INDOT Bridge Design Conference - 2016

Structural Analysis of Unbraced Piles

February 16, 2016

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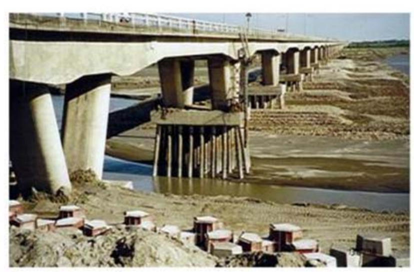
Structural Analysis of Unbraced Piles

Why do we need to worry about unbraced pile design?



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Structural Analysis of Unbraced Piles

Why do we need to worry about unbraced pile design?

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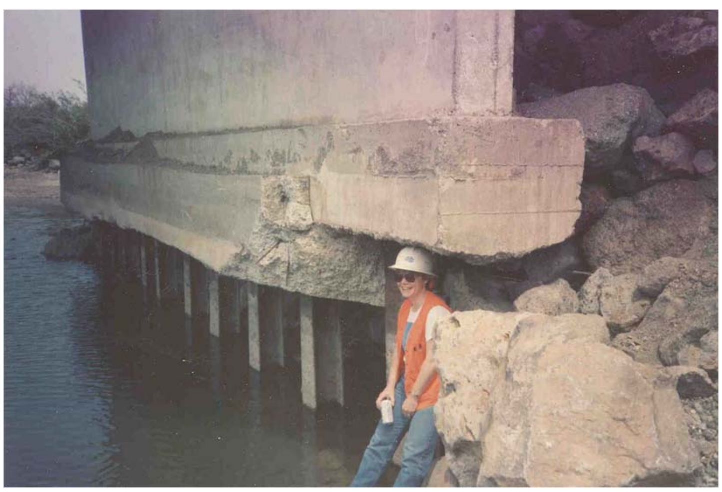
Structural Analysis of Unbraced Piles



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Structural Analysis of Unbraced Piles

Geotechnical Capacity
Vs.
Structural Pile Capacity

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Structural Analysis of Unbraced Piles

- Geotechnical Pile Capacity:
  - Pile capacity through soil resistance
  - Soil resistance will consist of skin friction capacity and/or end bearing capacity
  - Pile capacities and recommendations will be given in a Geotechnical Report
  - Maximum pile reaction will be calculated with applied axial force and additional forces due to moments for Strength and Extreme Load Cases

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Structural Analysis of Unbraced Piles

- Structural Pile Capacity:
  - Steel pile capacity calculated in Section 6 of the AASTHO LRFD Bridge Design Specifications
  - Section 6.9 addresses Compression Members
  - Combined Axial Compression and Flexure Section 6.9.2.2

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When will I need to design for an unbraced pile length?

IDM: Section 408-6.0

408-6.0 STRUCTURAL CONSIDERATIONS

Reference: *LRFD* 2.6.4.4.2, 3.7.5, 10.6, 10.7.3.13, 10.8.3.9.2

Scour is not a limit state in the context of *LRFD*. It is a change in foundation condition. All of the applicable *LRFD* limit states shall be satisfied for both the as-built and scoured bridge-foundation conditions.

The consequences of the change in foundation conditions resulting from the design flood for scour shall be considered at all applicable strength- and service-limit states. The design flood for scour is the more severe of the 100-year flood or an overtopping flood of lesser recurrence. The consequences of the change in foundation conditions resulting from the check flood for scour shall be considered at the Extreme Event limits. The check flood for scour shall not exceed the 500-year flood or an overtopping flood of lesser recurrence.

A spread footing shall be used only where the stream bed is extremely stable below the footing, and where the spread footing is founded at a depth below the maximum scour computed in Section 203-3.03(03). A footing may be founded above the scour elevation where it is keyed into non-erodible rock.

The pile cap for a deep foundation, driven-open-pile bent, or drilled shaft, shall be located such that the top of the cap is below the estimated contraction-scour depth. A lower elevation shall be considered where erosion or corrosion can damage the piles or shafts. Where the cap cannot be located below the maximum scour depth, soil loss surrounding the deep foundation results in piles or shafts with unbraced lengths. The unbraced length is equal to the length of the pile or shaft exposed by the scour, plus an estimated depth to fixity. The depth to fixity shall be determined as specified in *LRFD* 10.7.3.13 for driven piles, or *LRFD* 10.8.3.9.4 for drilled shafts. The piles or shafts exposed due to scour shall be designed structurally as unbraced-length columns in accordance with *LRFD* Section 5 for a concrete foundation, or *LRFD* Section 6 for a steel foundation. Unscoured piles or shafts can be considered in structural design as continuously-braced columns.

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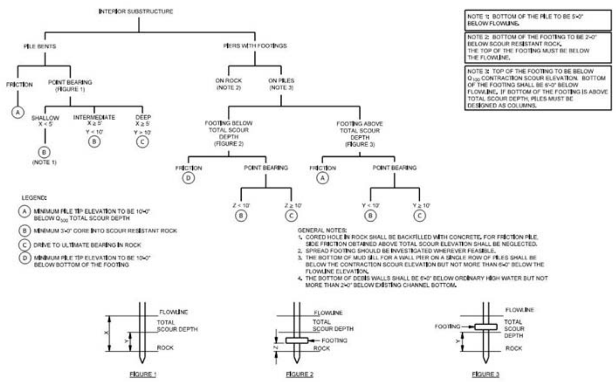
When will I need to design for an unbraced pile length?

IDM: Figure 408-3D



PILE TIP ELEVATION GUIDELINES
(For Body of Water)

Figure 408-3D

Back

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When will I need to design for an unbraced pile length?

Do I need to design for scour if my pier is not in the waterway?

YES

BUT…….

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When will I need to design for an unbraced pile length?

Do I need to design for scour if my pier is not in the waterway?

CONTACT INDOT HYDRAULICS IF ANY EXCEPTIONS CAN BE MADE.

THIS WILL BE HANDLED ON A CASE BY CASE BASIS!

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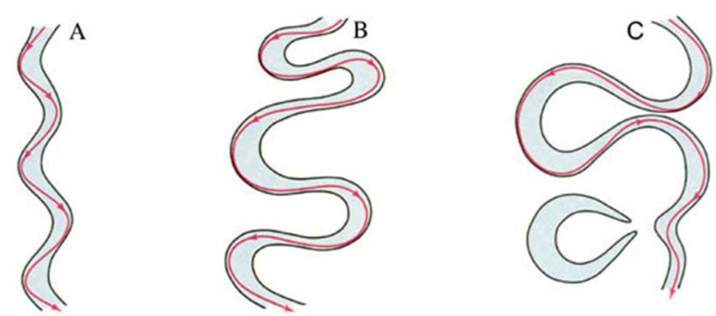
When will I need to design for an unbraced pile length?

Why do I need to design for scour if my pier is not in the waterway?

Waterways can change geometry over time:



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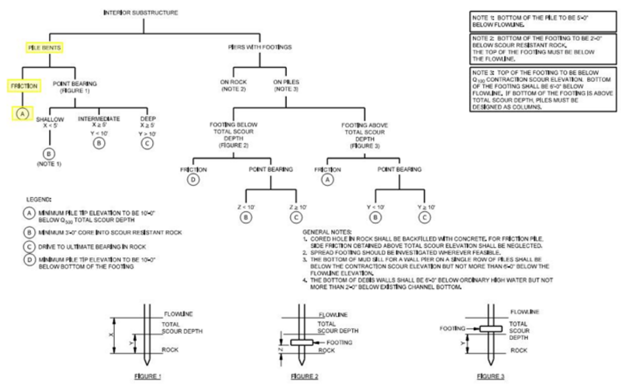
Design of Pile Bent Foundation

Friction Piles



PILE TIP ELEVATION GUIDELINES
(For Body of Water)

Figure 408-3D

Back

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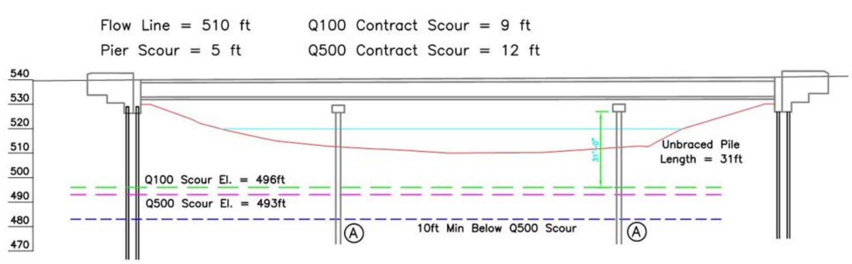
Design of Pile Bent Foundation

Friction Piles



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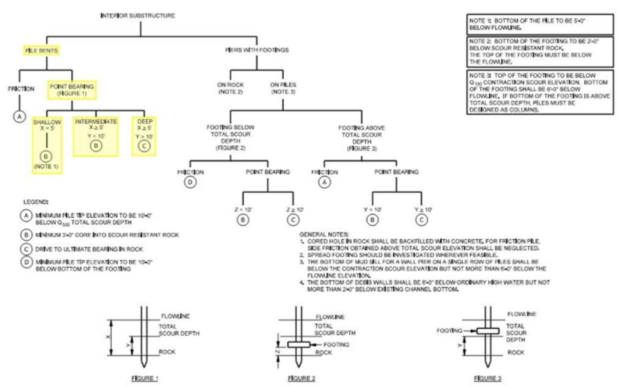
Design of Pile Bent Foundation

Point Bearing



PILE TIP ELEVATION GUIDELINES
(For Body of Water)

Figure 408-3D

Back

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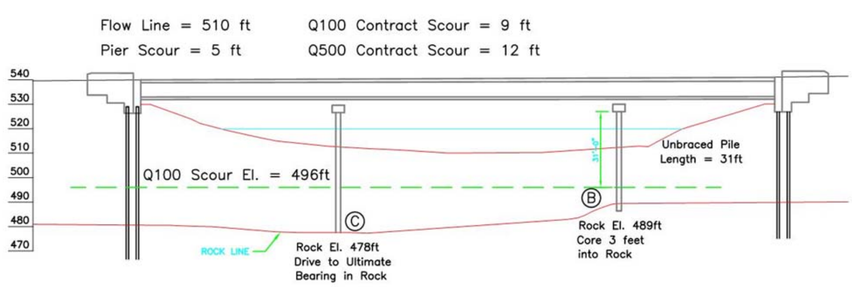
Design of Pile Bent Foundation

Point Bearing



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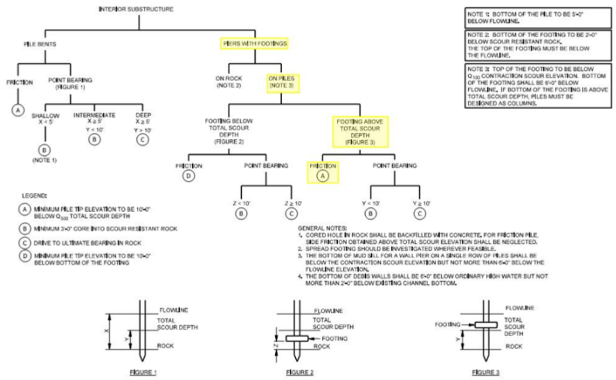
Design of Piers with Footings

Friction Piles



PILE TIP ELEVATION GUIDELINES
(For Body of Water)

