F. W. Klaiber, D. J. White, T. J. Wipf, B. M. Phares, V. W. Robbins

Development of Abutment Design Standards for Local Bridge Designs
Volume 3 of 3

Verification of Design Methodology

August 2004

Sponsored by the
Iowa Department of Transportation
Highway Division and the
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Iowa DOT Project TR - 486

Final

REPORT

IOWA STATE UNIVERSITY
OF SCIENCE AND TECHNOLOGY

Department of Civil and Construction Engineering

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ABSTRACT

Several superstructure design methodologies have been developed for low volume road bridges by the Iowa State University Bridge Engineering Center. However, to date no standard abutment designs have been developed. Thus, there was a need to establish an easy to use design methodology in addition to generating generic abutment standards and other design aids for the more common substructure systems used in Iowa.

The final report for this project consists of three volumes. The first volume summarizes the research completed in this project. A survey of the Iowa County Engineers was conducted from which it was determined that while most counties use similar types of abutments, only 17 percent use some type of standard abutment designs or plans. A literature review revealed several possible alternative abutment systems for future use on low volume road bridges in addition to two separate substructure lateral load analysis methods. These consisted of a linear and a non-linear method. The linear analysis method was used for this project due to its relative simplicity and the relative accuracy of the maximum pile moment when compared to values obtained from the more complex non-linear analysis method. The resulting design methodology was developed for single span stub abutments supported on steel or timber piles with a bridge span length ranging from 20 to 90 ft and roadway widths of 24 and 30 ft. However, other roadway widths can be designed using the foundation design template provided. The backwall height is limited to a range of 6 to 12 ft, and the soil type is classified as cohesive or cohesionless. The design methodology was developed using the guidelines specified by the American Association of State Highway Transportation Officials Standard Specifications, the Iowa Department of Transportation Bridge Design Manual, and the National Design Specifications for Wood Construction.

The second volume introduces and outlines the use of the various design aids developed for this project. Charts for determining dead and live gravity loads based on the roadway width, span length, and superstructure type are provided. A foundation design template was developed in which the engineer can check a substructure design by inputting basic bridge site information. Tables published by the Iowa Department of Transportation that provide values for estimating pile friction and end bearing for different combinations of soils and pile types are also included. Generic standard abutment plans were developed for which the engineer can provide necessary bridge site information in the spaces provided. These tools enable engineers to design and detail county bridge substructures more efficiently.

The third volume (this volume) provides two sets of calculations that demonstrate the application of the substructure design methodology developed in this project. These calculations also verify the accuracy of the foundation design template. The printouts from the foundation design template are provided at the end of each example. Also several tables provide various foundation details for a pre-cast double tee superstructure with different combinations of soil type, backwall height, and pile type.

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DESIGN VERIFICATION EXAMPLES

This document provides two sets of calculations that demonstrate the application of the substructure design methodology developed for the Iowa Department of Transportation (Iowa DOT) in Project TR-486. These calculations also verify the accuracy of the foundation design template (FDT) developed for Project TR-486. The printouts from the FDT are provided in this document at the end of each example. Also Tables 1, 2 and 3 present various foundation details for a pre-cast double tee superstructure (PCDT) with different combinations of soil type, backwall height, and pile type. It should be noted that the foundation details given in these tables are for a fictitious bridge site. Also, the information presented in these tables are not the only combination of details that will work for a given set of parameters; other pile sizes and anchor details could possibly be used.

A general description of two design examples provided herein follows:

Example 1: The first set of calculations demonstrates the design methodology for determining the foundation loads, performing the structural analysis, and calculating the capacity of timber piles with an anchor system. In this example, an abutment is designed for a PCDT superstructure with a span length and roadway width of 40 and 24 ft, respectively. The timber piles are embedded in a soil that is best described in the Iowa DOT Foundation Soils Information Chart as gravelly sand with an average standard penetration test blow count of 20. The backwall height and estimated depth of scour are six and two feet, respectively.

Example 2: The second set of calculations demonstrates the design methodology for determining the foundation loads, performing the structural analysis, and calculating capacity for steel piles without an anchor system. In this example, an abutment is designed for a prestressed concrete (PSC) superstructure with a span length and roadway width of 60 and 24 ft, respectively. The steel piles are embedded in soil that is best described in the Iowa DOT Foundation Soils Information Chart as a firm, glacial clay with an average penetration test blow count of 11. The backwall height and estimated depth of scour are six and two feet, respectively.

EXAMPLE 1

TIMBER PILE ABUTMENT WITH ANCHORS IN A COHESSIONLESS SOIL

EXAMPLE 1: TIMBER PILE ABUTMENT WITH AN ANCHOR IN A COHESIONLESS SOIL

BRIDGE INFORMATION

Pre-cast double tee superstructure (see Iowa DOT Report TR-410 Standards)

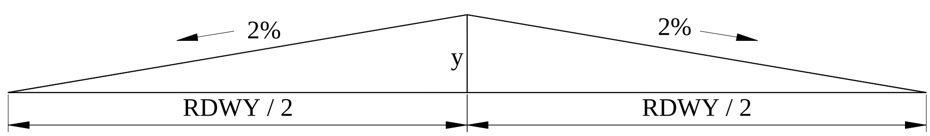
$SPAN := 40ft$		Span length
$RDWY := 24ft$		Roadway width
$BW := 6ft$		Backwall height
$ES := 2ft$		Estimated scour depth
$W21 := 21in$		W21x57 girder depth
Slab depth = 8in		
$Z_b := BW - 8in - W21$	$Z_b = 3.583\,ft$	Distance between bearing and stream elevations
$SPT := 20$		Standard penetration test blow count for a soil best described as a coarse sand in the Iowa DOT FSIC
$FB := 0.7 \cdot \frac{ton}{ft}$		Pile friction bearing resistance (Iowa DOT FSIC)
$NA := 2$		Number of abutments

GRAVITY LOADS

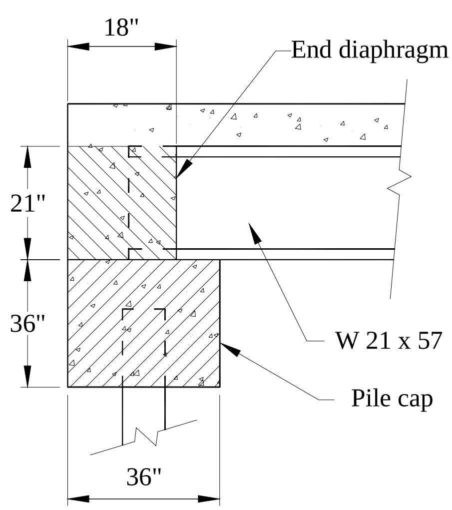
Dead Loads

$GL := SPAN + 2 \cdot (6in)$	$GL = 41.00\,ft$	Girder length
$BL := GL + 2 \cdot (6in)$	$BL = 42.00\,ft$	Bridge length
$G := 57plf$		W21x57 girder weight per foot (Iowa DOT TR-410)
$N_G := 8$		Number of girders

$BR := 50plf$		Conservatively assumed thrie-beam rail weight per foot
$FWS := 20psf$		Assumed future wearing surface
$\gamma_c := 0.150kcf$		Concrete unit weight
$Slab := (8in) \cdot BL \cdot RDWY \cdot \gamma_c$	$Slab = 100.80\,kip$	Calculated slab weight
$Girder := N_G \cdot G \cdot GL$	$Girder = 18.70\,kip$	Calculated girder weight
$Rail := 2 \cdot BR \cdot BL$	$Rail = 4.20\,kip$	Calculated barrier rail weight
$FWS_{wt} := FWS \cdot RDWY \cdot BL$	$FWS_{wt} = 20.16\,kip$	Calculated future wearing surface weight



$y := \frac{RDWY}{2} \cdot 2\%$	$y = 0.240\,ft$	
$A := \frac{1}{2} \cdot y \cdot \frac{RDWY}{2} \cdot 2$	$A = 2.880\,ft^2$	Cross sectional area of crown
$Crown := BL \cdot A \cdot \gamma_c$	$Crown = 18.14\,kip$	Calculated crown weight



<u>Abutment Cross Section</u>

$\text{Diaphragm} := (18\text{in})\cdot(21\text{in})\cdot\text{RDWY}\cdot\gamma_c\cdot\text{NA}$ $\qquad$ $\text{Diaphragm} = 18.90\,\text{kip}$ $\qquad$ Calculated end diaphragm weight (for conservative weight calculations only)

$\text{Cap} := (3\text{ft})\cdot(3\text{ft})\cdot\text{RDWY}\cdot\gamma_c\cdot\text{NA}$ $\qquad$ $\text{Cap} = 64.80\,\text{kip}$ $\qquad$ Calculated pile cap weight

$\text{Wale} := 2\cdot(20\text{plf})\cdot\text{RDWY}\cdot\text{NA}$ $\qquad$ $\text{Wale} = 1.92\,\text{kip}$ $\qquad$ Calculated abutment wale weight (2, 20 plf wales per abutment)

