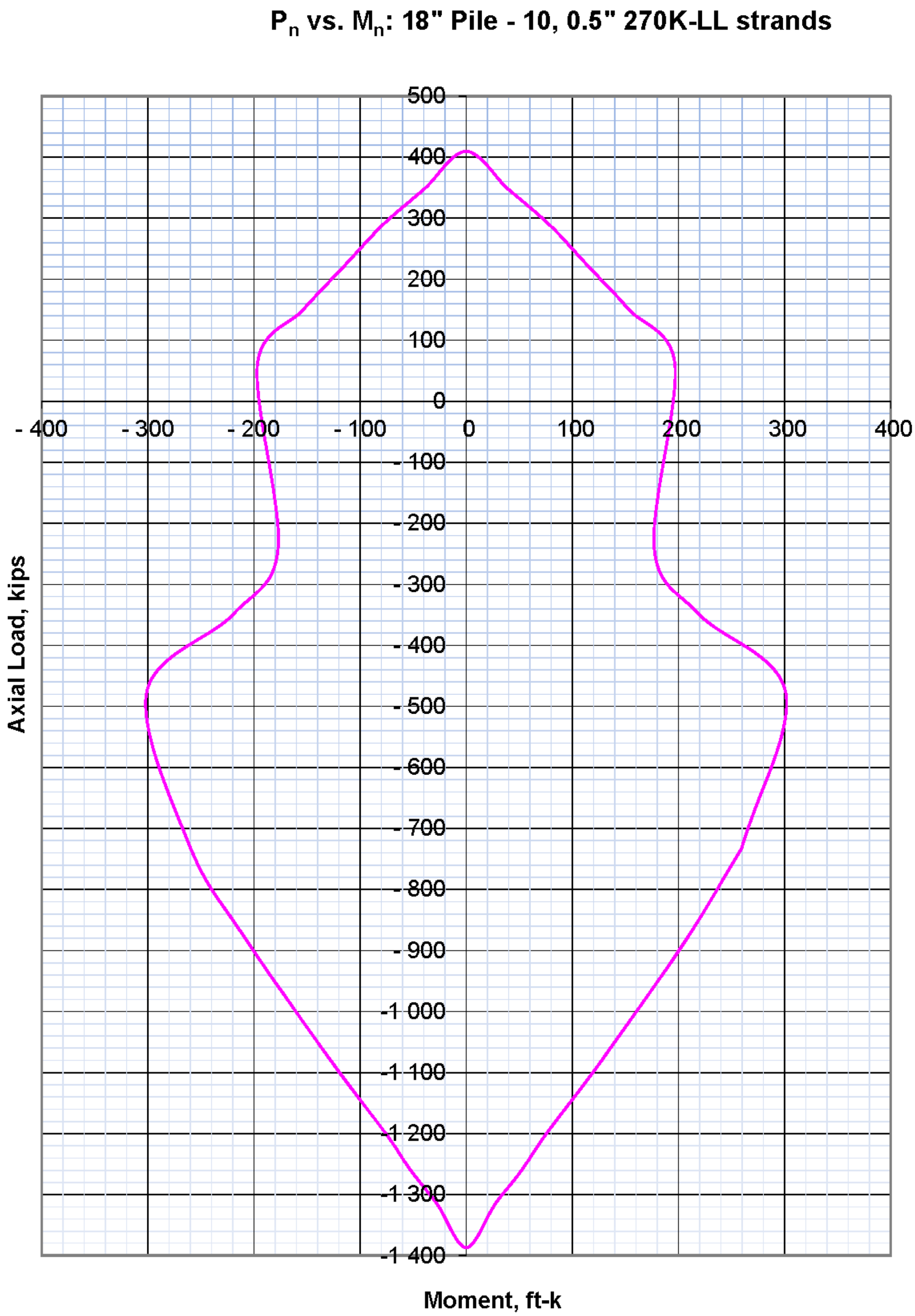