Figure 408-3D

Back

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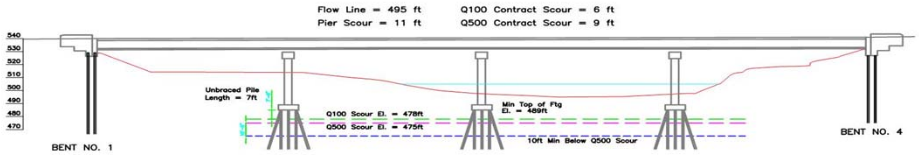
Design of Piers with Footings

Friction Piles



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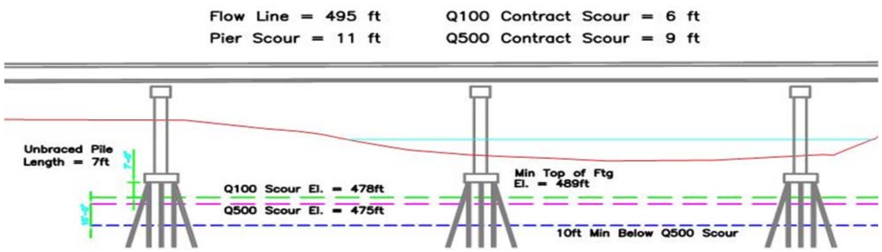
Design of Piers with Footings

Friction Piles



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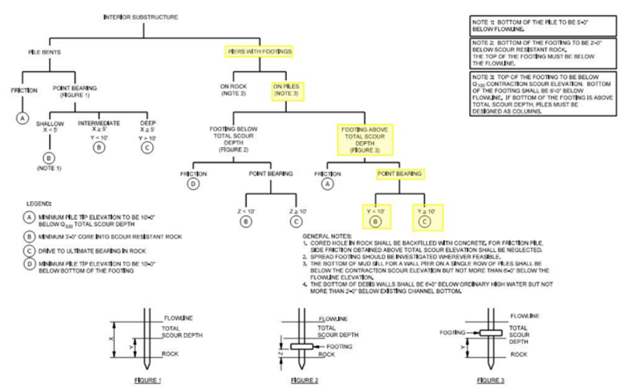
Design of Piers with Footings

Point Bearing



PILE TIP ELEVATION GUIDELINES
(For Body of Water)

Figure 408-3D

Back

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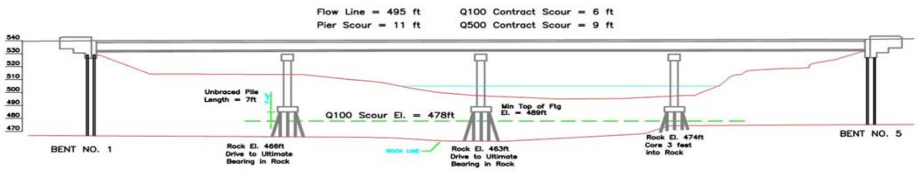
Design of Piers with Footings

Point Bearing



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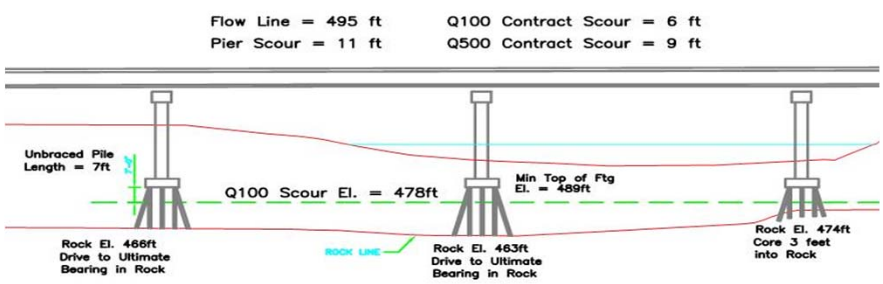
Design of Piers with Footings

Point Bearing



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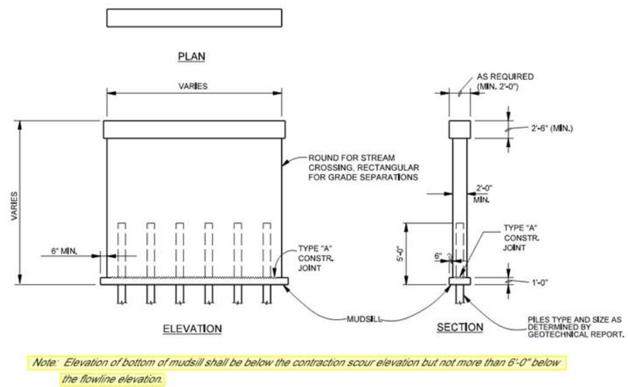
Design of Wall Piers

Friction / Point Bearing



WALL PIER ON SINGLE ROW OF PILES

Figure 409-6B

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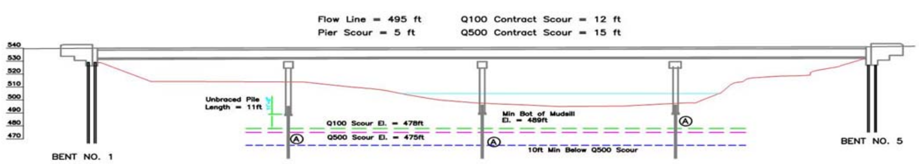
Design of Wall Piers

Friction Piles



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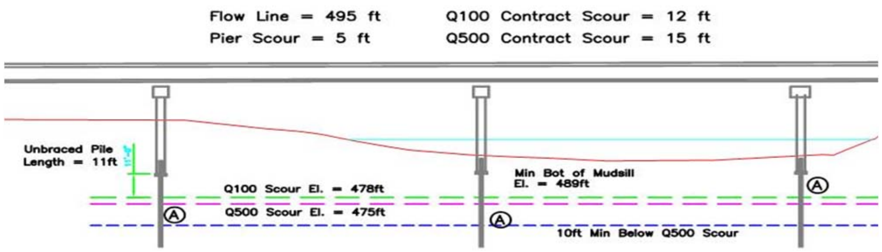
Design of Wall Piers

Friction Piles



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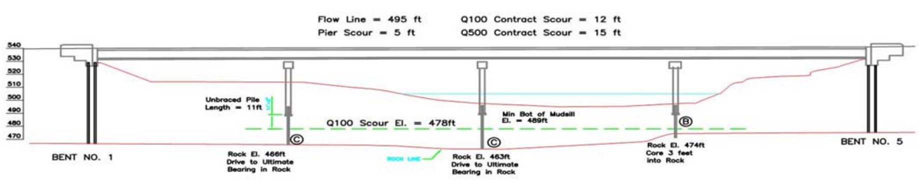
Design of Wall Piers

Point Bearing



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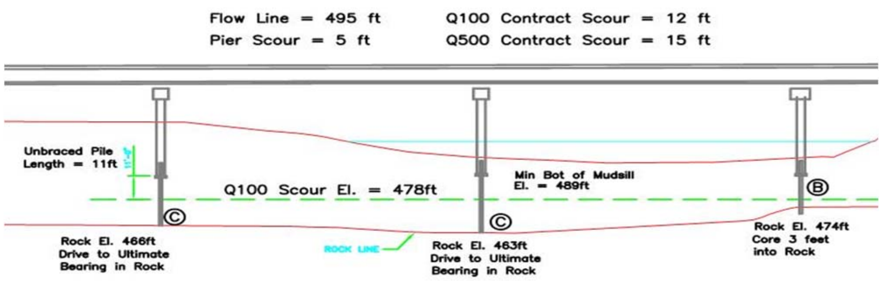
Design of Wall Piers

Point Bearing



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Structural Design of Pile

Check Combined Axial Compression and Flexure
AASHTO LRFD Section: 6.9.2.2

- Slenderness Ratio:

  $Kl/r \leq 120$ (Section 6.9.3)

- If P_u/P_r < 0.2, then

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}1)$$

- If $P_u/P_r \geq 0.2$, then

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}1)$$

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Structural Design of Pile

Combined Axial & Flexural Capacity (Compressive Resistance)

P_u = Axial Compressive Load (from Load Combinations)

P_r = Factored Compressive Resistance (Article 6.9.2.1)

$$P_r = \phi_c P_n$$

ϕ_c = 0.9 Strength Combinations (Article 6.5.4.2)
ϕ_c = 1.0 Extreme Combinations (Article 6.5.5)

P_n = Nominal Compressive Resistance (Article 6.9.4)

If $P_e/P_o \geq 0.44$, then

$$Pn = \left[0.658^{\frac{P_o}{P_e}}\right] * Po \qquad \text{(Eqn. 6.9.4.1.1-1)}$$

If $P_e/P_o < 0.44$, then

$$Pn = 0.877\text{Pe} \qquad \text{(Eqn. 6.9.4.1.1-2)}$$

Structural Design of Pile

Combined Axial & Flexural Capacity (Compressive Resistance)

P_e = Elastic Critical Buckling Resistance
Table 6.9.4.1.1-1 (FB)

$$P_e = \frac{(\pi^2)E}{\left(\frac{Kl}{r_s}\right)^2} * Ag \qquad \text{(Eqn. 6.9.4.1.2-1)}$$

K = Effective Length Factor
l = Unbraced Length
r_s = Radius of Gyration
A_g = Gross Cross-Sectional Area

(No need to check Eqn. 6.9.4.1.3-1, Elastic Torsional Buckling does not control in H or Shell Piles)

P_o = Equivalent Nominal Yield Stress
$P_o = QF_yA_g$
Q = Slender Element Reduction Factor
(H or Shell Piles are not Slender)

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Structural Analysis of Unbraced Piles

Combined Axial & Flexural Capacity (Flexural Resistance)

Back to Combined Axial & Flexural Capacity Equations:

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}1)$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}2)$$

M_{ux} = Factored Flexural Moment about x-axis (Strong-Axis)
M_{uy} = Factored Flexural Moment about y-axis (Weak-Axis)
(Obtained from Strength & Extreme Load Combinations)

M_{rx} = Factored Flexural Resistance about x-axis (Strong-Axis) = $\phi_f M_{nc}$
M_{ry} = Factored Flexural Resistance about y-axis (Weak-Axis) = $\phi_f M_n$

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Strong Axis)

To Calculate M_{rx} (Strong Axis): Both Flange Local Buckling (FLB) and Lateral Torsional Buckling (LTB) need to be calculated.