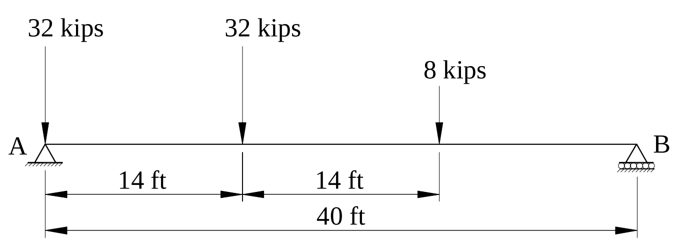
$$DL_{gb} := \text{Slab} + \text{Girder} + \text{Rail} + FWS_{wt} + \text{Crown} + \text{Diaphragm} + \text{Cap} + \text{Wale}$$

$DL_{gb} = 247.62\,\text{kip}$ $\qquad$ Total bridge dead load

$DL_g := \dfrac{DL_{gb}}{NA}\cdot 1.05$ $\qquad$ $DL_g = 130.00\,\text{kip}$ $\qquad$ Dead load abutment reaction (increased by 5 % because standards for nonspecific bridge sites were used)

Live Load

AASHTO HS20-44 design truck $\qquad$ (AASHTO 3.7)



$$\Sigma M_B = 0 = (8\text{kip})\cdot(40\text{ft} - 28\text{ft}) + (32\text{kip})\cdot(40\text{ft} + 14\text{ft}) + (32\text{kip})\cdot(40\text{ft}) - R_A\cdot(40\text{ft})$$

$$R_A := \frac{(8\text{kip})\cdot(40\text{ft} - 28\text{ft}) + (32\text{kip})\cdot(40\text{ft} - 14\text{ft}) + (32\text{kip})\cdot(40\text{ft})}{40\text{ft}}$$

$R_A = 55.20\,\text{kip}$ $\uparrow$

For 1 traffic lane, maximum live load abutment reaction = 55.20 kips.

$\dfrac{\text{RDWY}}{10\cdot\text{ft}} = 2.4$ $\qquad$ Number of 10 ft design traffic lanes (AASHTO 3.6.1)

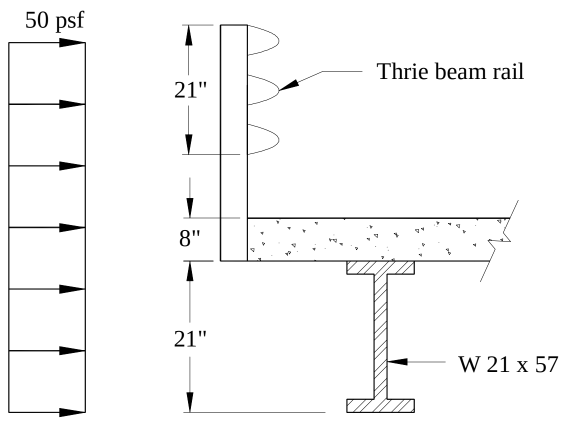
$LN := 2$		Round down to 2 traffic lanes
No lane reduction factor needed.		(AASHTO 3.12.1)
$LL_g := LN \cdot R_A$	$LL_g = 110.40\,kip$	Calculated live load abutment reaction
$TAR := LL_g + DL_g$	$TAR = 240.40\,kip$	Total abutment reaction
$pf := 1.40$		Nominal axial pile factor (Volume II, Chapter 2)
$FAR := TAR \cdot 1.4$	$FAR = 336.56\,kip$	Total factored abutment reaction
$MPL := 25ton$		Maximum axial pile load (assume embedded pile length is greater than 30 ft) (Iowa DOT BDM 6.2.6.3)
$N_1 := \dfrac{FAR}{\left[MPL \cdot \left(2 \cdot \dfrac{kip}{ton}\right)\right]}$	$N_1 = 6.73$	7 piles will work
$N := 7$		Use 7 piles
$S := \dfrac{RDWY - 2 \cdot (0.75ft)}{(N - 1)}$	$S = 3.750\,ft$	Pile spacing with 9 in. between edge of roadway and first exterior pile

LATERAL LOADS

Transverse Loads

Transverse wind loads are assumed to be divided equally among all piles and are transferred through shear at the bridge bearings.

WIND ON SUPERSTRUCTURE (Iowa DOT BDM 6.6.2.6.1)



$EA := (1.75ft + 8in + W21) \cdot SPAN$	$EA = 166.67\,ft^2$	Bridge superstructure elevation surface area
$WS := \dfrac{EA \cdot (50psf)}{NA \cdot N}$	$WS = 0.60\,kip$	Wind on superstructure force per pile

WIND ON LIVE LOAD

$LL_W := 100plf$		Line load applied to entire bridge length (Iowa DOT BDM 6.6.2.6.2)
$WL := LL_W \cdot \dfrac{SPAN}{(NA \cdot N)}$	$WL = 0.29\,kip$	Wind on live load force per pile

Longitudinal Loads

BRAKING FORCE (Iowa DOT BDM 6.6.2.4)

5% of the AASHTO lane gravity loading multiplied by the number of 10 ft design lanes.

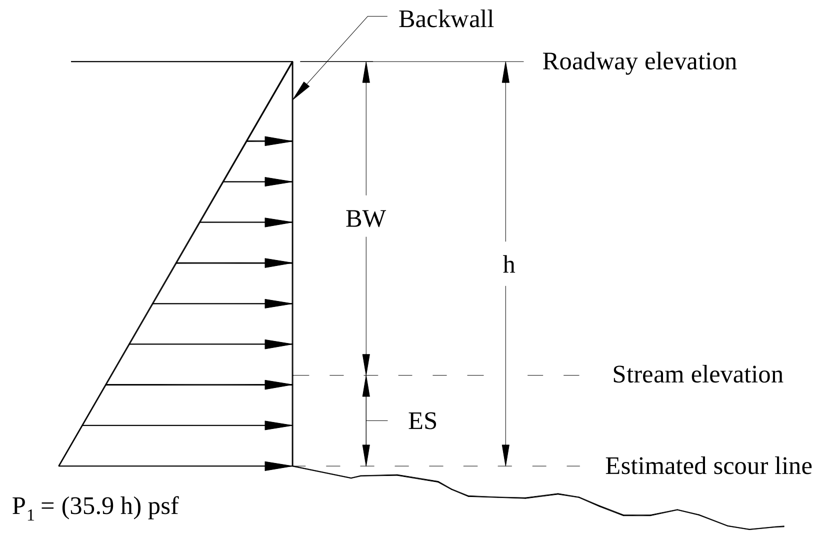
$W := 0.64klf$

$F := 18kip$

$BFP := \dfrac{LN \cdot (W \cdot SPAN + F) \cdot 0.05}{NA \cdot N}$ $\quad BFP = 0.31\,kip$ $\quad$ Braking force per pile

DEAD LOAD EARTH PRESSURE

(Iowa DOT BDM 6.5.2.4)



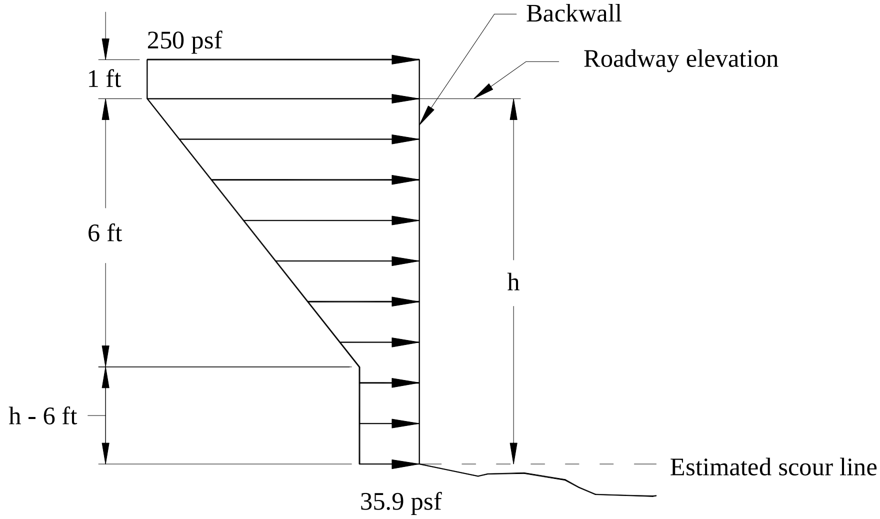
$h := BW + ES$ $\quad h = 8.00\,ft$

$P_1 := (35.9pcf) \cdot h$ $\quad P_1 = 287.2\,psf$

$w_1 := P_1 \cdot S$ $\quad w_1 = 1.077\,klf$ $\quad$ Convert P_1 to a distributed pile line load

$EDL := \dfrac{1}{2} \cdot w_1 \cdot h$ $\quad EDL = 4.31\,kip$ $\quad$ Total lateral force per pile from active earth pressure

LIVE LOAD SURCHARGE (Iowa DOT BDM 6.5.2.2)



$w_2 := (250psf) \cdot S$ $w_2 = 0.938\,klf$ Convert soil pressures into distributed loads

$w_3 := (35.9psf) \cdot S$ $w_3 = 0.135\,klf$

$LL_{sur} := (1ft) \cdot w_2 + \frac{1}{2} \cdot \left(w_2 - w_3\right) \cdot (6ft) + h \cdot w_3$ $LL_{sur} = 4.42\,kip$ Total lateral force per pile from live load surcharge

DETERMINE DEPTH TO PILE FIXITY

f = depth to fixity $f = 0.82 \cdot \sqrt{\frac{H}{\gamma_c \cdot B \cdot K_p}}$ For a cohessionless soil (Broms, 1964)

For this example, an anchor system is used. This requires an interative consisten deformation process starting with an initial assumption for the anchor force per pile.