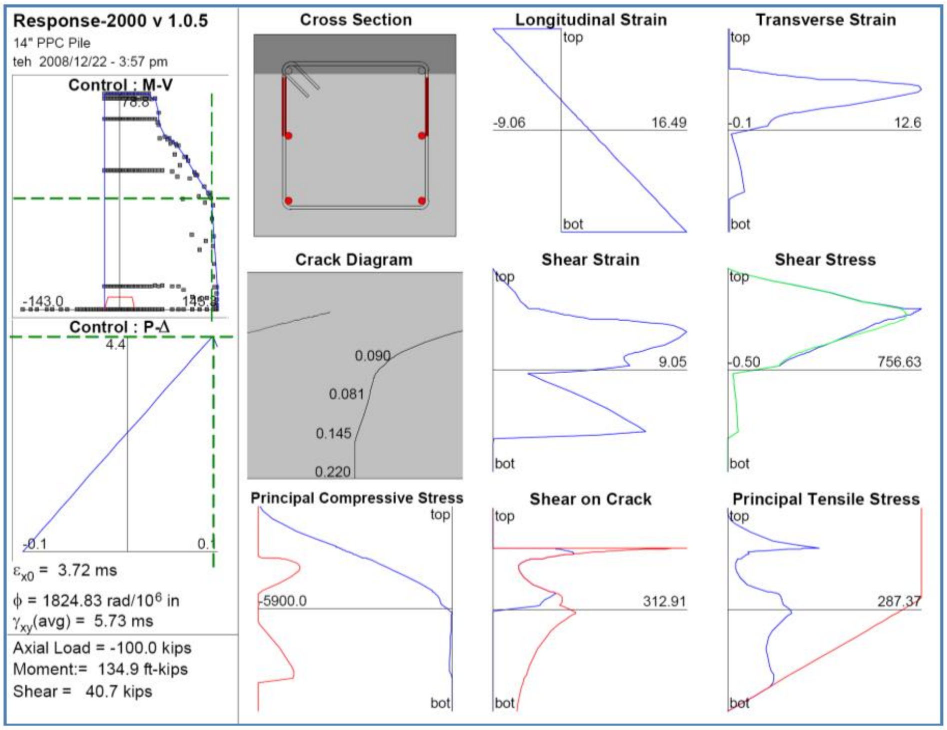