FLB:
Check Section Ratios (A6.3.2):

Slenderness Ratio for Compression Flange: $\lambda_f = \frac{b_{fc}}{2tf_c}$

b_{fc}: Compression Flange Width
t_{fc}: Compression Flange Thickness

Limiting Slenderness Ratio for Compact Flange: $\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yc}}}$

E: Young's Modulus
F_{yc}: Yield Strength of Compression Flange

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Strong Axis)

FLB:

Check Section Ratios (A6.3.2):

Limiting Slenderness Ratio for Noncompact Flange: $\lambda_{rf} = 0.95\sqrt{\frac{Ek_c}{F_{yr}}}$

k_c: Flange local buckling coefficient (for rolled shapes = 0.76)

F_{yr}: Compression-flange stress at the onset of nominal yielding within cross-section

F_{yr}: taken as the smaller of:
- $0.7F_{yc}$
- $R_h F_{yt} * (\frac{S_{xt}}{S_{xc}})$
- F_{yw}
- (But Not Less than $0.5F_{yc}$)

F_{yc}: Yield Strength of Compression Flange

R_h: Hybrid Factor = 1.0

F_{yt}: Yield Strength of Tension Flange

S_{xt}: Elastic Sect Modulus with respect to strong-axis tension flange

S_{xc}: Elastic Sect Modulus with respect to strong-axis compress flange

F_{yw}: Yield Strength of Web

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Strong Axis)

FLB:

Calculate Flexural Resistance based on Compression Flange Local Buckling:

if $\lambda_f \leq \lambda_{pf}$, then:

$$M_{nc} = R_{pc}M_{yc} \qquad \text{(Eqn. A6.3.2-1)}$$

Otherwise:

$$M_{nc} = [1 - (1 - \frac{F_{yr}S_{xc}}{R_{pc}M_{yc}})(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}})]R_{pc}M_{yc} \qquad \text{(Eqn. A6.3.2-2)}$$

R_{pc}: Web Plastification Factor for Comp. Flange
For rolled I-Shapes, R_{pc} = Shape Factor
Shape Factor = Z_x / S_x (Approx 1.10)

M_{yc}: Yield Moment = $F_y S_x$

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to LTB (Strong Axis)

LTB:
Calculate Plastic Length Limit (L_p) and Elastic Length Limit (L_r):
Section A6.3.3

$$L_p = 1.0 r_t \sqrt{\frac{E}{F_{yc}}} \qquad \text{(Eqn. A6.3.3-4)}$$

E: Young's Modulus
F_{yc}: Yield Strength of Compression Flange
r_t: Effective radius of gyration for lateral torsional buckling

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1+\frac{1}{3}\frac{D_c t_w}{b_{fc} t_{fc}}\right)}} \qquad \text{(Eqn. A6.3.3-10)}$$

`

b_{fc}: Compression Flange Width
t_{fc}: Compression Flange Thickness
D_c: Depth of Web
t_w: Web Thickness

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to LTB (Strong Axis)

LTB:
Calculate Plastic Length Limit (L_p) and Elastic Length Limit (L_r):
Section A6.3.3

$$L_r = 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_{yr}}{E} * \frac{S_{xc} h}{J}\right)^2}} \qquad \text{(Eqn. A6.3.3-5)}$$

S_{xc}: Elastic Section Modulus about Strong Axis
h: Depth between centerline of flanges
F_{yr}: Yield Strength of Compression Flange
F_{yr}: taken as the smaller of:
- $0.7F_{yc}$
- $R_h F_{yt} * (\frac{S_{xt}}{S_{xc}})$
- F_{yw}
- (But Not Less than $0.5F_{yc}$)

J: St. Venant torsional constant (Eqn. A6.3.3-9)

$$J = \left(\frac{D t^3_w}{3}\right) + \left(\frac{b_{fc} t^3_{fc}}{3}\right)\left(1 - 0.63\frac{t_{fc}}{b_{fc}}\right) + \frac{b_{ft} t^3_{ft}}{3}\left(1 - 0.63\frac{t_{ft}}{b_{ft}}\right)$$

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to LTB (Strong Axis)

LTB:

- The Limit Lengths of L_p and L_r have been calculated.
- Compare where the unbraced length, L_b, is in relation to L_p and L_r.

If $L_b \leq L_p$,

$$M_{nc} = R_{pc}M_{yc} \qquad \text{(Eqn. A6.3.3-1)}$$

R_{pc}: Web Plastification Factor for Comp. Flange
For rolled I-Shapes, R_{pc} = Shape Factor
Shape Factor = Z_x / S_x (Approx 1.10)

M_{yc}: Yield Moment = F_yS_x

Structural Analysis of Unbraced Piles

Flexural Resistance Due to LTB (Strong Axis)

LTB:

If $L_p < L_b \leq L_r$,

$$M_{nc} = Cb\left[1-\left(1-\frac{F_{yr}S_{xc}}{R_{pc}M_{yc}}\right)\left(\frac{L_b-Lp}{L_r-L_p}\right)\right]R_{pc}M_{yc} \leq Rp_{cMyc} \qquad \text{(Eqn. A6.3.3-2)}$$

F_{yr}: taken as the smaller of:

- $0.7F_{yc}$
- $R_hF_{yt} * (\frac{S_{xt}}{S_{xc}})$
- F_{yw}
- (But Not Less than $0.5F_{yc}$)

R_{pc}: Web Plastification Factor for Comp. Flange
For rolled I-Shapes, R_{pc} = Shape Factor
Shape Factor = Z_x / S_x (Approx 1.10)

M_{yc}: Yield Moment = F_yS_x
S_{xc}: Elastic Section Modulus about Strong Axis
C_b: Moment Gradient Modifier (Eqn. A6.3.3-6)

Structural Analysis of Unbraced Piles

Flexural Resistance Due to LTB (Strong Axis)

LTB:

Moment Gradient Modifier, C_b:

For an Unbraced Cantilever, C_b = 1.0 (Eqn. A6.3.3-6)

For all other cases:

$$C_b = 1.75 - 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \leq 2.3 \quad \text{(Eqn. A6.3.3-2)}$$

M_1: Moment at brace point opposite to M_2

M_2: Largest Moment at either end of the unbraced length

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to LTB (Strong Axis)

LTB:

If $L_b > L_r$,

$M_{nc} = F_{cr}S_{xc} \leq R_{pc}M_{yc}$ (Eqn. A6.3.3-3)

R_{pc}: Web Plastification Factor for Comp. Flange
For rolled I-Shapes, R_{pc} = Shape Factor
Shape Factor = Z_x / S_x (Approx 1.10)

M_{yc}: Yield Moment = F_yS_x

S_{xc}: Elastic Section Modulus about Strong Axis

F_{cr}: Elastic Lateral Torsional Buckling Stress

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t}\right)^2}\sqrt{1 + 0.078\frac{J}{S_{xc}h}\left(\frac{L_b}{r_t}\right)^2} \quad \text{(Eqn. A6.3.3-8)}$$

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Structural Analysis of Unbraced Piles

Flexural Capacity (M_{rx}, Strong Axis)

Flexural Resistance based on compression flange local buckling has been calculated.

- Compare the values of M_{nc} for FLB and LTB
- The lower value of M_{nc} will be used
- $M_{rx} = \phi_f M_{nc}$

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}1)$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}2)$$

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Structural Analysis of Unbraced Piles

Flexural Capacity (M_{ry}, Weak Axis)

Flexural Resistance for Weak Axis (Section 6.12.2.2)

- LTB does not control
- FLB controls and needs to be checked
- $M_{ry} = \phi_f M_n$

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}1)$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}2)$$

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Weak Axis)

FLB:

Check Section Ratios (Article 6.12.2.2):

Slenderness Ratio for Flange: $\lambda_f = \frac{b_f}{2tf}$

b_f: Flange Width

t_f: Flange Thickness

Limiting Slenderness Ratio for Compact Flange: $\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}}$

E: Young's Modulus

F_{yf}: Yield Strength of Flange

STRUCTUREPOINT

Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Weak Axis)

FLB:

Check Section Ratios (Article 6.12.2.2):

Limiting Slenderness Ratio for Noncompact Flange: $\lambda_{rf} = 0.83\sqrt{\frac{E}{F_{yf}}}$

E: Young's Modulus

F_{yf}: Yield Strength of Flange

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Weak Axis)

FLB:
Calculate Flexural Resistance based on Flange Local Buckling:

if $\lambda_f \leq \lambda_{pf}$, then:

$$M_n = M_p \qquad \text{(Eqn. 6.12.2.2.1-1)}$$

$$M_p = F_{yf} Z_y$$

F_{yf}: Yield Strength of Flange
Z_y: Plastic Section Modulus about Weak-Axis

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Structural Analysis of Unbraced Piles

Flexural Resistance Due to FLB (Weak Axis)

FLB:
Calculate Flexural Resistance based on Flange Local Buckling:

if $\lambda_{pf} < \lambda_f \leq \lambda_{rf}$ then:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_{yf}}}}\right)\right] F_{yf} Z_y \qquad \text{(Eqn. 6.12.2.2.1-2)}$$

S_y: Section Modulus about Weak-Axis
E: Young's Modulus
F_{yf}: Yield Strength of Flange
Z_y: Plastic Section Modulus about Weak-Axis

STRUCTUREPOINT

Structural Analysis of Unbraced Piles

Flexural Capacity (M_{ry}, Weak Axis)

Flexural Resistance based on weak-axis bending.

- $M_{ry} = \phi_f M_n$

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}1)$$

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0 \quad (6.9.2.2\text{-}2)$$

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Structural Design of Pile

Check Combined Axial Compression and Flexure
AASHTO LRFD Section: 6.9.2.2

Unbraced Pile Design is complete for combined Axial Compression and Flexure!

- Slenderness Ratio (Section 6.9.3)
  $Kl/r \leq 120$
- Combined Axial Compression and Flexure (Section 6.9.2.2)
  - Factored Axial Forces, Strong-Axis and Weak Axis Moments Calculated
  - Compressive Resistance, P_r, Calculated
  - Strong-Axis Flexural Resistance, M_{rx}, Calculated
  - Weak-Axis Flexural Resistance, M_{ry}, Calculated

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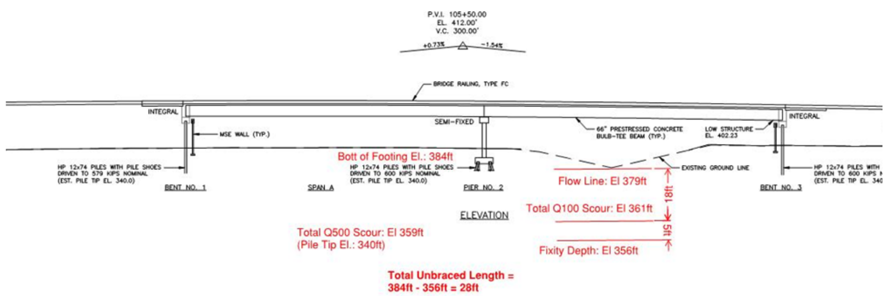
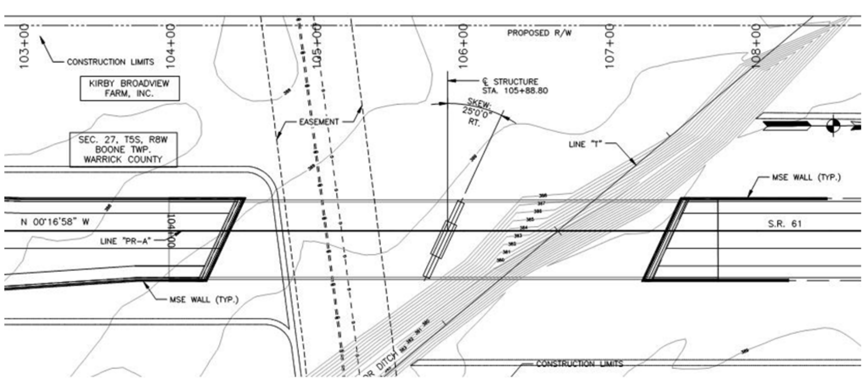
Structural Analysis of Unbraced Piles

Example 1: Two Span Structure (153ft Ea)