$F := 5.00kip$ Assumed anchor force per pile

$H := BFP + LL_{sur} + EDL - F$ $H = 4.04\,kip$ Total lateral force per pile

B = pile width $D_b := 13in$ Pile butt diameter

$D_t := 10in$ Pile tip diameter

To account for the change in cross section use a representative pile diameter.

$B := D_t + 0.33 \cdot (D_b - D_t)$ $B = 10.99\,in$ (AASHTO 13.7.3.4.3)

$\phi = 53.881deg - (27.603deg) \cdot e^{-0.0147 \cdot SPT}$ $\phi := 33.309deg$

$K_p := \dfrac{1 + \sin(\phi)}{1 - \sin(\phi)}$ $K_p := 3.436$ Rankine passive earth pressure coefficient (assume soil surface behind the backwall is horizontal)

$\gamma_s := 0.125kcf$ Soil unit weight

$f := 0.82 \cdot \sqrt{\dfrac{H}{(\gamma_s \cdot B \cdot K_p)}}$ $f = 2.629\,ft$ Depth below estimated scour line to pile fixity

Pile and Anchor Properties

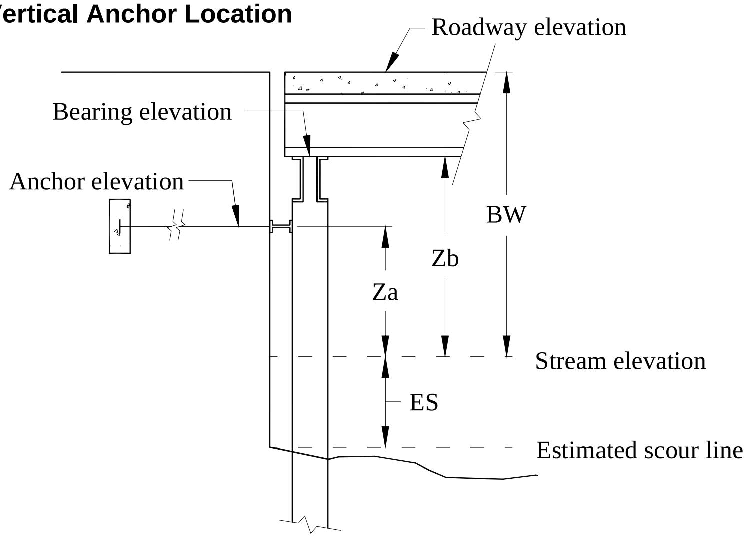
$A := \dfrac{\pi}{4} \cdot (B^2)$ $A = 94.86\,in^2$ Representative pile area

$\gamma_t := 0.05kcf$ Timber unit weight

$PSW := A \cdot \gamma_t$ $PSW = 0.033\,klf$ Pile self-weight per foot

$I := \dfrac{\pi}{64} \cdot B^4$ $I = 716.1\,in^4$ Pile moment of intertia

Vertical Anchor Location



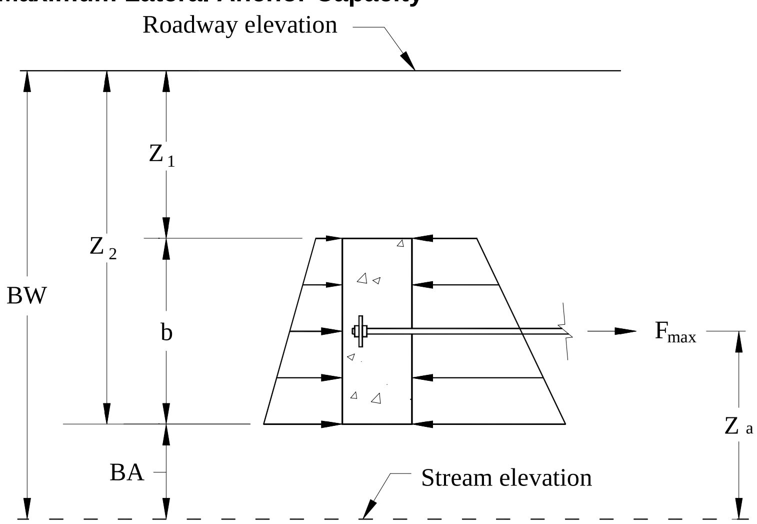
$BW = 6.00\,ft$

$ES = 2.00\,ft$

$Z_b = 3.583\,ft$

$Z_a := 2.583ft$ | Distance between stream and anchor elevations (1ft below the bearings)

Maximum Lateral Anchor Capacity



$\gamma_s = 0.125\,kcf$

$b := 3.00ft$ — Anchor height

$BA := Z_a - \frac{1}{2} \cdot b$ $\quad BA = 1.083\,ft$ — Distance between stream elevation and bottom of anchor block

$Z_1 := BW - BA - b$ $\quad Z_1 = 1.917\,ft$ — Distance between roadway elevation and top of anchor block

$Z_2 := Z_1 + b$ $\quad Z_2 = 4.917\,ft$ — Distance between roadway elevation and bottom of anchor block

F_{max} = maximum lateral anchor capacity = $(\gamma_s*b/2)*(Z_1 + Z_2)*(K_p - K_a)$ (Bowles 13-8.3)

$\phi_b := 33.69deg$ — Backfill soil friction angle (Iowa DOT BDM 6.5.2.5)

$K_{pa} := \frac{1 + \sin(\phi_b)}{1 - \sin(\phi_b)}$ $\quad K_{pa} := 3.491$ — Rankine passive earth pressure coefficient for backfill soil

$K_{aa} := K_{pa}^{-1}$ $\quad K_{aa} = 0.286$ — Rankine active earth pressure coefficient for backfill soil

$FM := \frac{\gamma_s \cdot (b)}{2} \cdot (Z_2 + Z_1) \cdot (K_{pa} - K_{aa})$	$FM = 4.106\ \text{klf}$	Maximum lateral anchor capacity per foot
$FS := 1.5$		Factor of safety
	$S = 3.750\ \text{ft}$	Pile spacing
$FMP := \frac{FM \cdot S}{FS}$	$FMP = 10.27\ \text{kip}$	Maximum anchor block capacity per pile

Therefore 5 kips anchor force assumption is **OK**.

Compute New Anchor Force Per Pile

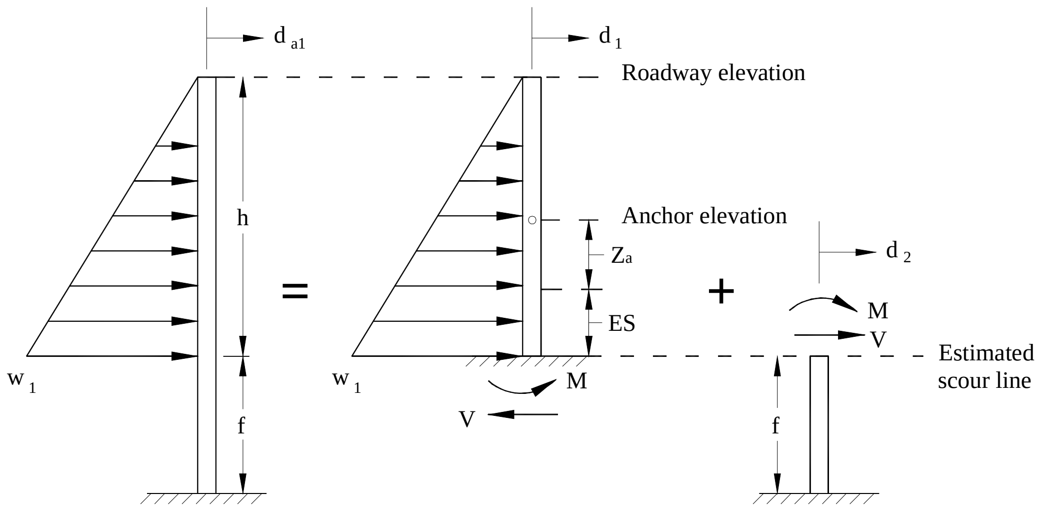
$F = \sigma \cdot A$ $\quad\quad$ $\sigma = E \cdot \varepsilon$ $\quad\quad$ $\varepsilon = \frac{\Delta L}{L_o}$ $\quad\quad$ ΔL= Pile deflection at anchor elevation

Use superposition to compute deflection of the pile at the elevation of the anchor rod.