Figure 6. 18-inch Pile Interaction Diagram

14" Pre-stressed Concrete Pile – Section Analysis

M-V interaction with N = -100 kips:



M-N interaction with V = 38 kips:

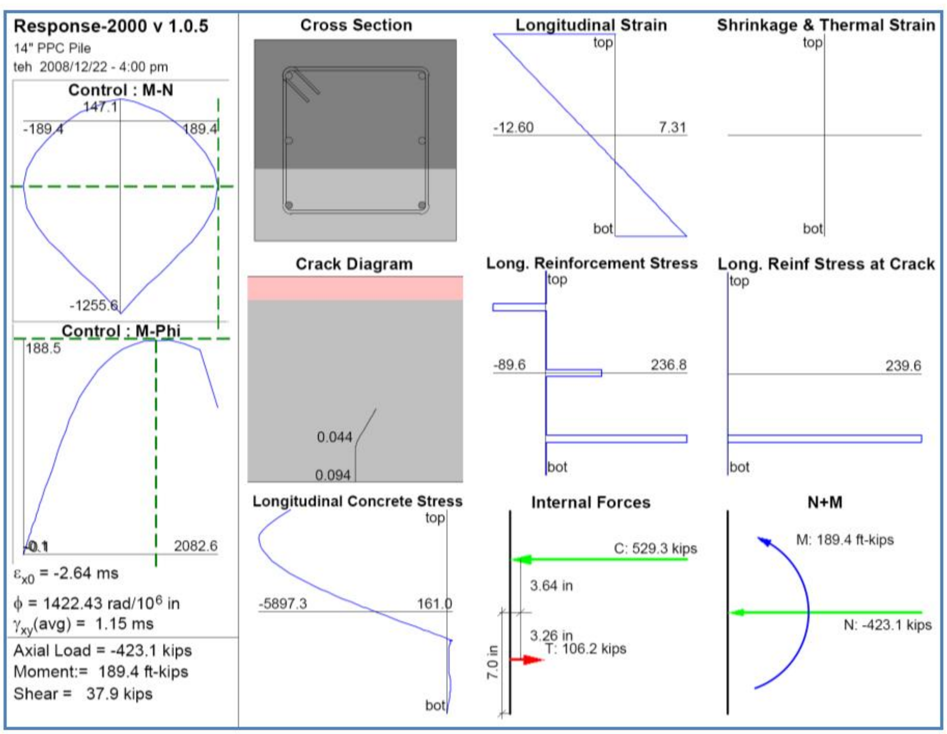
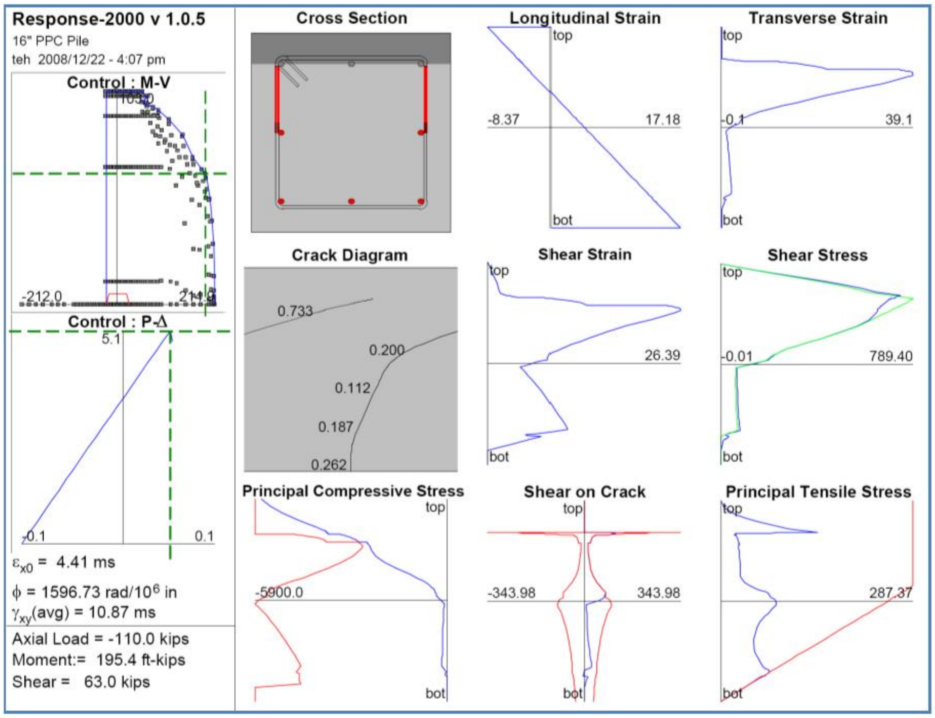


Figure 7. 14-inch Pile Shear Capacity

16" Pre-stressed Concrete Pile – Section Analysis

M-V interaction with N = -110 kips:



M-N interaction with V = 56 kips:

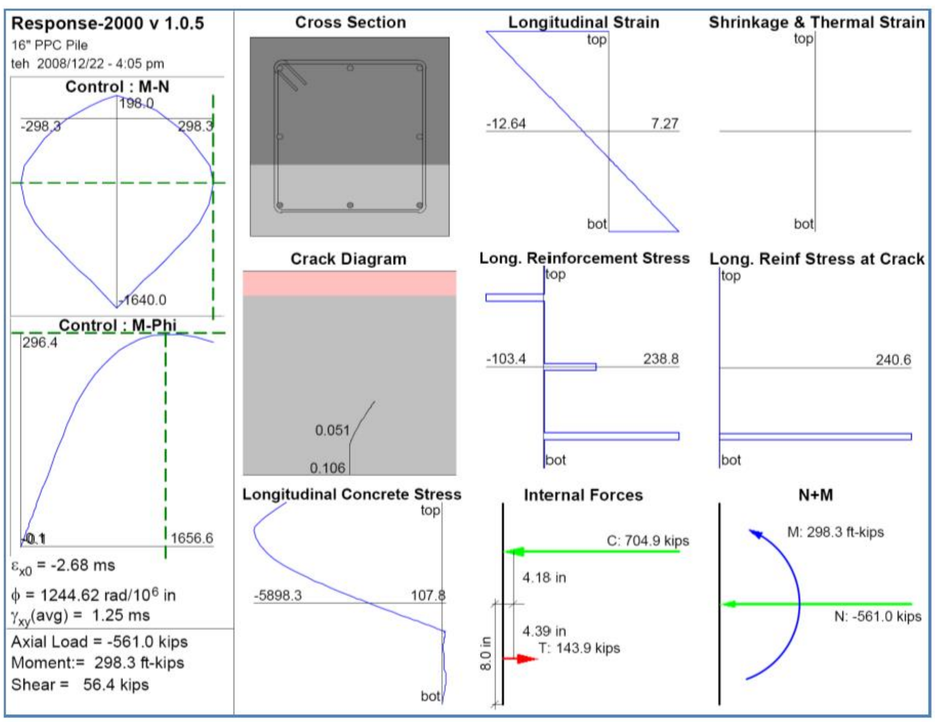
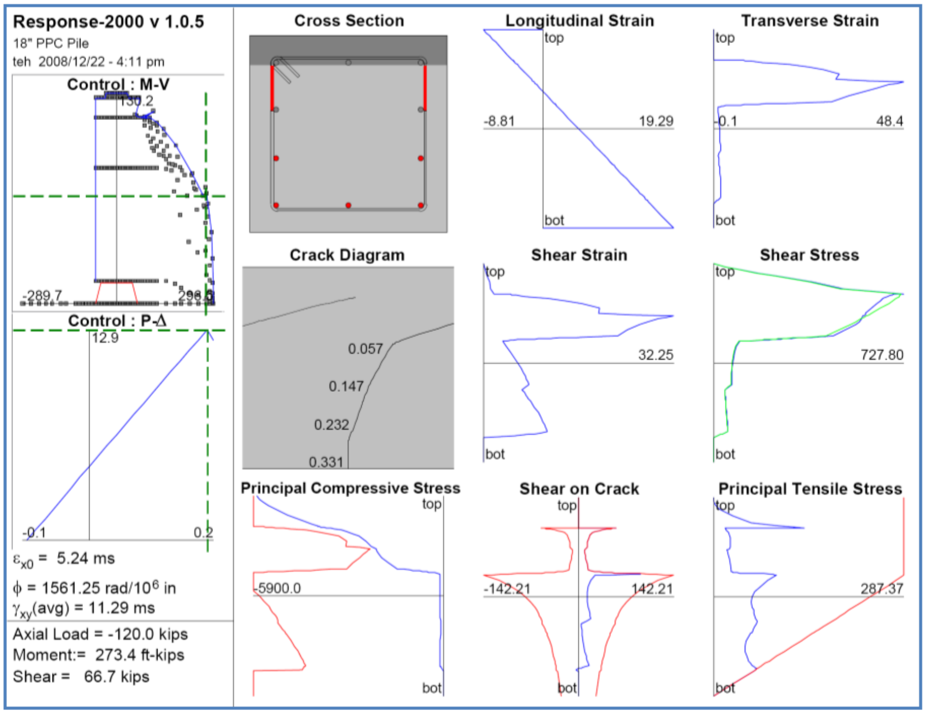


Figure 8. 16-inch Pile Shear Capacity

18” Pre-stressed Concrete Pile – Section Analysis

M-V interaction with N = -120 kips:



M-N interaction with V = 60 kips:

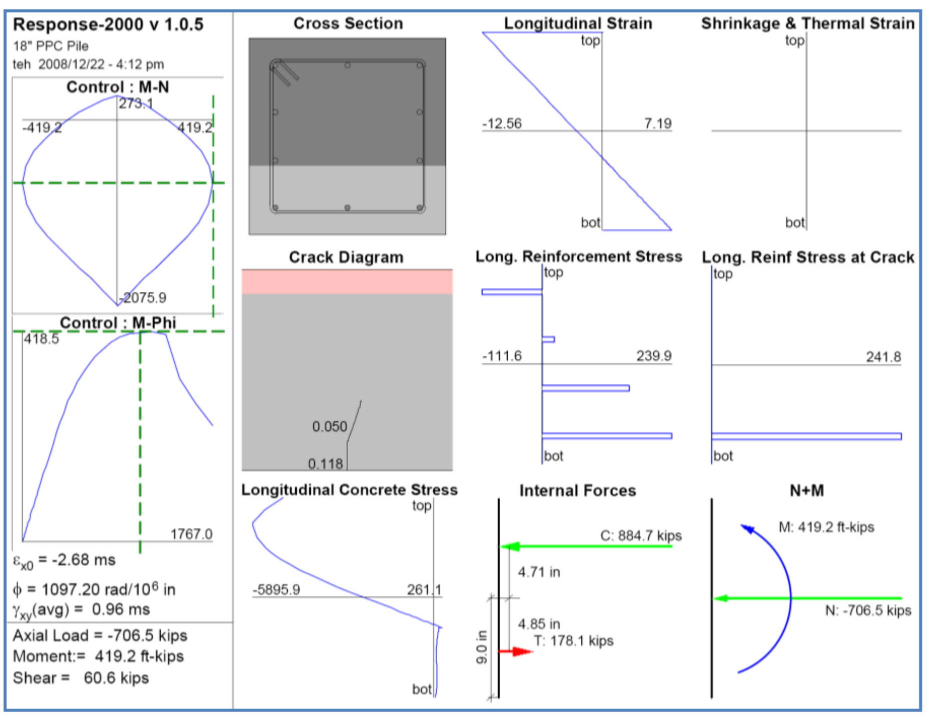


Figure 9. 18-inch Pile Shear Capacity

3.2.3 Geotechnical Capacity

The geotechnical capacity from static analysis methods is used to estimate pile lengths for the contract documents. For both friction piles and point bearing piles, larger loads are permitted at the Extreme Event Limit State than at the Strength Limit State.

1. During the design, use the ultimate end bearing, Q_b, and side friction, F_s, values provided in the geotechnical report to estimate pile lengths. The values in the report are ultimate values and need to be modified by a φ-factor taken from AASHTO LRFD, Table 10.5.5.2.3-1. Since our geotechnical group uses procedures based on Nordland methods, use φ = 0.45 for axial compression and φ = 0.35 for uplift. Develop load vs. pile depth curves for axial compression and for uplift. Incorporate the above φ-factors into the curve. Also construct a curve for axial compression using φ=1.0 to be used for pile length estimates in determining uplift capacity. If there is any question as to whether values in the geotechnical report are allowable or ultimate, contact the author of the report for clarification.
2. Using the maximum Strength Limit State pile compression reported by RC-Pier (or other methods if appropriate), read the required pile depth from the axial compression (φ=0.45) curve.
3. To be conservative on estimating permissible uplift on piles, determine the length of pile needed to achieve required bearing from the axial compression (φ=1.0) load vs. depth curve. Use this depth to read the geotechnical capacity in uplift from the φ=0.35 uplift curve developed in Step 1. The structural capacity of TDOT’s seismic attachment (see STD-6-1) is 95 kips. Use the smaller of the geotechnical capacity or the structural capacity as the permissible uplift on a single pile at the Strength Limit State. If this is greater than the actual uplift from RC-Pier, return to Step 1 with a modified pile arrangement.

The sole purpose of Steps 1 through 3 is to establish pile length estimates, determine the pile group arrangement to satisfy Strength Limit State requirements, and to set required bearing values. Once any iteration is completed and an acceptable arrangement has been determined, proceed to evaluation of pile loads at the Extreme Event Limit State.

4 Pile Bents

The depth-to-fixity for displacement should be taken as 4D-5D as recommended in Seismic Design and Retrofit of Bridges (Priestley, Seible and Calvi 1996), page 284. The depth-to-fixity for moment is usually taken as 1D-2D, but use the same point as for displacement when this is conservative to do so. 'D' is the pile width or diameter. The following notes on seismic design for pile bent substructures should be used as a general guide in the seismic design of these structures.

1. For analyzing the scoured condition when scour is severe, it may be advisable to treat the Pile Bent as being braced by diaphragm action of the deck carrying load to the abutments. Choose the k-factor accordingly after consulting with the CE Manager 1 and CE Manager 2 on the project.
2. Specify minimum tip elevation to be at least 10 feet below scour. In addition, determine the elevation required to obtain Strength Limit State bearing with all material above the scour-line absent, using the same resistance factors as for the non-scoured condition (See AASHTO LRFD Article 10.7.3.6). The minimum tip from this analysis should be compared to the 10’ below scour criteria. The lower elevation of the two should be specified as the minimum tip elevation.
3. Perform seismic analysis on the structure for both the scoured and the un-scoured conditions. It may be difficult to perform a suitable conventional design in West Tennessee bridges with high scour potential due to the need for larger piles to meet stability requirements in the scoured condition contrasted with the need for smaller piles in the un-scoured condition to meet displacement capacity limits. In such cases, non-conventional strategies such as seismic isolation or ductile superstructures may need to be considered
4. Liquefiable layers should be handled similarly to scour.