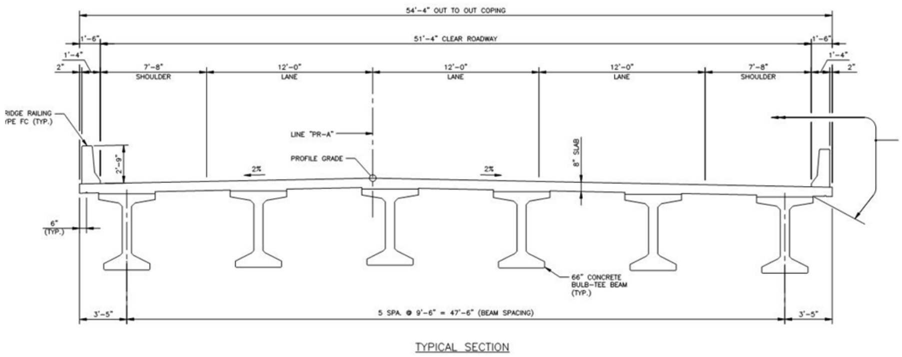
Structural Analysis of Unbraced Piles

<u>Plan View</u>



Structural Analysis of Unbraced Piles

Cross-Section View



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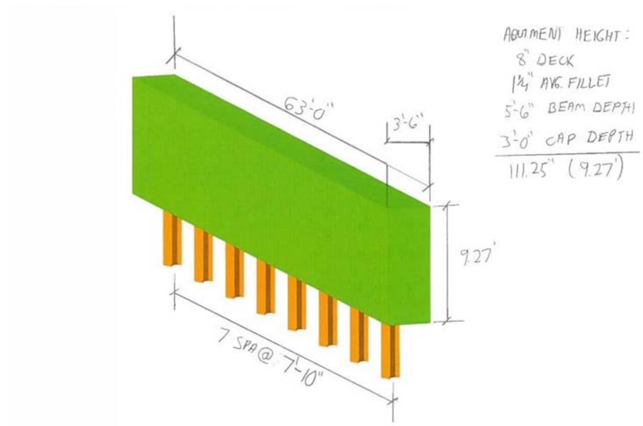
Hammer Head Pier: Example 1

Abutment Isometric View



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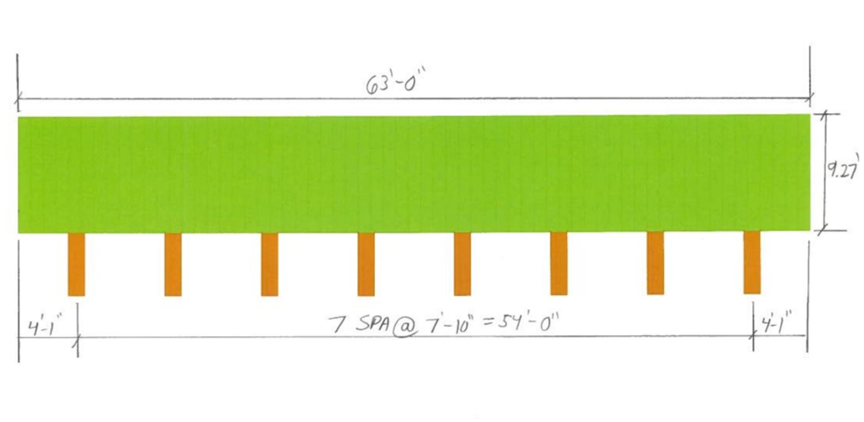
Hammer Head Pier: Example 1
Abutment Plan View



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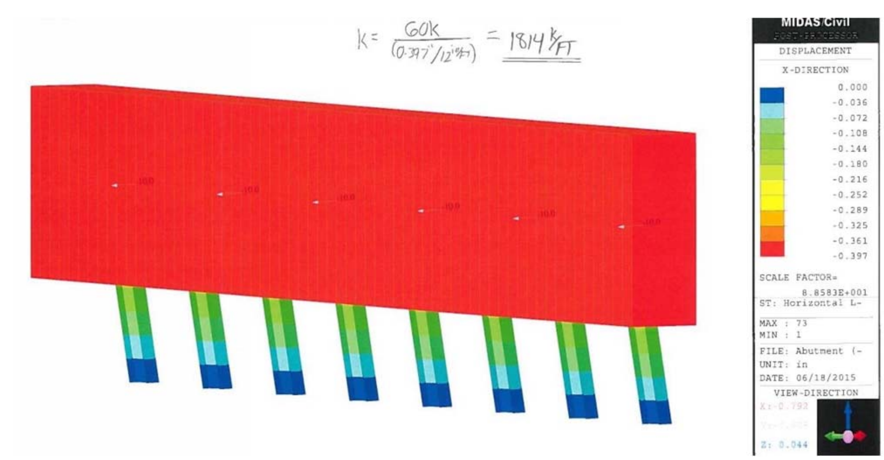
Hammer Head Pier: Example 1
Abutment Stiffness



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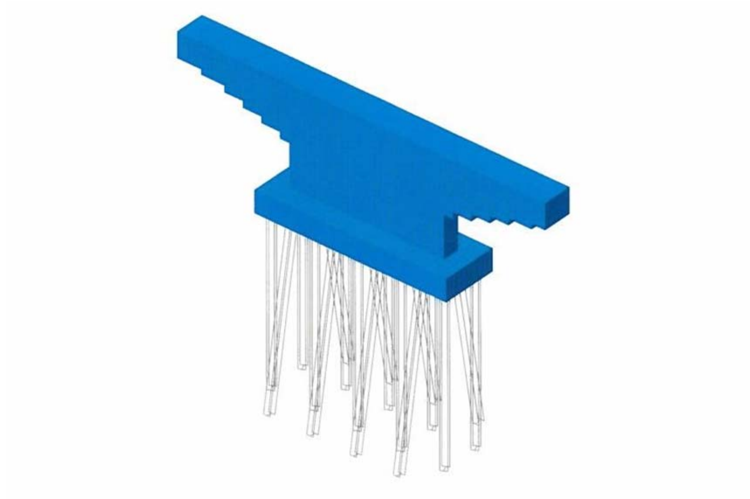
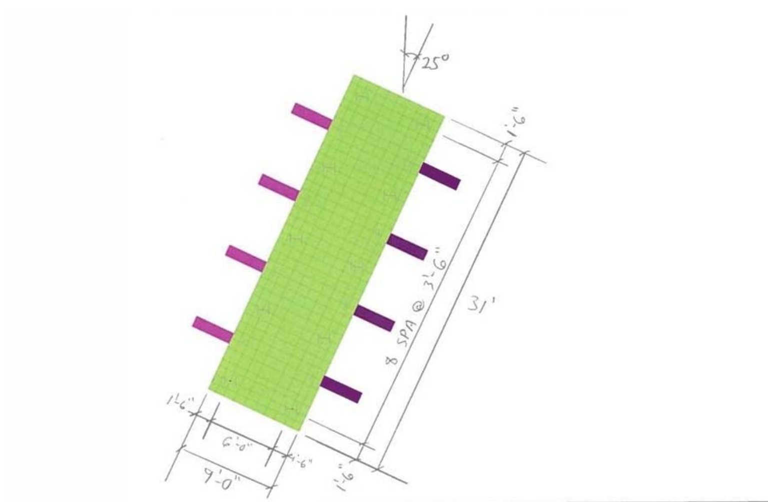
Hammer Head Pier: Example 1

Pier Isometric View



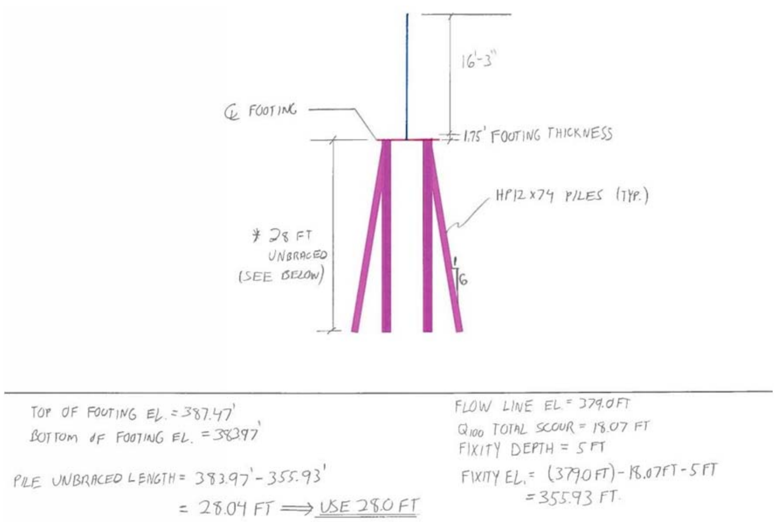
Hammer Head Pier: Example 1

Pier Plan View



Hammer Head Pier: Example 1

Pier Elevation View



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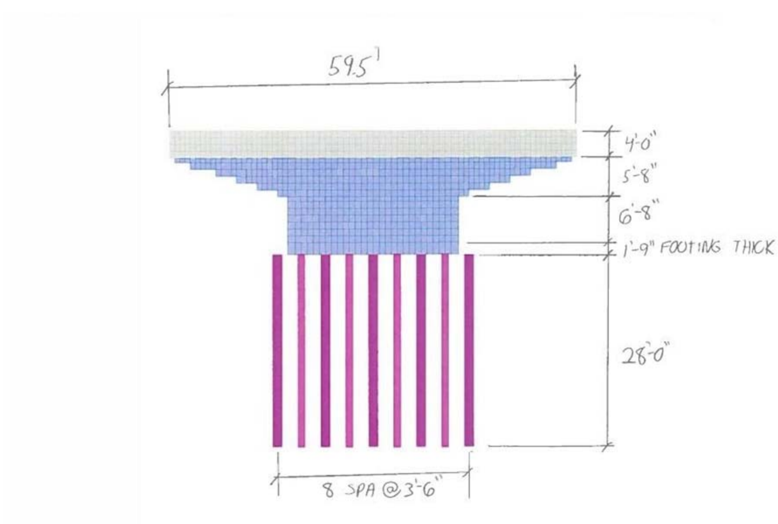
Hammer Head Pier: Example 1

Pier Elevation View



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Hammer Head Pier: Example 1

Pier Stiffness Calculation

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Hammer Head Pier: Example 1

Pier Stiffness Calculation

BEARING STIFFNESS:

$$K_{BRG} = \frac{A \times G}{t}$$

$A = 36" \times 12" = 432 in^2$

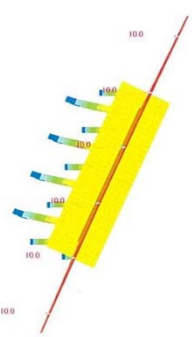
$G = 110 psi$

$t_{ELASTOMER} = (5 \times 1/2") + (2 \times 9/32") = 3.0625"$

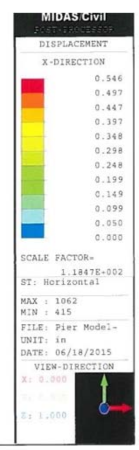
$$K_{BRG} = \frac{(432 in^2)(110 psi) \times 1k/1000 lb}{(3.0625"/12 in/ft)} = 186.2 \ k/ft$$

12 BEARINGS TOTAL:

$12 \times 186.2 \ k/ft = 2234.4 \ k/ft$



$$K_{PIER} = \frac{60 k}{(0.546 in/12 in/ft)} = 1319 \ k/ft$$



TOTAL STIFFNESS:

$$\frac{1}{K_{TOTAL}} = \frac{1}{K_{PIER}} + \frac{1}{K_{BRG}} = \frac{1}{1319 \ k/ft} + \frac{1}{2234.4 \ k/ft} \Longrightarrow \boxed{K_{TOTAL} = 829.4 \ k/ft}$$