5 loadings to consider:

1) Dead load earth pressure
2) Live load surcharge
3) Assumed anchor force
4) Braking force
5) Passive soil pressure on pile

1) DEAD LOAD EARTH PRESSURE



$ES = 2.00\,ft$ $BW = 6.00\,ft$ $Z_a = 2.583\,ft$

$h = 8.00\,ft$ $f = 2.629\,ft$

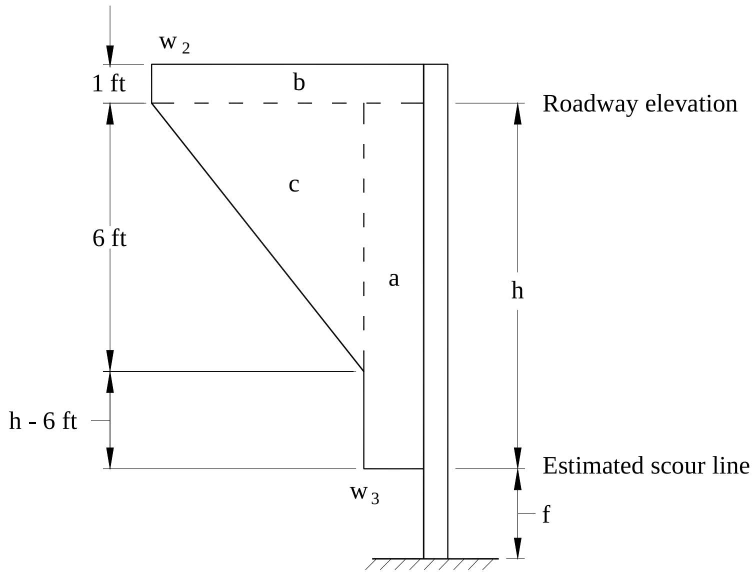
$x := Z_a + ES$	$x = 55.0\,in$	Distance between anchor elevation and estimated scour line
$E := 1600ksi$		Timber modulus of elasticity for southern pine (AASHTO Table 13.5.1A)

$w_1 = 1.077\,klf$ $I = 716.1\,in^4$

$$d_1 := \frac{w_1 \cdot x^2}{120 \cdot h \cdot E \cdot I} \cdot \left[\left(10 \cdot h^3 - 10 \cdot h^2 \cdot x \right) + 5 \cdot h \cdot x^2 - x^3 \right]$$

	$d_1 = 0.104\,in$	Pile deflection at anchor elevation
$M := \frac{1}{2} \cdot h \cdot w_1 \cdot \frac{h}{3}$	$M = 11.49\,ft \cdot kip$	Moment at estimated scour line
$V := \frac{1}{2} \cdot h \cdot w_1$	$V = 4.31\,kip$	Shear at estimated scour line
$d_2 := \frac{1}{E \cdot I} \left[\frac{\left(M \cdot f^2 \right)}{2} + \frac{\left(V \cdot f^3 \right)}{3} \right]$	$d_2 = 0.099\,in$	Pile deflection at estimated scour line
$\theta := \frac{1}{E \cdot I} \cdot \left[M \cdot f + \frac{V \cdot \left(f^2 \right)}{2} \right]$	$\theta = 0.006\,rad$	Pile slope at estimated scour line
$d_{a1} := d_1 + d_2 + \theta \cdot \left(Z_a + ES \right)$	$d_{a1} = 0.515\,in$	Total pile deflection at anchor elevation from active earth pressure

2) LIVE LOAD SURCHARGE

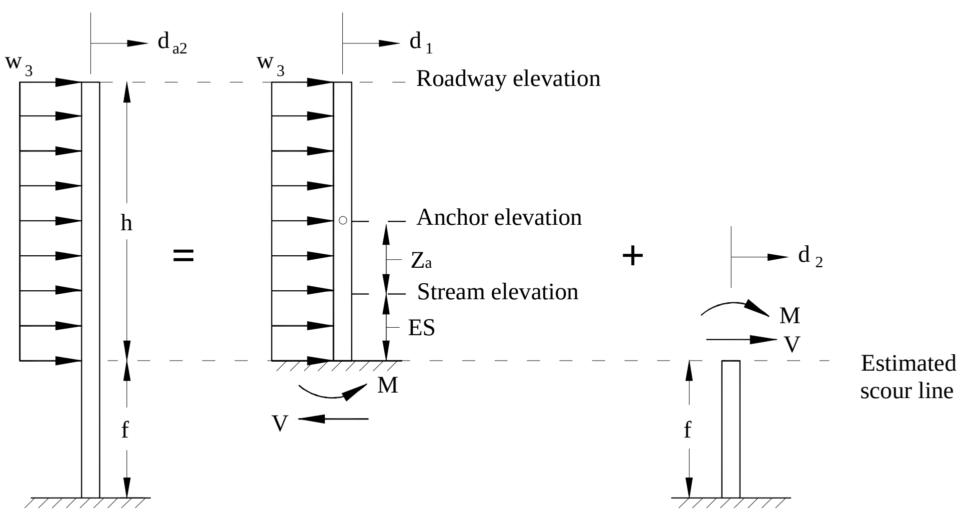


$f = 2.629\,\text{ft}$ $ES = 2.00\,\text{ft}$ $BW = 6.00\,\text{ft}$

$h = 8.00\,\text{ft}$ $Z_a = 2.583\,\text{ft}$

$w_2 = 0.938\,\text{klf}$ $w_3 = 0.135\,\text{klf}$

Part a)



$x := Z_a + ES$ $x = 55.00\,\text{in}$ Distance between anchor elevation and estimated scour line

$d_1 := \frac{w_3 \cdot (x^2)}{24 \cdot E \cdot I} \cdot (6 \cdot h^2 - 4 \cdot h \cdot x + x^2)$ | $d_1 = 0.046\,in$ | Pile deflection at anchor elevation

$M := \frac{w_3 \cdot h^2}{2}$ | $M = 4.31\,ft \cdot kip$ | Moment at estimated scour line

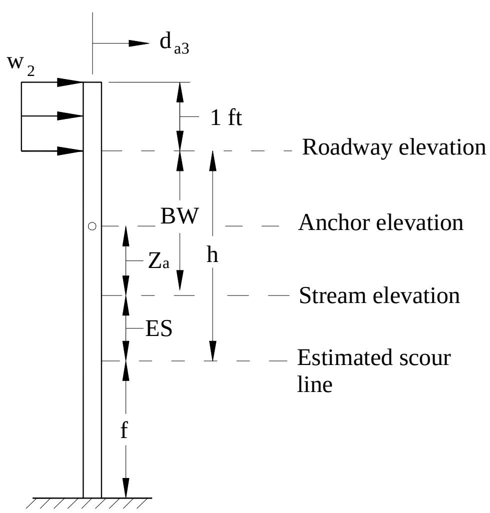
$V := w_3 \cdot h$ | $V = 1.08\,kip$ | Shear at estimated scour line

$d_2 := \frac{1}{E \cdot I}\left[\frac{(M \cdot f^2)}{2} + \frac{(V \cdot f^3)}{3}\right]$ | $d_2 = 0.032\,in$ | Pile deflection at estimated scour line

$\theta := \frac{1}{E \cdot I} \cdot \left(M \cdot f + \frac{V \cdot f^2}{2}\right)$ | $\theta = 0.002\,rad$ | Pile slope at estimated scour line

$d_{a2} := d_1 + d_2 + \theta \cdot x$ | $d_{a2} = 0.182\,in$ | Total pile defelction at anchor elevation from Part a) of live load surcharge

Part b)

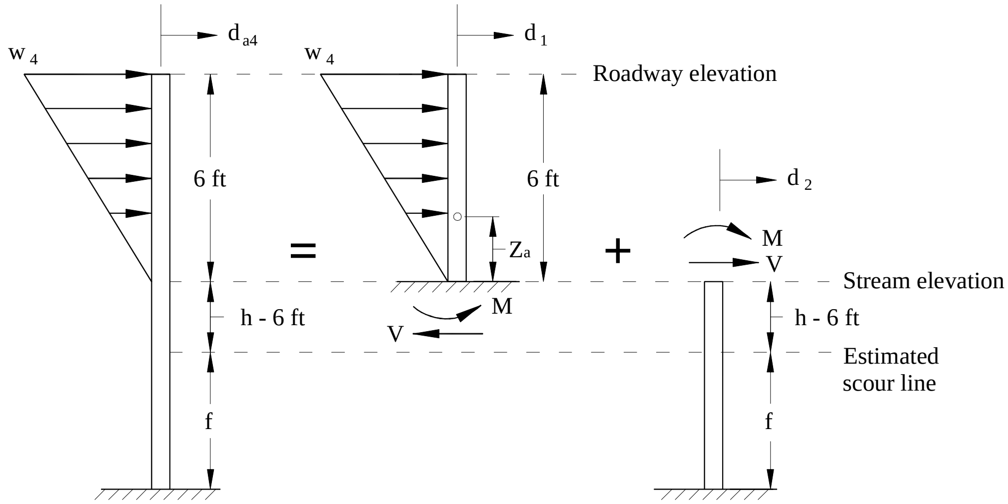


$L := ES + BW + f + 1 \cdot ft$ | $L = 11.629\,ft$ | Pile length from point of fixity to 1 ft above roadway elevation