4.1 Prestressed Concrete Pile Bents

Prestressed concrete pile bents are frequently used in West Tennessee.

Guide Specification Article 8.4.3 limits the strain in pre-stressing strand to 0.03. Most of the software we use defaults the rupture strain to 0.043. So if you do a section analysis of a PPC pile, make sure you use a maximum strain in the strand of 0.03. In fact, the article gives other limits which may be used if it is desired to limit the amount of damage to in-ground hinges.

Guide Specification Article 4.9 limits the displacement ductility demand, μ_D, for in-ground hinges in PPC pile bents to 4 for Seismic Design Category D.

For determining section capacities of PPC Piles at the top of the pile, assume that the effective pre-strain in the strands is 0, i.e., essentially a reinforced section and not a prestressed section. In determining section capacities at in-ground hinges of PPC Piles, use an effective pre-strain in accordance with STD-5-1 - use an effective strand pre-strain corresponding to 700 psi uniform stress on the concrete.

4.2 Steel Pipe Pile Bents

Steel pipe pile bents are sometimes required in high seismic regions when it is difficult to make prestressed pile bents satisfy displacement capacity requirements or when unusually large pile lengths to fixity are required.

For pipe pile bents, the appropriate limiting value on ductility demand should be that for multi-column bents in Guide Specification Article 4.9, $\mu_D = 6$.

Note on the plans that Pipe Piles are to be fabricated in accordance with ASTM A 252, Grade 2 or 3. Material is to ASTM A 572 with Supplemental Requirement S18 that the tensile strength shall be no more than 69 ksi. Base the Extreme Event limit state moment capacity of the pipe piles on a stress of 69 ksi with no over-strength factor in determining the reinforcing requirements needed to protect the cap. Base the pile capacity for Strength and Service limit states on a stress of 35 ksi, the minimum yield for the specified material.

The D/t ratio must be within the limits of Guide Specification Article 7.4.2. If the expected ductility demand, μ_D, is greater than 1, then λ_p for Ductile Members must be used. If the piles remain elastic, then the Essentially Elastic λ_p may be used. Note that the requirements are more severe the higher the yield strength of the material. Therefore, do not permit substitutions of higher grade material than that specified on the plans. ($D/t \leq \lambda_p = 0.044E/F_y$ for ductile members; $D/t \leq \lambda_p = 0.09E/F_y$ for essentially elastic members).

In situations where it is desirable to use thinner walls, use piles strong enough to remain elastic or consider using Concrete-Filled-Tubes (CFT). See Guide Specification Article 7.6 for CFT design requirements.

For pipe piles, the fully plastic moment can be calculated with Equations 31-37.

D = outside diameter of pipe

t = pipe wall thickness

P_u = Axial compressive load on the pile

F_y = Yield stress of the pipe pile material

$$R = \frac{D - t}{2} \tag{31}$$

$$\alpha = \frac{2\pi R F_y t - P_u}{4 R F_y t} \tag{32}$$

$$T = 2R\alpha F_y t \tag{33}$$

$$C = 2R(\pi - \alpha) F_y t \tag{34}$$

$$x_T = \frac{R sin\alpha}{\alpha} \tag{35}$$

$$x_C = \frac{\alpha}{\pi - \alpha} \cdot x_T \tag{36}$$

$$M_p = T \cdot x_T + C \cdot x_C \tag{37}$$

Required embedment of piles into caps may be determined from the literature (Harn, Mays and Johnson 2010). Figure 10 details the calculation of the embedment capacity, V_u. Figure 11 provides additional guidance on general details not covered by Standards 5-1, 5-2, 6-1 and 6-2.