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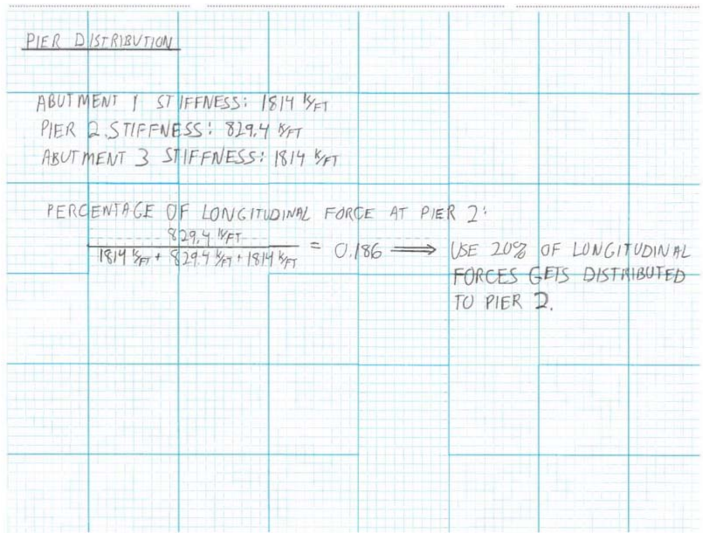
Hammer Head Pier: Example 1

Pier Stiffness Calculation



PIER DISTRIBUTION

ABUTMENT 1 STIFFNESS: 1814 K/FT
PIER 2 STIFFNESS: 829.4 K/FT
ABUTMENT 3 STIFFNESS: 1814 K/FT

PERCENTAGE OF LONGITUDINAL FORCE AT PIER 2:

$$\frac{829.4 \text{ K/FT}}{1814 \text{ K/FT} + 829.4 \text{ K/FT} + 1814 \text{ K/FT}} = 0.186 \Longrightarrow$$

USE 20% OF LONGITUDINAL FORCES GETS DISTRIBUTED TO PIER 2.

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Hammer Head Pier: Example 1

Controlling Load Case

- Strength Cases I – V were Checked
- Strength Case I controlled
- 1.25DC + 1.5DW + 1.75LL + 1.75BR

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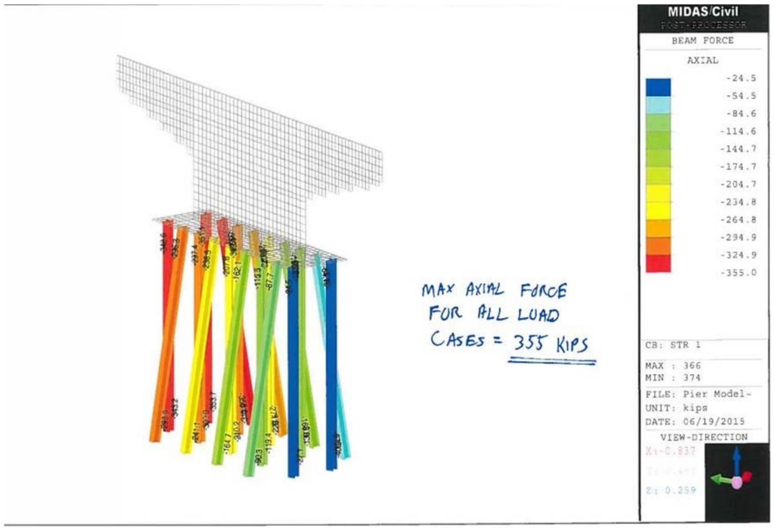
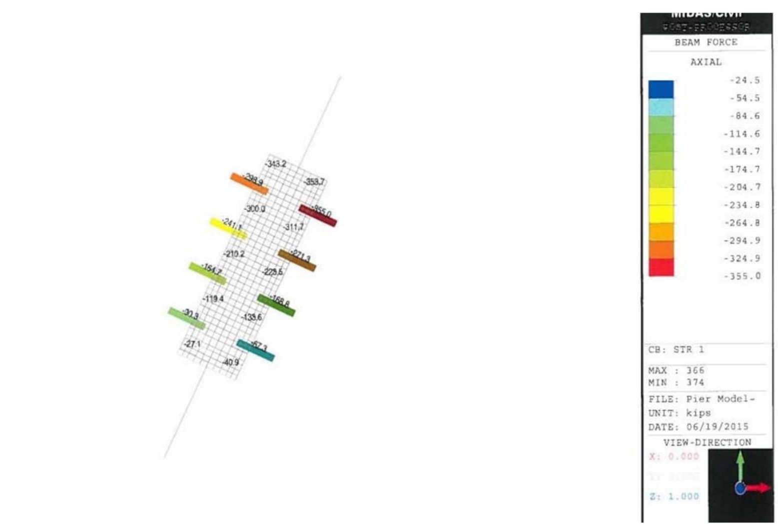
Hammer Head Pier: Example 1

Strength I: Max Axial Force = 355kips



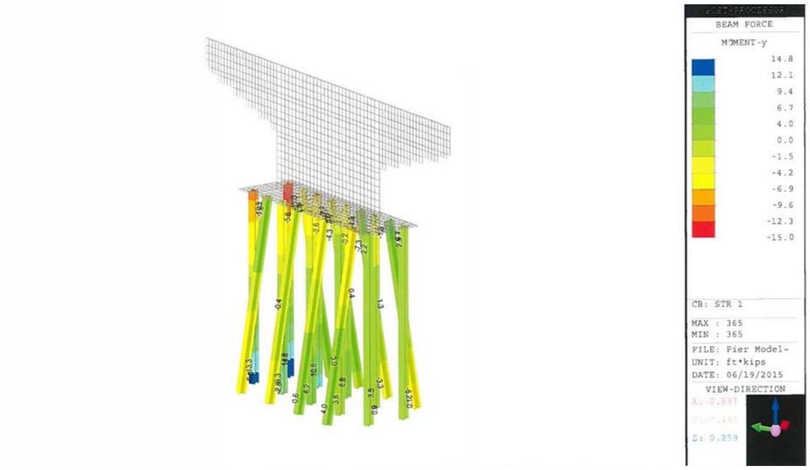
Hammer Head Pier: Example 1

Strength I: Max Axial Force = 355kips



Hammer Head Pier: Example 1

STR I: Max Strong-Axis Moment = 15.0ft-k



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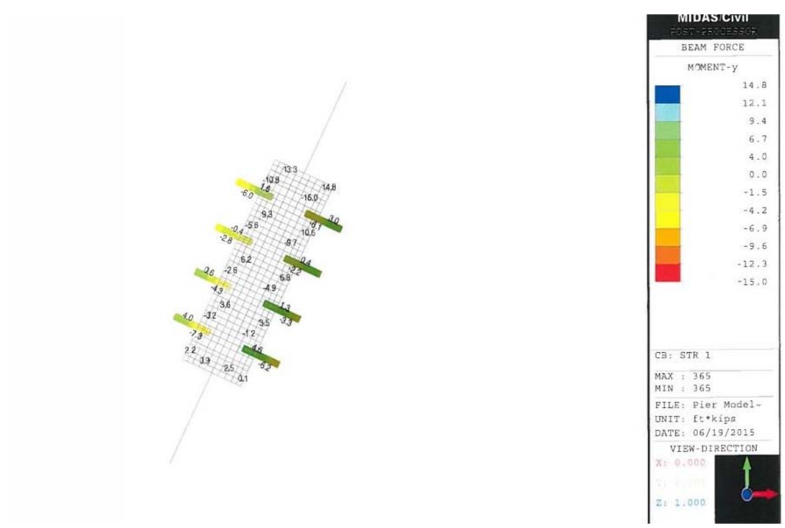
Hammer Head Pier: Example 1

STR I: Max Strong-Axis Moment = 15.0ft-k



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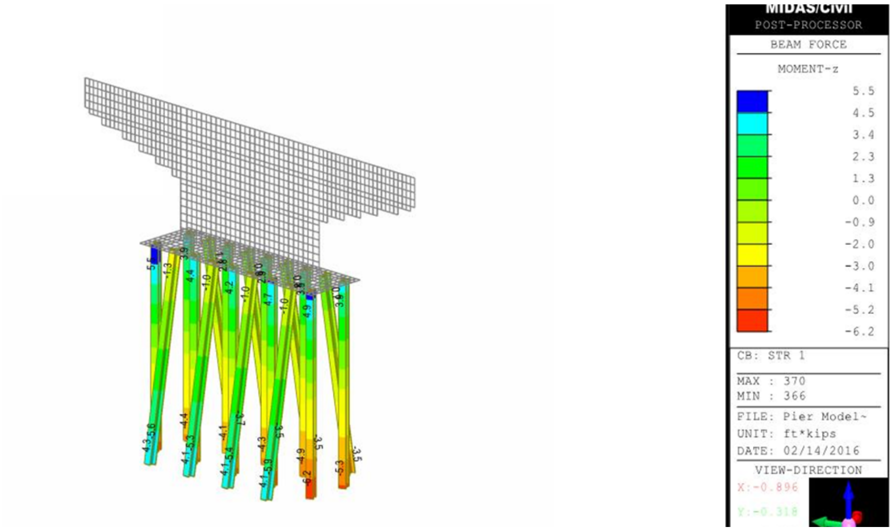
Hammer Head Pier: Example 1

STR I: Max Weak-Axis Moment = 4.5ft-k



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Hammer Head Pier: Example 1

HP12x74 Section Properties

Area = 21.8 in^2

Web thick = 0.605 in

Flange thick = 0.610 in

Depth = 12.1 in

Flange Width = 12.2 in

I_x = 569.0 in^4

S_x = 93.8 in^3

r_x = 5.110 in

Z_x = 105.0 in^3

I_y = 186.0 in^4

S_y = 30.4 in^3

r_y = 2.920 in

Z_y = 46.6 in^3

r_t = 3.260 in

J = 2.98 in^4

C_w = 6170 in^6

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Hammer Head Pier: Example 1

Calculate Slenderness Ratio

- Slenderness Ratio:

$Kl/r \leq 120$ (Section 6.9.3)

0.85(28ft x 12 in/ft) / 2.92in = 97.8

Hammer Head Pier: Example 1

Calculate Pu / Pr

- Pu = 200.1 kips
- To Calculate P_r, Find P_e/P_o

$P_e = \frac{(\pi^2)E}{\left(\frac{Kl}{r_s}\right)^2} * Ag$ (Eqn. 6.9.4.1.2-1)

$P_e = \frac{(\pi^2)29000ksi}{(97.8in)^2} * 21.8in^2$ = 652.3k

$P_o = QF_yA_g$

P_o = 1.0(50ksi)(21.8in^2) = 1090k

P_e/P_o = 652.3k / 1090.0k = 0.60

Hammer Head Pier: Example 1
Calculate Pu / Pr

Since $P_e/P_o \geq 0.44$, then

$$Pn = \left[0.658^{\frac{P_o}{P_e}}\right] * Po \qquad \text{(Eqn. 6.9.4.1.1-1)}$$

$$Pn = \left[0.658^{\frac{1090k}{652.3k}}\right] * 1090k = 541.6k$$

P_r = Factored Compressive Resistance (Article 6.9.2.1)

$P_r = \phi_c P_n = 0.9(541.6k) = 487.4k$

$P_u/P_r = (355.0k / 487.4) = 0.728$

Since $P_u/P_r \geq 0.2$, Use Eqn. (6.9.2.2-2)

STRUCTUREPOINT

Hammer Head Pier: Example 1
Combined Axial & Flexural Eqn.