$x := f + ES + Z_a$ | $x = 7.212\,ft$ | Distance between anchor elevation and point of fixity

$d_{a3} := \frac{w_2 \cdot (1ft) \cdot x^2}{2 \cdot E \cdot I} \cdot \left[\left(\frac{-1}{3}\right) \cdot x - \frac{1}{2} \cdot (1ft) + L\right]$ | $d_{a3} = 0.321\,in$ | Total pile deflection at anchor elevation from Part b) of live load surcharge

Part c)



$w_4 := w_2 - w_3$ $\quad w_4 = 0.803\,\text{klf}$

$$d_1 := \frac{w_4 \cdot Z_a^{\,2}}{120 \cdot (6 \cdot ft) \cdot E \cdot I} \cdot \left[20 \cdot (6 \cdot ft)^3 - 10 \cdot (6 \cdot ft)^2 \cdot Z_a + Z_a^{\,3}\right]$$

$d_1 = 0.038\,\text{in}$ — Pile deflection at anchor elevation

$V := \frac{1}{2} \cdot w_4 \cdot (6ft)$ $\quad V = 2.41\,\text{kip}$ — Shear at stream elevation

$M := V \cdot \left(\frac{2}{3}\right) \cdot (6ft)$ $\quad M = 9.63\,\text{ft} \cdot \text{kip}$ — Moment at stream elevation

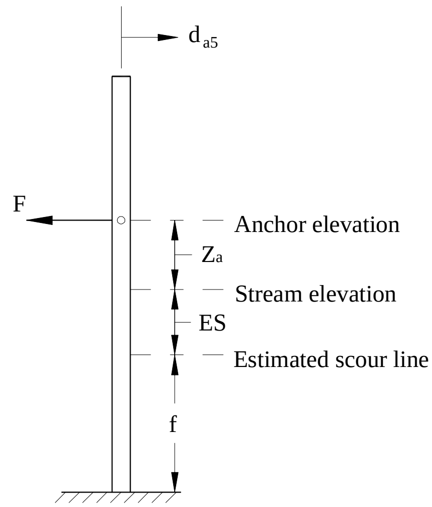
$x := f + ES$ $\quad x = 4.629\,\text{ft}$ — Distance between point of fixity and stream elevation

$d_2 := \frac{1}{E \cdot I} \left(\frac{M \cdot x^2}{2} + \frac{V \cdot x^3}{3}\right)$ $\quad d_2 = 0.276\,\text{in}$ — Pile deflection at stream elevation

$\theta := \frac{1}{E \cdot I} \cdot \left(M \cdot x + \frac{V \cdot x^2}{2}\right)$ $\quad \theta = 0.009\,\text{rad}$ — Pile slope at stream elevation

$d_{a4} := d_1 + d_2 + \theta \cdot Z_a$ $\quad d_{a4} = 0.588\,\text{in}$ — Total pile deflection at anchor elevation from Part c) of live load surcharge

3) ANCHOR FORCE



$F = 5.00\,kip$

$Z_a = 2.583\,ft$

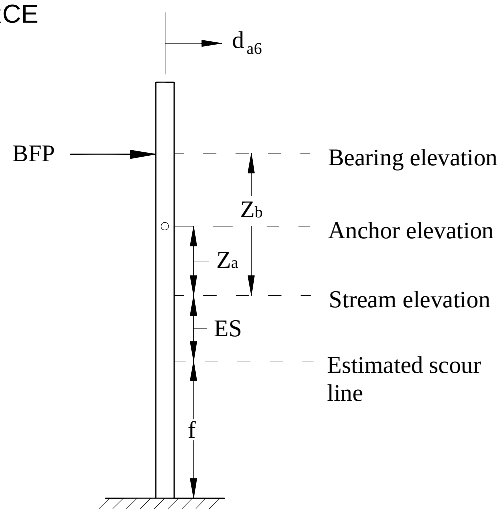
$f = 2.629\,ft$

$ES = 2.00\,ft$

$x := f + ES + Z_a$ — $x = 7.212\,ft$ — Distance between point of fixity and anchor elevation

$d_{a5} := \dfrac{-F \cdot x^3}{3 \cdot E \cdot I}$ — $d_{a5} = -0.943\,in$ — Total pile deflection at anchor elevation from assumed anchor force

4) BRAKING FORCE



$f = 2.629\,ft$

$ES = 2.00\,ft$

$BFP = 0.31\,kip$

$Z_a = 2.583\,ft$

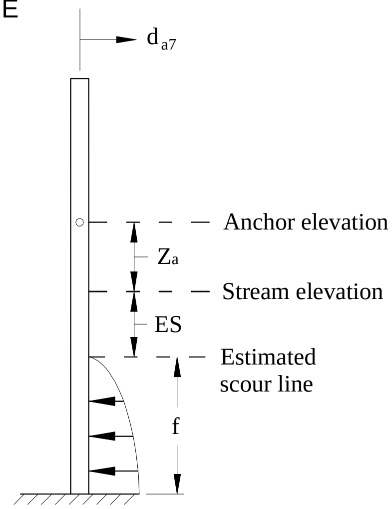
$Z_b = 3.583\,ft$

$x_1 := f + ES + Z_a$ — $x_1 = 7.212\,ft$ — Dist. between point of pile fixity and anchor elevation

$x_2 := f + ES + Z_b$ — $x_2 = 8.212\,ft$ — Dist. between point of pile fixity and bearing elevation

$d_{a6} := \dfrac{BFP \cdot x_1^{\,2}}{6E \cdot I} \cdot \left[3 \cdot (x_2) - x_1\right]$ — $d_{a6} = 0.071\,in$ — Pile deflection at anchor elevation from braking force

5) PASSIVE EARTH PRESSURE



$f = 2.629\,\mathrm{ft}$

$ES = 2.00\,\mathrm{ft}$

$Z_a = 2.583\,\mathrm{ft}$

$H = 4.04\,\mathrm{kip}$

$$\alpha := \frac{1.92 \cdot H}{f^2} \qquad \alpha = 1.123\,\mathrm{ksf}$$

$$\xi := 0.12 \cdot \frac{H}{f^3} \qquad \xi = 0.027\,\mathrm{kcf}$$

Constants in equation of parabolic passive soil reaction distribution

$$x := f + ES + Z_a \qquad x = 7.212\,\mathrm{ft}$$

Distance between point of fixity and anchor elevation

$$d_{a7} := \left(\frac{\alpha \cdot f^4 \cdot x}{24} + \frac{\xi \cdot f^5 \cdot x}{60} - \frac{\alpha \cdot f^5}{120} - \frac{\xi \cdot f^6}{120}\right) \cdot \left(\frac{-1}{E \cdot I}\right)$$

Total pile deflection at anchor elevation from passive soil reaction

$$d_{a7} = -0.023\,\mathrm{in}$$

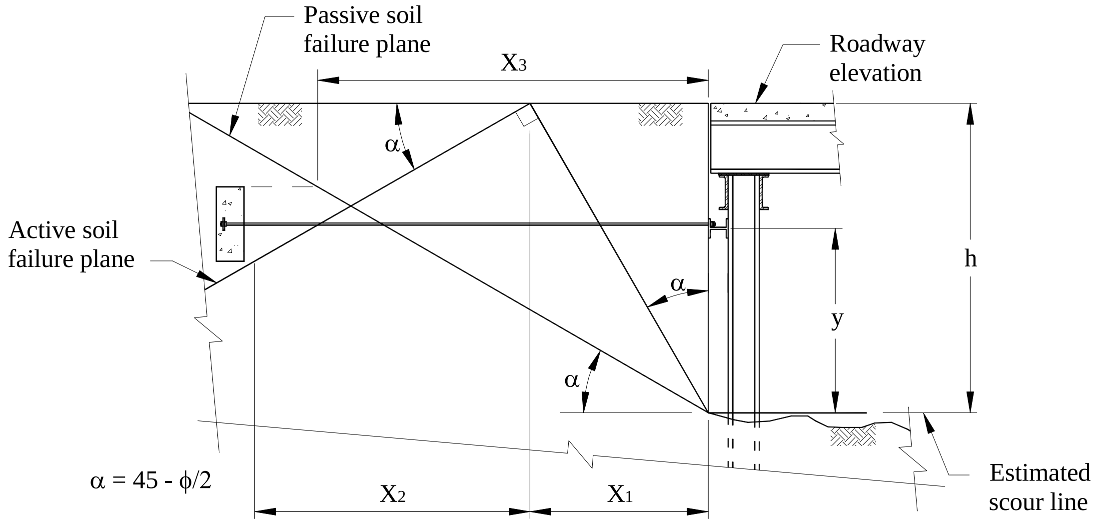
$$d_{aT} := d_{a1} + d_{a2} + d_{a3} + d_{a4} + d_{a5} + d_{a6} + d_{a7}$$

$$d_{aT} = 0.711\,\mathrm{in}$$

Total pile deflection at anchor elevation

Pile deflection at the anchor location = **0.711 in.** with an assumed anchor force of 5 kips per pile.