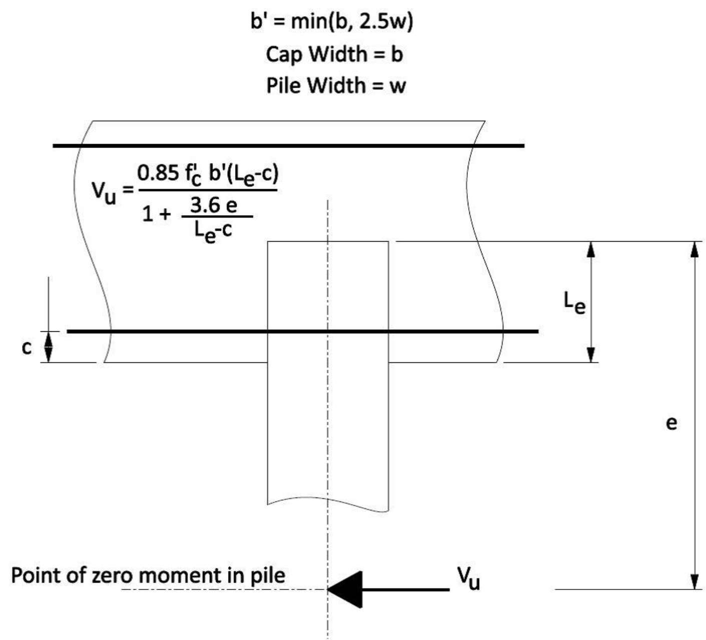


Figure 10. Pile Embedment Requirements

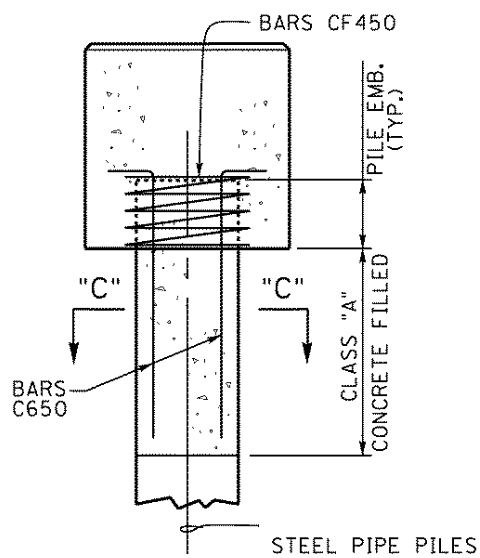


Figure 11. Pipe Pile Connection Details

5 Drilled Shafts

Design shaft socket length estimates should be based on φ-factors of 0.50 for both end bearing and side resistance in rock (AASHTO LRFD Table 10.5.5.2.4-1) at the Strength Limit State. Use φ-factors of 1.0 at the Extreme Event Limit State in accordance with AASHTO LRFD Article 10.5.5.3.3.

Cross-hole Sonic Logging tests (Item Number 625-02.46) should be specified for all drilled shafts. Specify 1 tube per foot of shaft diameter, but not less than 3. It is best to specify Core Drilling and Sampling (Item Number 204-05.01) at each shaft as a means of verifying estimated shaft tip elevations. This is strongly recommended. The other alternative is to specify inspection of the shaft by soundings performed by a geotechnical engineer after completing drilling of the rock socket, which is dangerous to the person performing the soundings and troublesome if the soundings indicate the need to extend the socket. Drilled Shafts have item specific item numbers and quantities should be broken down to include the specific pay items instead of lumping Drilled Shaft materials in with other items. The pay items are:

- 625-02.01. Drilled Shaft - Soil (xx diameter)
- 625-02.13. Drilled Shaft - Rock (xx diameter)
- 625-02.25. Drilled Shaft Casing - Permanent (xx diameter)
- 625-02.40. Drilled Shaft Concrete
- 625-02.44. Drilled Shaft Reinforcing Steel
- 625-02.46. Sonic Logging Testing
- 625-02.47. Drilled Shaft Load Test

In general, load testing is not specified. Unusual circumstances may require load testing however. Permanent casing is not required on all jobs, but the Geotechnical section should be consulted to determine any requirements for permanent casing.

For shaft design, a point of fixity 5 shaft diameters below the top of the ground may be assumed. Whenever possible, rely only upon bearing capacity and ignore side resistance. When required for design, the Geotechnical section should be consulted regarding the possibility of using a nominal resistance based on combined side friction and end bearing.