Eqn. 6.9.2.2-1

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

$P_u = 355.0k$

$P_r = 487.4k$

$M_{ux} = 15.0$ ft-k

$M_{uy} = 4.5$ ft-k

Calculate Flexural Capacities M_{rx} & M_{ry}

STRUCTUREPOINT

Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

Calculate Section Ratios: λ_f, λ_{pf} and λ_{rf}

$$\lambda_f = \frac{b_{fc}}{2tf_c} = \frac{12.2in}{2(0.610in)} = 10.0in$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}} = 0.38\sqrt{\frac{29000ksi}{50ksi}} = 9.15in$$

$$\lambda_{rf} = 0.95\sqrt{\frac{Ek_c}{F_{yr}}} = 0.95\sqrt{\frac{29000ksi(0.76)}{50ksi}} = 19.9in$$

STRUCTUREPOINT

Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

Since $\lambda_f > \lambda_{pf}$, then:

$$M_{nc} = [1 - (1 - \frac{F_{yr}S_{xc}}{R_{pc}M_{yc}})(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}})]R_{pc}M_{yc} \text{ (Eqn. A6.3.2-2)}$$

R_{pc} = 105in^3/93.8in^3 = 1.12
M_{yc} = 50ksi(93.8in^3) = 4690 in-k

F_{yr}: taken as the smaller of:

- $0.7F_{yc}$ = 35ksi
- $R_h F_{yt} * (\frac{S_{xt}}{S_{xc}})$ = 50ksi
- F_{yw} = 50ksi
- (But Not Less than $0.5F_{yc}$)

STRUCTUREPOINT

Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

$$M_{nc} = [1 - (1 - \frac{(35ksi)(93.8in^3)}{(1.12)(4690in-k)})(\frac{10.0" - 9.15"}{19.9" - 9.15"})](1.12x4690\text{in-k})$$

$$M_{nc} = 5097.0\text{in-k} = 424.8\text{ft-k}$$

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Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

LTB

Calculate Plastic Length Limit (L_p) and Elastic Length Limit (L_r): Section A6.3.3

$$L_b = 28ft$$

$$L_p = 1.0rt\sqrt{\frac{E}{F_{yc}}}$$ (Eqn. A6.3.3-4)

$$L_p = 1.0(3.26in)\sqrt{\frac{29000ksi}{50ksi}} = 78.5in = 6.5ft$$

AMERICAN STRUCTUREPOINT INC.

Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

LTB

Calculate Plastic Length Limit (L_p) and Elastic Length Limit (L_r): Section A6.3.3

$$L_r = 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1 + \sqrt{1 + 6.76\left(\frac{F_{yr}}{E} * \frac{S_{xc} h}{J}\right)^2}} \quad \text{(Eqn. A6.3.3-5)}$$

$$L_r = 1.95(3.260in)\frac{29000ksi}{35ksi}\sqrt{\frac{2.98in^4}{93.8in^3(10.88in)}}\sqrt{1 + \sqrt{1 + 6.76\left(\frac{35ksi}{29000ksi} * \frac{93.8in^3(10.88in)}{2.98in^4}\right)^2}}$$

$$L_r = 447.1in = 37.3\text{ft}$$

STRUCTUREPOINT

Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

LTB

Since $L_p < L_b \leq L_r$,

$$M_{nc} = Cb\left[1 - \left(1 - \frac{F_{yr} S_{xc}}{R_{pc} M_{yc}}\right)\left(\frac{L_b - Lp}{L_r - Lp}\right)\right] R_{pc} M_{yc} \leq Rp_{cMyc} \quad \text{(Eqn. A6.3.3-2)}$$

$$M_{nc} = 1.0\left[1 - \left(1 - \frac{35ksi(93.8in^3)}{1.12(4690in-k)}\right)\left(\frac{28ft - 6.5ft}{37.3ft - 6.5ft}\right)\right] 1.12(4690in-k)$$

$$M_{nc} = 3877.8in - k = 323.2ft - k$$

STRUCTUREPOINT

Hammer Head Pier: Example 1

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

M_{nc} = 5097.0in−k = 424.8ft−k

LTB

M_{nc} = 3877.8in−k = 323.2ft−k

M_{nc} for LTB Controls

$M_{rx} = \phi_f M_{nc}$ = 0.9(323.2ft-k) = 290.0ft-k

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Hammer Head Pier: Example 1

Calculate Weak-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB

FLB

Calculate Section Ratios: λ_f, λ_{pf} and λ_{rf}

$$\lambda_f = \frac{b_{fc}}{2tf_c} = \frac{12.2in}{2(0.610in)} = 10.0\text{in}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}} = 0.38\sqrt{\frac{29000ksi}{50ksi}} = 9.15\text{in}$$

$$\lambda_{rf} = 0.83\sqrt{\frac{E}{F_{yf}}} = 0.83\sqrt{\frac{29000ksi}{50ksi}} = 20.0\text{in}$$

AMERICAN STRUCTUREPOINT INC.

Hammer Head Pier: Example 1

Calculate Weak-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB

FLB

Since $\lambda_f > \lambda_{pf}$, then:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_{yf}}}}\right)\right] F_{yf} Z_y \qquad \text{(Eqn. 6.12.2.2.1-2)}$$

$$M_n = \left[1 - \left(1 - \frac{30.4in^3}{46.6in^3}\right)\left(\frac{10.0\text{in} - 9.15\text{in}}{0.45\sqrt{\frac{29000ksi}{50ksi}}}\right)\right] 50ksi(46.6in^3)$$

$$M_n = 2266.5in - k = 188.9ft - k$$

$$\mathbf{M_{ry} = \phi_f M_n = 0.9(188.9ft\text{-}k) = 170.0ft\text{-}k}$$

Hammer Head Pier: Example 1

Combined Axial & Flexural Eqn.

Eqn. 6.9.2.2-2

$$\frac{P_u}{P_r} + \frac{8.0}{9.0}\left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

P_u = 355.0k

P_r = 487.4k

M_{ux} = 15.0 ft-k

M_{uy} = 4.5 ft-k

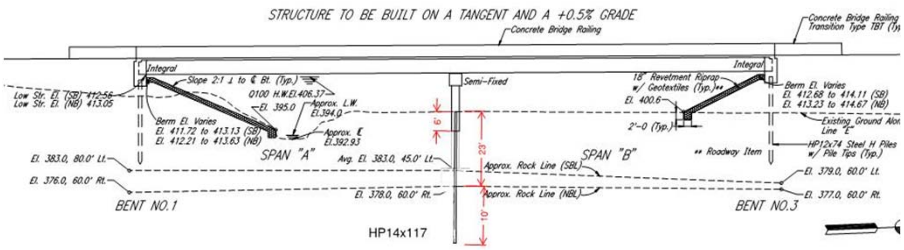
M_{rx} = 290.0 ft-k

M_{ry} = 170.0 ft-k

$$\frac{\mathbf{355.0k}}{\mathbf{487.4k}} + \frac{8.0}{9.0}\left(\frac{\mathbf{15.0ft-k}}{\mathbf{290.0ft-k}} + \frac{\mathbf{4.5ft-k}}{\mathbf{170.0ft-k}}\right) = \mathbf{0.798}$$

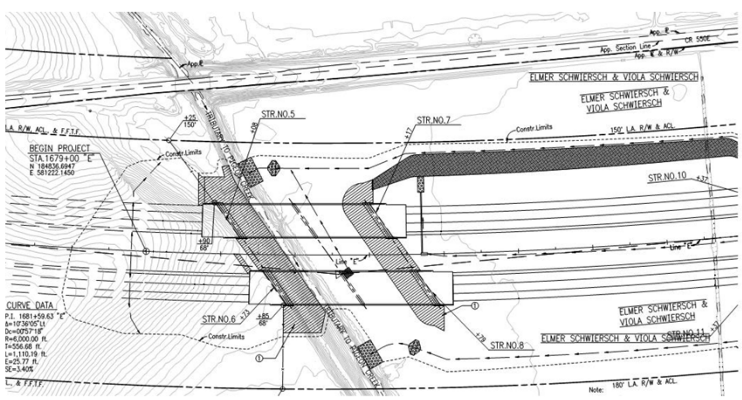
Structural Analysis of Unbraced Piles

Example 2: Two Span Structure (100ft Ea)



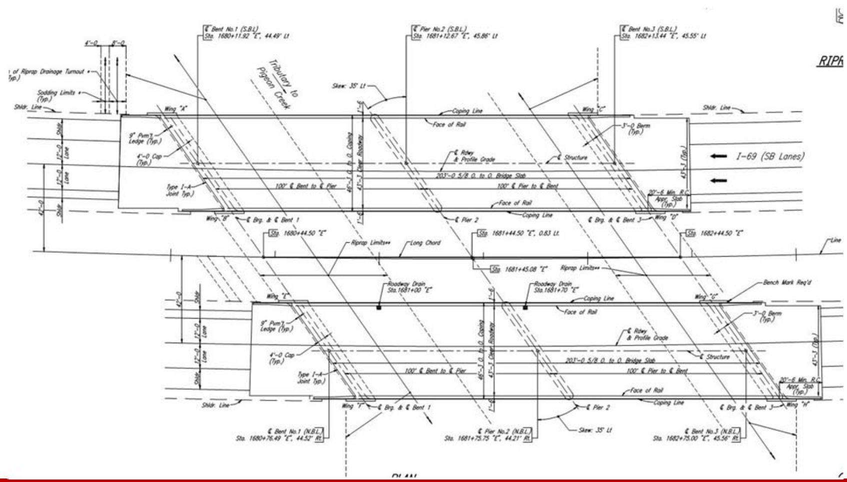
Structural Analysis of Unbraced Piles

Plan View



Structural Analysis of Unbraced Piles

Plan View



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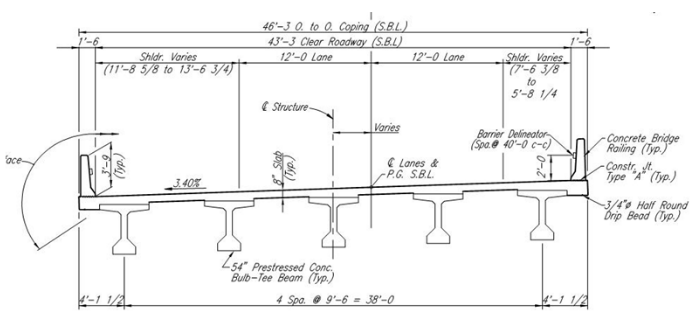
Structural Analysis of Unbraced Piles

Cross-Section View



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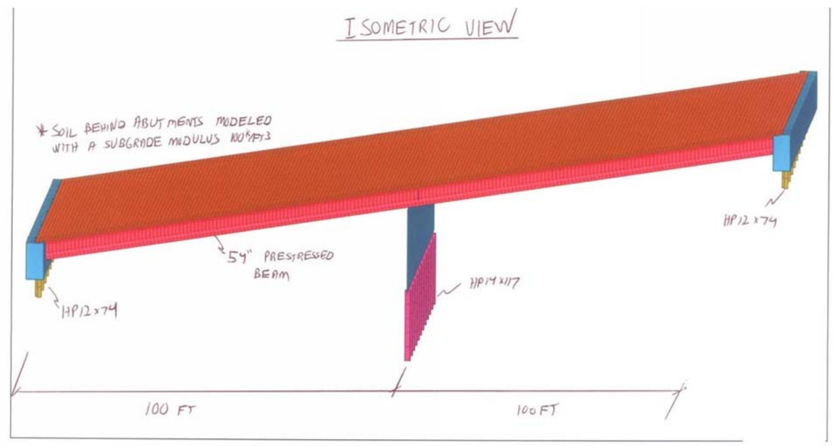
Single of Row Piles: Example 2