Anchor Rod Length (Bowles, 1997)



$h = 8.00\,ft$ $b = 3.00\,ft$

$ES = 2.00\,ft$ $Z_a = 2.583\,ft$

$y := Z_a + ES - \frac{1}{2}\cdot b$ $y = 3.083\,ft$ Distance between estimated scour line and anchor elevation

Two possibilities:

a) active failure plane controls

b) passive failure plane controls

<u>Case A:</u> minimum rod length = x_1+x_2

$\phi_b := 33.69deg$ Backfill soil friction angle (Iowa DOT BDM 6.5.2.4)

$\alpha := 45deg - \frac{\phi_b}{2}$ $\alpha = 28.16\,deg$

$\tan(\theta) := \frac{x_1}{h}$

$x_1 := \tan(\alpha)\cdot h$ $x_1 := 4.28\cdot ft$

$x_2 := \frac{h - y}{\tan(\alpha)}$ $x_2 := 9.20\cdot ft$

$x_1 + x_2 = 13.480\,ft$ Case A) minimum anchor rod length

Case B:

$x_3 := \dfrac{y + b}{\tan(\alpha)}$ $x_3 := 11.37 \cdot ft$ Case B) minimum anchor rod length

$13.47ft > 11.48ft$ Minimum anchor rod length = 13.47 ft.

$x_r := 15ft$ Anchor rod length used for this analysis

$d_{aT} = 0.711\,in$ Anchor rod elongation and pile deflection at anchor elevation

$\varepsilon_r := \dfrac{d_{aT}}{x_r}$ $\varepsilon_r = 0.0040$ Anchor rod strain

$f_y := 60ksi$ Anchor rod yield stress

$\varepsilon_y := \dfrac{f_y}{29000ksi}$ $\varepsilon_y = 0.0021$ Anchor rod yield strain

$\varepsilon_y < \varepsilon_r$

Therefore:

$\sigma_r := 60ksi$ Anchor rod stress

Assume the axial stiffness of all anchor rods are evenly distributed to the piles.

$N_r := 5$ Anchor rods per abutment

$\phi_r := 0.75in$ Anchor rod diameter

$N = 7$ Number of piles

$A_{rp} := \dfrac{\pi}{4} \cdot \phi_r^{\,2} \cdot \dfrac{N_r}{N}$ $A_{rp} = 0.316\,in^2$ Anchor rod area per pile

$F_{ap} := \sigma_r \cdot A_{rp}$ $F_{ap} = 18.93\,kip$ Calculated anchor force per pile

$F := 9.00kip$ Use a new anchor rod force of 9.0 kip/pile instead of 18.93kip (less than maximum anchor capacity of 10.3 kips).

Determine Depth to Pile Fixity

$F = 9.00\,kip$ $LL_{sur} = 4.42\,kip$

f = depth to fixity $f := 0.82 \cdot \sqrt{\dfrac{H}{\gamma_c \cdot B \cdot K_p}}$

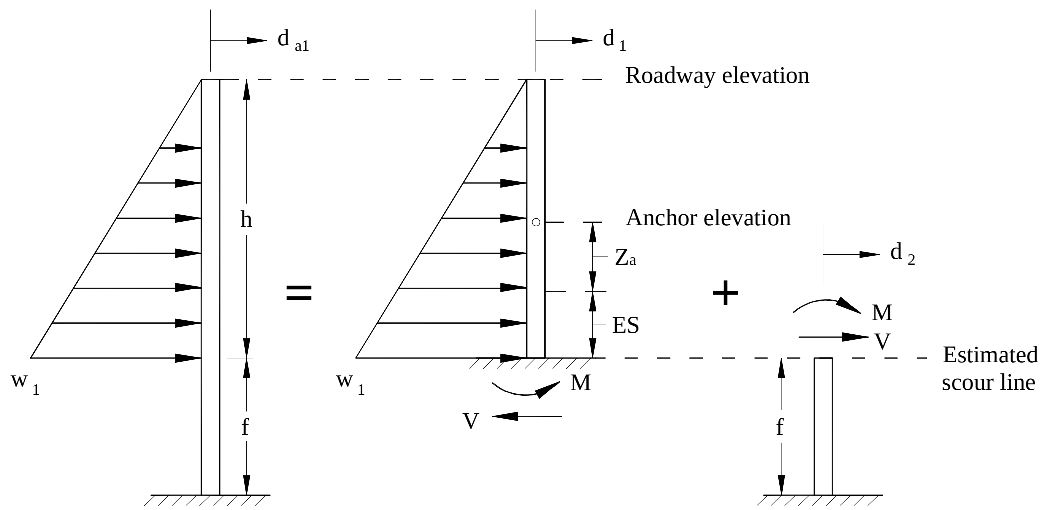
$BFP = 0.31\,kip$ $EDL = 4.31\,kip$

$H := BFP + LL_{sur} + EDL - F$ | $H = 0.04\,kip$ | Total above ground lateral pile load

$f := 0.82 \cdot \sqrt{\dfrac{H}{\gamma_s \cdot B \cdot K_p}}$ | $f = 0.270\,ft$

Compute the deflection of the pile at the elevation of the anchor rod.

1) DEAD LOAD EARTH PRESSURE



$f = 0.270\,ft$ $ES = 2.00\,ft$ $h = 8.00\,ft$

$Z_a = 2.583\,ft$ $w_1 = 1.077\,klf$

$x := Z_a + ES$ | $x = 55.00\,in$ | Distance between estimated scour line and anchor elevation

$E = 1600\,ksi$ | Timber modulus of elasticity

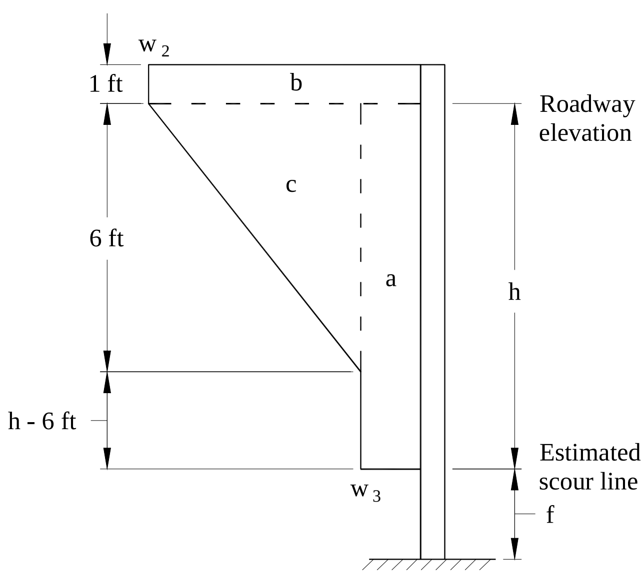
$I = 716.1\,in^4$ | Representative moment of inertia

$$d_1 := \frac{w_1 \cdot \left(x^2\right)}{120 \cdot h \cdot E \cdot I} \cdot \left[\left(10 \cdot h^3 - 10 \cdot h^2 \cdot x\right) + 5 \cdot h \cdot x^2 - x^3\right]$$

$d_1 = 0.104\,in$ | Pile deflection at anchor elevation

$M := \frac{1}{2} \cdot h \cdot w_1 \cdot \left(\frac{h}{3}\right)$	$M = 11.49\,ft \cdot kip$	Moment at estimated scour line
$V := \frac{1}{2} \cdot h \cdot w_1$	$V = 4.31\,kip$	Shear at estimated scour line
$d_2 := \frac{1}{E \cdot I}\left[\frac{\left(M \cdot f^2\right)}{2} + \frac{\left(V \cdot f^3\right)}{3}\right]$	$d_2 = 0.001\,in$	Pile deflection at estimated scour line
$\theta := \frac{1}{E \cdot I}\left[(M) \cdot f + \frac{V \cdot \left(f^2\right)}{2}\right]$	$\theta = 0.000\,rad$	Pile slope at estimated scour line
$d_{a1} := d_1 + d_2 + \theta \cdot \left(Z_a + ES\right)$	$d_{a1} = 0.127\,in$	Total pile deflection at anchor elevation from active earth pressure