6 Cofferdams for Seal Footings

When pier footings are to be placed a significant depth below the water surface, the use of cofferdams is necessary. After constructing the cofferdam, it must be sealed so as to make it as nearly water-tight as possible. Pumps are then used to de-water the cofferdam. If the sealing efforts are successful, the cofferdam will remain relatively dry, allowing construction of the structural footing.

For foundations on porous material or fractured rock, de-watering may be impossible as water removed by the pumps is replaced with water forced up through the foundation material. A structural footing cannot be constructed in turbulent or unstable water conditions. In this situation a seal footing will be required. The seal serves to resist the static head created by the water elevation differential between the inside and the outside of the cofferdam.

Prior to pouring the seal, the cofferdam is filled with water to an elevation equal to that outside the cofferdam. The hydrostatic pressure is thus stabilized and the seal concrete may be poured using a tremie to deposit concrete evenly and uniformly in the bottom of the cofferdam. The structural footing could not be constructed in this manner since the reinforcing would prevent the necessary freedom of movement.

The cofferdam must be de-watered after construction of the seal, so the seal must be thick enough to balance the hydrostatic pressure from the water outside the cofferdam. Using unit weights of 62.4 pcf for water and 145 pcf for unreinforced concrete gives a required seal footing thickness of 0.43 times the outside water depth. If the cofferdam is constructed in a manner to provide adequate anchorage to the seal footing, then the cofferdam weight may be included in the calculations to reduce the required seal footing thickness.

When seal footings are required for submerged footings on piles, a portion of the frictional resistance of the piles in uplift may be used in resisting the hydrostatic forces. An upward load test may be required to verify this resistance.

When sizing a seal footing, a 3'0” work zone is to be provided on two sides of the structural footing on top of the seal. On the other two sides a 1'6” dimension is to be provided. The 3'0” work zone gives the Contractor room to place pumps for de-watering the cofferdam. The least plan dimension of the seal shall not be less than 1/2 the seal depth.

6.1 References for Seal Footings

Seal footings have been used on each of the following projects and the referenced drawing numbers may be consulted for guidance on seal footing details.

- M-212-46, SR-76 over the Tennessee River in Henry-Stewart counties
- M-382-67, SR-58 over the Clinch River in Roane County
- M-149-1, Briley Parkway over the Cumberland River in Davidson County
- M-329-1, SR-53 over Martin Creek in Jackson County

Provisions regarding seal footings and cofferdams are also found in Articles 204.09, 204.10 and 604.18 of the TDOT Standard Specifications for Road and Bridge Construction and in Special Provision 604F.

6.2 Foundation Preparation Guidelines

Depending on the foundation conditions, one or more of the following bid items may be required.

- 204-09.01 Cofferdam (DESC)
- 204-10.01 Foundation Preparation (DESC)
- 604-03.05 Class A Concrete (Foundation Seal)
- 604-03.25 Class S Concrete (Foundation Seal)

Foundation preparation items in the plans are generally not required where one of the following conditions exists:

- The bottom of the footing is above the datum elevation provided by Hydraulics.
- An overflow bridge with normally dry conditions.
- The depth of normal flow is 2 feet or less measured at the datum elevation.
- The channel bed is exposed rock.
- The channel bed is within 1 foot of the ground-line.
- The channel width is no more than 15 feet.
- The average flow is less than 10 cfs.
- The drainage area is less than 10 square miles.

For conditions which fall outside of those defined above, the required bid items depend on whether or not a seal footing is required. If a seal footing is not required, use the

Foundation Preparation bid item on the plans. The following rules are helpful in deciding whether or not a seal footing is required.

A seal footing is generally not required when both of the following conditions are met.

- The water depth from datum to the bottom of the footing is less than 15 feet.
- The footing is either (a) a pile cap in coarse-grained material or (b) founded on flat, solid rock with no seams or cavities.

For pile caps in fine-grained material, a seal footing is generally not required when it is reasonable to assume that a dry hole is attainable. This is generally achievable by the Contractor through driving sheet piling sufficiently below the bottom of the footing to effectively plug the cofferdam.

In cases where it is determined that a seal footing is required, the contract plans should include bid items for both Cofferdams and Seal Concrete as well as Special Provision 604F.

Works Cited

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