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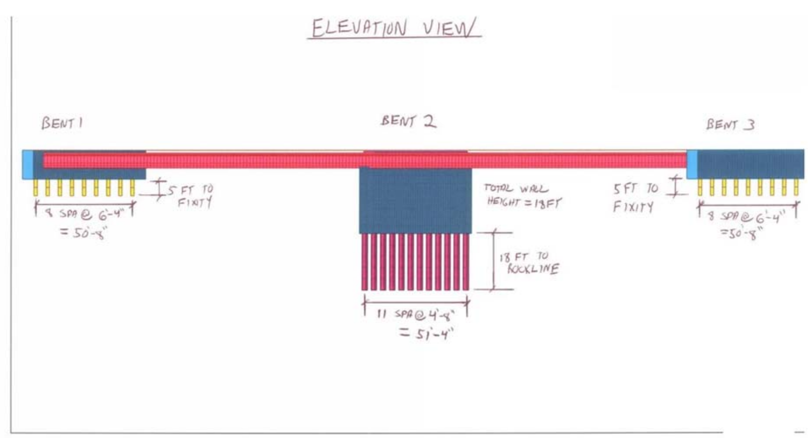
Single of Row Piles: Example 2



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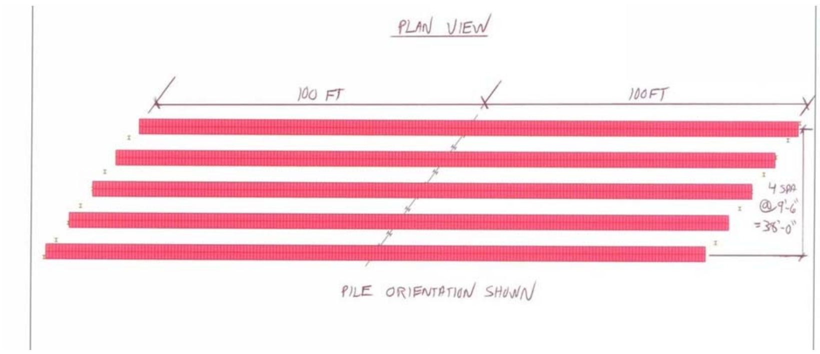
Single of Row Piles: Example 2



STRUCTUREPOINT

Single of Row Piles: Example 2
Find Seismic Loads

- Calculate Seismic Forces
- Used Single Mode Spectral Method

STRUCTUREPOINT

Single of Row Piles: Example 2

Find Seismic Loads

SEISMIC DEAD LOADS

ABUTMENT: (FROM CONSPAN)

EXTERIOR BEAM: $F_v = 110.9k \times 2$ BEAMS = 221.8 KIPS

INTERIOR BEAM: $F_v = 108.9k \times 3$ BEAMS = 326.7 KIPS

END BENT DIAPHRAGM: $\frac{1}{2}(4.0')(9.0')(59.104') \times 0.150\ k/ft^3$ = 159.6 KIPS

TOTAL = 708.1 KIPS

INTERIOR PIER: (FROM CONSPAN)

EXTERIOR BEAM: $F_v = 231.8k \times 2$ BEAMS = 463.6 KIPS

INTERIOR BEAM: $F_v = 227.8k \times 3$ BEAMS = 683.4 KIPS

DIAPHRAGM: $[(46.4')(4.5') - 4(6.31\ ft^2)] \times (4.0') \times 0.150\ k/ft^3$ = 110.1 KIPS (6.31 FT² = AREA OF BEAM)

TOTAL = 1257.1 KIPS

TOTAL LOAD FROM SUPER: 2 (708.1 KIPS) + 1257.1 KIPS = <u>2673 KIPS</u>

SUBSTRUCTURE:

CAP WT = 123.1 KIPS

STEM WT = 326.9 KIPS

USE 50% TOTAL = $\frac{1}{2}$ (123.1 + 326.9) KIPS = <u>225 KIPS</u>

* TOTAL SEISMIC LOAD = 2673 KIPS + 225 KIPS = <u>2898 KIPS</u>

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Single of Row Piles: Example 2

Find Seismic Loads

SINGLE MODE SPECTRAL METHOD

TOTAL BRIDGE DL FROM SUPERSTRUCTURE: 2898K $\quad \frac{2898K}{200FT} = 14.49\ k/FT$

DEFLECTION OF DECK, $V_s(x)$ = 0.0024 FT (FROM MIDAS MODEL)

$$\alpha = \int V_s(x)\,dx = 0.00240\ FT \times 200\ FT = 0.48$$

$$\beta = \int w(x) V_s(x)\,dx = 14.49\ k/FT \times 0.48 = 6.96$$

$$\gamma = \int w(x) V_s^2(x)\,dx = 6.96 \times 0.00240 = 0.0167$$

CALCULATE PERIOD:

$$T_m = 2\pi\sqrt{\frac{\gamma}{p_o g \alpha}} = 2\pi\sqrt{\frac{0.0167}{1.0 \times 32.2 \times 0.48}} = \underline{0.207}$$

CALCULATE T_S:

$$T_S = \frac{S_{D1}}{S_{DS}} = \frac{0.24}{0.532} = 0.451\ SEC$$

CALCULATE T_0:

$$T_0 = 0.2 T_S = 0.2(0.451\ SEC) = 0.090\ SEC$$

SINCE: $T_0 < T_m < T_S$

USE $C_{sm} = S_{DS} = \underline{0.532}$

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Single of Row Piles: Example 2
Find Seismic Loads

SINGLE MODE SPECTRAL METHOD

CALCULATE EQUIVALENT STATIC LOAD:

$$p_e(x) = \frac{\beta C_{sm}}{\gamma} w(x) v_s(x)$$

$$= \frac{6.96 \times 0.532}{0.0167} (14.49 \text{ k/FT}) (0.0024 \text{ FT})$$

$$= \underline{7.71 \text{ KLF}}$$

(TRANSVERSE LOAD YIELDS SAME SEISMIC LOAD SINCE $C_{sm} = S_{DS}$)

TOTAL HOR. LOAD = 7.71 KLF (200 FT) = 1542 k

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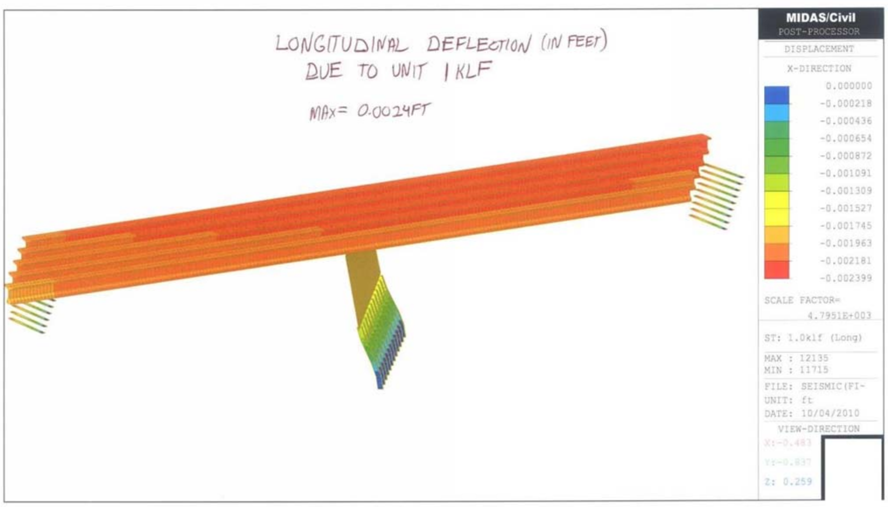
Single of Row Piles: Example 2
Find Seismic Loads



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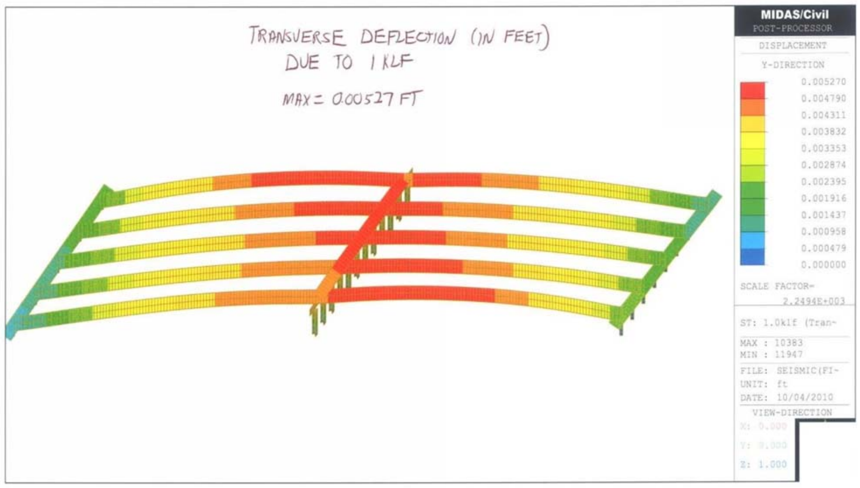
Single of Row Piles: Example 2
Find Seismic Loads



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Single of Row Piles: Example 2
Controlling Load Case

- Both Seismic Load Cases were checked
- Transverse Seismic Load Case controlled
- 1.25DC + 1.5DW + 0.3EQ + 1.0EQ

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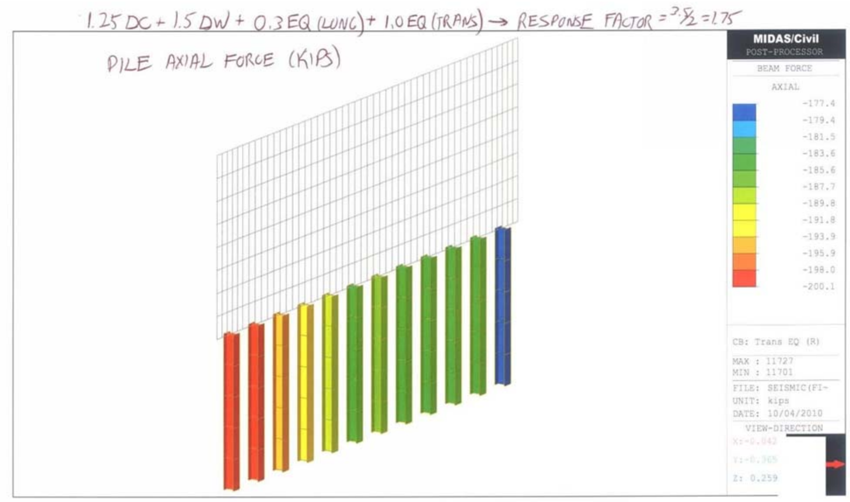
Single of Row Piles: Example 2

(Max Axial Force = 200.1 kips)