2) LIVE LOAD SURCHARGE



$w_2 = 0.938\,klf$

$w_3 = 0.135\,klf$

$w_4 = 0.803\,klf$

$f = 0.270\,ft$

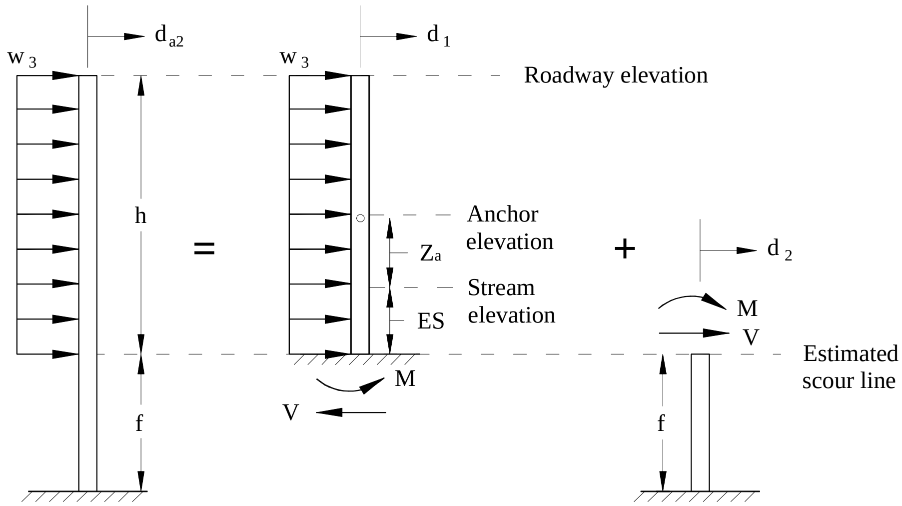
$ES = 2.00\,ft$

$BW = 6.00\,ft$

$h = 8.00\,ft$

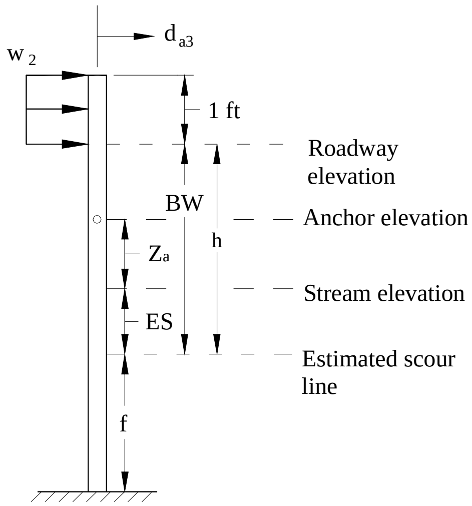
$Z_a = 2.583\,ft$

Part a)



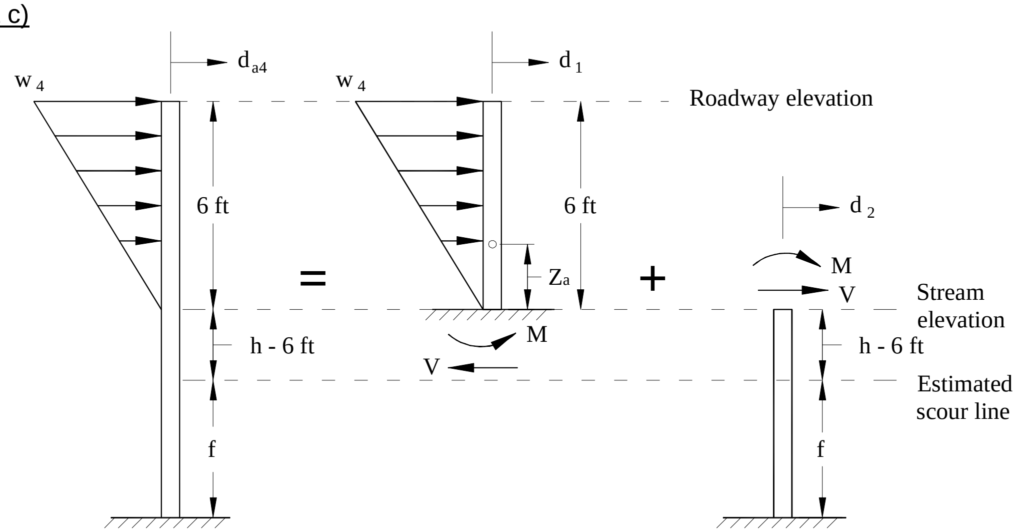
$x := Z_a + ES$	$x = 55.00\,in$	Distance between estimated scour line and anchor elevation
$d_1 := \dfrac{w_3 \cdot \left(x^2\right)}{24 \cdot E \cdot I} \cdot \left(6 \cdot h^2 - 4 \cdot h \cdot x + x^2\right)$	$d_1 = 0.046\,in$	Pile slope at anchor elevation
$M := \dfrac{w_3 \cdot h^2}{2}$	$M = 4.31\,ft \cdot kip$	Moment at estimated scour line
$V := w_3 \cdot h$	$V = 1.08\,kip$	Shear at estimated scour line
$d_2 := \dfrac{1}{E \cdot I}\left[\dfrac{\left(M \cdot f^2\right)}{2} + \dfrac{\left(V \cdot f^3\right)}{3}\right]$	$d_2 = 0.0002\,in$	Pile deflection at estimated scour line
$\theta := \dfrac{1}{E \cdot I}\left(M \cdot f + \dfrac{V \cdot f^2}{2}\right)$	$\theta = 0.0002$	Pile slope at estimated scour line
$d_{a2} := d_1 + d_2 + \theta \cdot x$	$d_{a2} = 0.054\,in$	Total pile deflection at anchor elevation from Part a) of live load surcharge

Part b)



$L := ES + BW + f + 1ft$	$L = 9.270\,ft$	Distance between point of fixity and 1 ft above roadway elevation
$x := f + ES + Z_a$	$x = 4.853\,ft$	Distance between and anchor elevation and point of fixity
$d_{a3} := \dfrac{w_2 \cdot (1ft) \cdot x^2}{2 \cdot E \cdot I} \cdot \left[\left(\dfrac{-1}{3}\right) \cdot x - \dfrac{1}{2} \cdot (1ft) + L\right]$	$d_{a3} = 0.119\,in$	Total pile deflection at anchor elevation from Part b) of live load surcharge

Part c)



$$d_1 := \frac{w_4 \cdot Z_a^{\,2}}{120 \cdot (6ft) \cdot E \cdot I} \cdot \left[20 \cdot (6ft)^3 - 10 \cdot (6ft)^2 \cdot Z_a + Z_a^{\,3}\right]$$

$d_1 = 0.038\,in$ — Pile deflection at anchor elevation

$V := \frac{1}{2} \cdot w_4 \cdot (6ft)$ — $V = 2.41\,kip$ — Shear at stream elevation

$M := V \cdot \left(\frac{2}{3}\right) \cdot 6ft$ — $M = 9.63\,ft \cdot kip$ — Moment at stream elevation

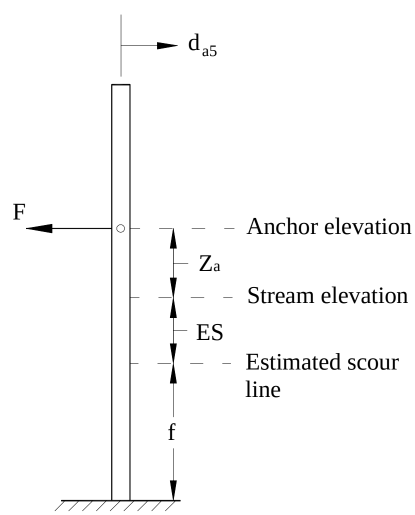
$x := f + ES$ — $x = 2.270\,ft$ — Distance between pile fixity and stream elevation

$d_2 := \frac{1}{E \cdot I}\left(\frac{M \cdot x^2}{2} + \frac{V \cdot x^3}{3}\right)$ — $d_2 = 0.052\,in$ — Pile deflection at stream elevation

$\theta := \frac{1}{E \cdot I}\left(M \cdot x + \frac{V \cdot x^2}{2}\right)$ — $\theta = 0.004\,rad$ — Pile slope at stream elevation

$d_{a4} := d_1 + d_2 + \theta \cdot Z_a$ — $d_{a4} = 0.199\,in$ — Total pile deflection at anchor elevation from Part c) of live load surcharge

3) ANCHOR FORCE



$f = 0.270\,ft$

$ES = 2.00\,ft$

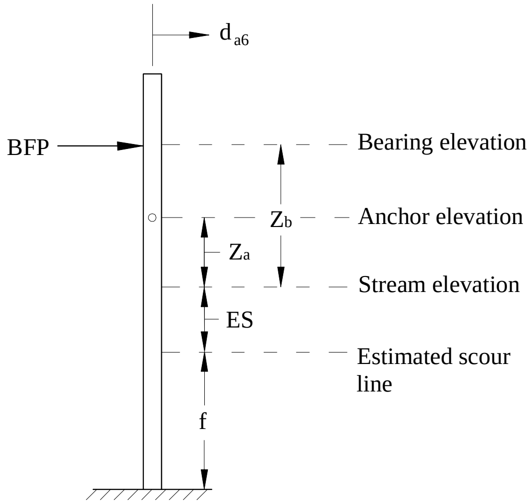
$Z_a = 2.583\,ft$

$F = 9.00\,kip$

$x := f + ES + Z_a$ — $x = 4.853\,ft$ — Distance between pile fixity and anchor elevation