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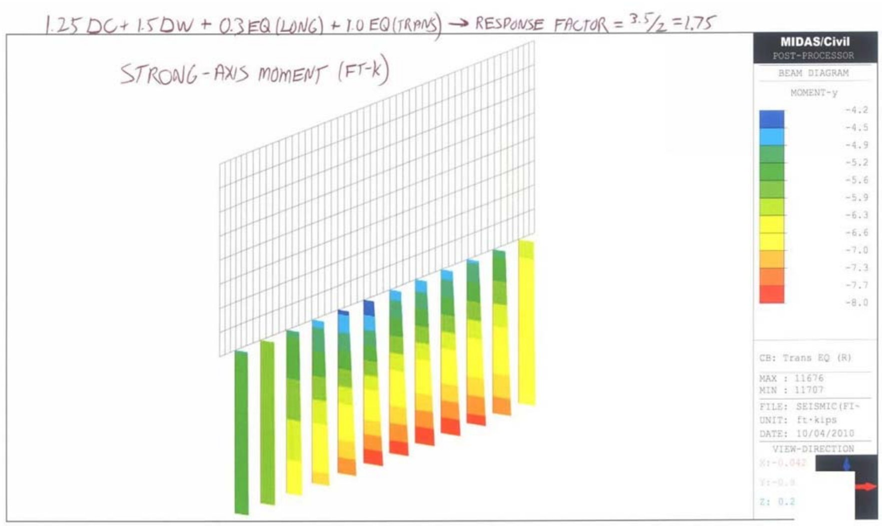
Single of Row Piles: Example 2

Strong Axis Moment = 5.6ft-k



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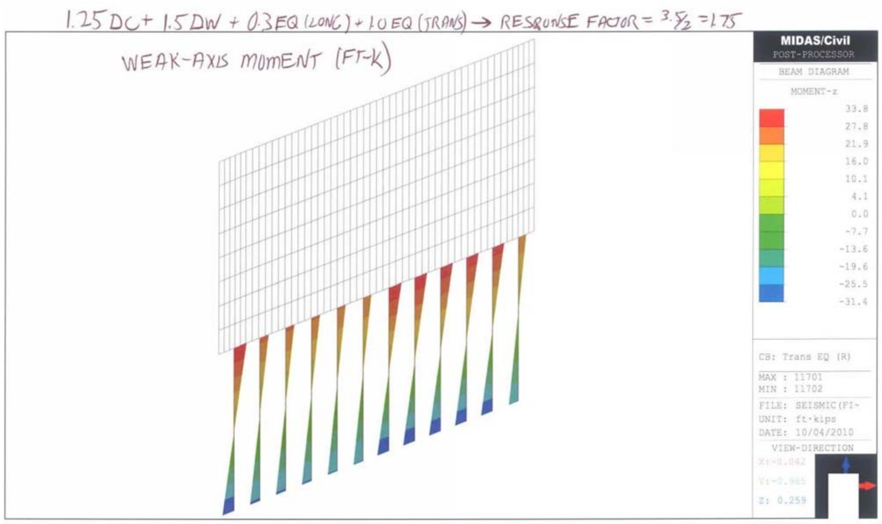
Single of Row Piles: Example 2

Weak Axis Moment = 33.8ft-k



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Single of Row Piles: Example 2

HP14x117 Section Properties

Area = 34.4 in^2
Web thick = 0.805 in
Flange thick = 0.805 in

Depth = 14.2 in
Flange Width = 14.9 in

I_x = 1220.0 in^4
S_x = 172.0 in^3
r_x = 5.960 in
Z_x = 194.0 in^3

I_y = 443.0 in^4
S_y = 59.5 in^3
r_y = 3.590 in
Z_y = 91.4 in^3

r_t = 4.000 in
J = 8.02 in^4
C_w = 19900 in^6

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Single of Row Piles: Example 2

Calculate Slenderness Ratio

- Slenderness Ratio:

$Kl/r \leq 120$ (Section 6.9.3)

1.2(17ft x $12^{in}/_{ft}$) / 3.59in = 68.2

Single of Row Piles: Example 2

Calculate Pu / Pr

- Pu = 200.1 kips
- To Calculate P_r, Find P_e/P_o

$$P_e = \frac{(\pi^2)E}{\left(\frac{Kl}{r_s}\right)^2} * Ag$$ (Eqn. 6.9.4.1.2-1)

$$P_e = \frac{(\pi^2)29000ksi}{(68.2in)^2} * 34.4in^2$$ = 2116.8k

$P_o = QF_yA_g$

P_o = 1.0(50ksi)(34.4in²) = 1720.0k

P_e/P_o = 2118.8k / 1720.0k = 1.23

Single of Row Piles: Example 2
Calculate Pu / Pr

Since $P_e/P_o \geq 0.44$, then

$$Pn = \left[0.658^{\frac{P_o}{P_e}}\right] * Po \qquad \text{(Eqn. 6.9.4.1.1-1)}$$

$$Pn = \left[0.658^{\frac{1720k}{2116.8k}}\right] * 1720k = 1224.1k$$

P_r = Factored Compressive Resistance (Article 6.9.2.1)

$P_r = \phi_c P_n = 1.0(1224.1k) = 1224.1k$

$P_u/P_r = (200.1k / 1224.1k) = 0.163$

Since $P_u/P_r < 0.2$, Use Eqn. (6.9.2.2-1)

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Single of Row Piles: Example 2
Combined Axial & Flexural Eqn.

Eqn. 6.9.2.2-1

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

$P_u = 200.1k$

$P_r = 1224.1k$

$M_{ux} = 5.6$ ft-k

$M_{uy} = 33.8$ ft-k

Calculate Flexural Capacities M_{rx} & M_{ry}

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Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

Calculate Section Ratios: λ_f, λ_{pf} and λ_{rf}

$$\lambda_f = \frac{b_{fc}}{2tf_c} = \frac{14.9in}{2(0.805in)} = 9.25\text{in}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}} = 0.38\sqrt{\frac{29000ksi}{50ksi}} = 9.15\text{in}$$

$$\lambda_{rf} = 0.95\sqrt{\frac{Ek_c}{F_{yr}}} = 0.95\sqrt{\frac{29000ksi(0.76)}{50ksi}} = 19.9\text{in}$$

Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

Since $\lambda_f > \lambda_{pf}$, then:

$$M_{nc} = [1 - (1 - \frac{F_{yr}S_{xc}}{R_{pc}M_{yc}})(\frac{\lambda_f - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}})]R_{pc}M_{yc} \text{ (Eqn. A6.3.2-2)}$$

$R_{pc} = 194in^3/172in^3 = 1.13$
$M_{yc} = 50ksi(172in^3) = 8600$ in-k

F_{yr}: taken as the smaller of:

- $0.7F_{yc} = 35$ksi
- $R_hF_{yt} * (\frac{S_{xt}}{S_{xc}}) = 50$ksi
- $F_{yw} = 50$ksi
- (But Not Less than $0.5F_{yc}$)

Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

$M_{nc} = [1-(1-\frac{(35ksi)(172in^3)}{(1.13)(8600in-k)})(\frac{9.25"-9.15"}{19.9"-9.15"})](1.13x8600in\text{-}k)$

$M_{nc} =$ 9683.6in-k = 807.0ft-k

Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

LTB

Calculate Plastic Length Limit (L_p) and Elastic Length Limit (L_r): Section A6.3.3

$L_b = 17ft$

$L_p = 1.0rt\sqrt{\frac{E}{F_{yc}}}$ (Eqn. A6.3.3-4)

$L_p = 1.0(4.000in)\sqrt{\frac{29000ksi}{50ksi}} = 96.3in = 8.0ft$

Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

LTB

Calculate Plastic Length Limit (L_p) and Elastic Length Limit (L_r): Section A6.3.3

$$L_r = 1.95 r_t \frac{E}{F_{yr}} \sqrt{\frac{J}{S_{xc} h}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_{yr}}{E} * \frac{S_{xc} h}{J} \right)^2}}$$ (Eqn. A6.3.3-5)

$$L_r = 1.95(4.000in) \frac{29000ksi}{35ksi} \sqrt{\frac{8.02in^4}{172in^3(12.59in)}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{35ksi}{29000ksi} * \frac{172in^3(12.59in)}{8.02in^4} \right)^2}}$$

$$L_r = 597.0in = 49.8\text{ft}$$

STRUCTUREPOINT

Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

LTB

Since $L_p < L_b \leq L_r$,

$$M_{nc} = Cb \left[1 - \left(1 - \frac{F_{yr} S_{xc}}{R_{pc} M_{yc}} \right) \left(\frac{L_b - Lp}{L_r - Lp} \right) \right] R_{pc} M_{yc} \leq Rp_{cMyc}$$ (Eqn. A6.3.3-2)

$$M_{nc} = 1.0 \left[1 - \left(1 - \frac{35ksi(172in^3)}{1.13(8600in-k)} \right) \left(\frac{17ft - 8.0ft}{49.8ft - 8.0ft} \right) \right] 1.13(8600in-k)$$

$$M_{nc} = 8921.8in - k = 743.5ft - k$$

STRUCTUREPOINT

Single of Row Piles: Example 2

Calculate Strong-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB & LTB

FLB

M_{nc} = 9683.6in−k = 807.0ft−k

LTB

M_{nc} = 8921.8in−k = 743.5ft−k

M_{nc} for LTB Controls

$M_{rx} = \phi_f M_{nc}$ = 1.0(743.5ft-k) = 743.5ft-k

Single of Row Piles: Example 2

Calculate Weak-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB

FLB

Calculate Section Ratios: λ_f, λ_{pf} and λ_{rf}

$$\lambda_f = \frac{b_{fc}}{2tf_c} = \frac{14.9in}{2(0.805in)} = 9.25\text{in}$$

$$\lambda_{pf} = 0.38\sqrt{\frac{E}{F_{yf}}} = 0.38\sqrt{\frac{29000ksi}{50ksi}} = 9.15\text{in}$$

$$\lambda_{rf} = 0.83\sqrt{\frac{E}{F_{yf}}} = 0.83\sqrt{\frac{29000ksi}{50ksi}} = 20.0\text{in}$$

Single of Row Piles: Example 2

Calculate Weak-Axis Flexural Capacity

Calculate Flexural Resistance based on FLB

FLB

Since $\lambda_f > \lambda_{pf}$, then:

$$M_n = \left[1 - \left(1 - \frac{S_y}{Z_y}\right)\left(\frac{\lambda_f - \lambda_{pf}}{0.45\sqrt{\frac{E}{F_{yf}}}}\right)\right] F_{yf} Z_y \quad \text{(Eqn. 6.12.2.2.1-2)}$$

$$M_n = \left[1 - \left(1 - \frac{59.5in^3}{91.4in^3}\right)\left(\frac{9.25\text{in} - 9.15\text{in}}{0.45\sqrt{\frac{29000ksi}{50ksi}}}\right)\right] 50ksi(91.4in^3)$$

$$M_n = 4555.3in - k = 379.6ft - k$$

$M_{ry} = \phi_f M_n = 1.0(379.6\text{ft-k}) = 379.6\text{ft-k}$

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Single of Row Piles: Example 2

Combined Axial & Flexural Eqn.

Eqn. 6.9.2.2-1

$$\frac{P_u}{2Pr} + \left(\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}\right) \leq 1.0$$

P_u = 200.1k
P_r = 1224.1k
M_{ux} = 5.6 ft-k
M_{uy} = 33.8 ft-k
M_{rx} = 743.5 ft-k
M_{ry} = 379.6 ft-k

$$\frac{200.1k}{2(1224.1k)} + \left(\frac{5.6ft-k}{743.5ft-k} + \frac{33.8ft-k}{379.6ft-k}\right) = \mathbf{0.178}$$

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Structural Analysis of Unbraced Piles

QUESTIONS?

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