$d_{a5} := \frac{-F \cdot x^3}{3 \cdot E \cdot I}$ — $d_{a5} = -0.517\,in$ — Total pile deflection at anchor elevation from assumed anchor force

4) BRAKING FORCE



$f = 0.270\,ft$

$ES = 2.00\,ft$

$Z_a = 2.583\,ft$

$Z_b = 3.583\,ft$

$BFP = 0.31\,kip$

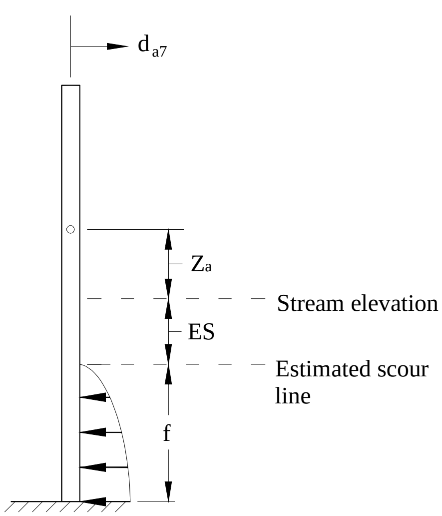
$x_1 := f + ES + Z_a$ $\qquad x_1 = 4.853\,ft$ $\qquad$ Distance between point of pile fixity and anchor elevation

$x_2 := f + ES + Z_b$ $\qquad x_2 = 5.853\,ft$ $\qquad$ Distance between point of pile fixity and bearing elevation

$$d_{a6} := \frac{BFP \cdot x_1^{\,2}}{6 \cdot E \cdot I} \cdot \left[3 \cdot \left(x_2\right) - x_1\right] \qquad d_{a6} = 0.023\,in$$

Pile deflection at anchor elevation due to braking force

5) PASSIVE EARTH PRESSURE



$f = 0.270\,ft$

$ES = 2.000\,ft$

$Z_a = 2.583\,ft$

$H = 0.043\,kip$

$\alpha := \frac{1.92 \cdot H}{f^2}$ $\alpha = 1.123\,\text{ksf}$

Constants in equation of parabolic passive soil reaction distribution

$\xi := \frac{0.12 \cdot H}{f^3}$ $\xi = 0.260\,\text{kcf}$

$$d_{a7} := \left(\frac{\alpha \cdot f^4 \cdot x}{24} + \frac{\xi \cdot f^5 \cdot x}{60} - \frac{\alpha \cdot f^5}{120} - \frac{\xi \cdot f^6}{120} \right) \cdot \left(\frac{-1}{E \cdot I} \right)$$

$d_{a7} = -1.84 \times 10^{-6}\,\text{in}$ Pile deflection at anchor elevation from passive soil reaction

$$d_{aT} := d_{a1} + d_{a2} + d_{a3} + d_{a4} + d_{a5} + d_{a6} + d_{a7}$$

$d_{aT} = 0.006\,\text{in}$ Total pile deflection at anchor elevation

Pile deflection at the anchor location **= 0.006 in**. with assumed anchor force of 9.00 kips per pile.

$\varepsilon_r := \frac{d_{aT}}{x_r}$ $\varepsilon_r = 3.56 \times 10^{-5}$ Anchor rod strain

$\sigma_r := \varepsilon_r \cdot 29000 \cdot \text{ksi}$ $\sigma_r = 1.032\,\text{ksi}$ Anchor rod stress

$F := \sigma_r \cdot A_{rp}$ $F = 0.326\,\text{kip}$ Calculated anchor force per pile

Using this calculated anchor rod force per pile, the process is repeated to determine the pile deflection at the anchor rod elevation and a new anchor rod force per pile.

The first assumed anchor force of 5 kips was too low. The next assumed value of 9 kips yielded a calculated force of 0.326 kip, thus it was too high. Therefore the next estimate should be between 5 and 9 kips. Repeat this iterative process until the assumed and calculated anchor force are equal.

After several iterations:

$F := 7.628\text{kip}$ Final anchor force per pile

$f := 1.5549\text{ft}$ Final depth to fixity below estimated scour line

$\delta_{aT} := 0.1500\text{in}$ Final pile deflection and anchor rod elongation

$\varepsilon_r := \dfrac{\delta_{aT}}{15 \cdot ft}$ | $\varepsilon_r = 0.001$ | Final anchor rod strain

$\sigma := 29000ksi \cdot (\varepsilon_r)$ | $\sigma = 24.17\,ksi$ | Final anchor rod stress (OK if 60 ksi steel is used)

$H := BFP + LL_{sur} + EDL - F$ | $H = 1.41\,kip$ | Total lateral pile load

DETERMINE MAXIMUM PILE MOMENT

Longitudinal Moment

*Use superposition and check various points along the pile length
- a) point of pile fixity (x=0)
- b) anchor location (x_b)
- c) (x_c)

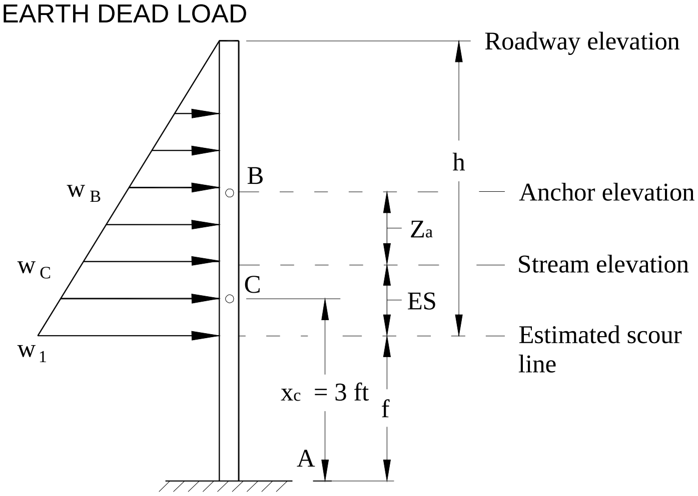
$ES = 2.00\,ft$ $Z_a = 2.583\,ft$

$x_b := f + ES + Z_a$ | $x_b = 6.138\,ft$ | Distance between pile fixity and anchor elevation

$x_c := \dfrac{x_b}{2}$ | $x_c = 3.069\,ft$ | Halfway between pile fixity and anchor elevation

$x_c := 3.00ft$ | Use $x_c = 3$ ft

EARTH DEAD LOAD



$w_1 = 1.077\,klf$ $h = 8.00\,ft$ $Z_a = 2.583\,ft$

$w_B := 0.460klf$ $f = 1.555\,ft$ $ES = 2.00\,ft$

$w_C := 0.883klf$

$$M_{A1} := \frac{1}{2} \cdot w_1 \cdot h \cdot \left(f + \frac{h}{3} \right) \qquad M_{A1} = 18.19\,ft \cdot kip$$

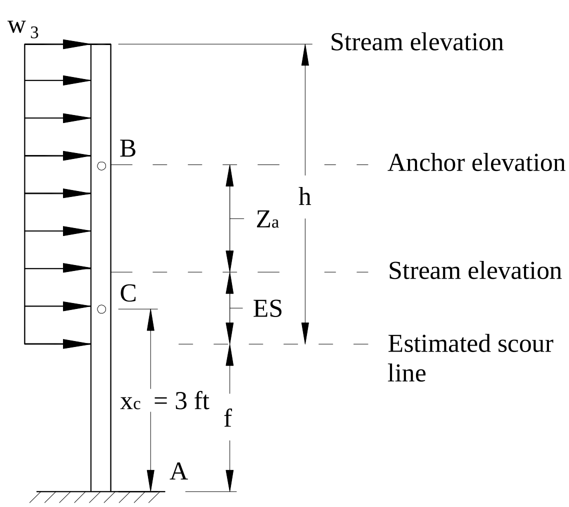
$$M_{B1} := \frac{1}{2} \cdot w_B \cdot \left(h - ES - Z_a \right)^2 \cdot \frac{1}{3} \qquad M_{B1} = 0.895\,ft \cdot kip$$

$$M_{C1} := \frac{1}{2} \cdot w_C \cdot \left(h + f - x_c \right)^2 \cdot \left(\frac{1}{3} \right) \qquad M_{C1} = 6.32\,ft \cdot kip$$

LIVE LOAD SURCHARGE

$f = 1.555\,ft$	$Z_a = 2.583\,ft$	$w_4 = 0.803\,klf$
$ES = 2.00\,ft$	$w_2 = 0.938\,klf$	
$h = 8.00\,ft$	$w_3 = 0.135\,klf$	

Part a)



$x_1 := f + h \qquad x_1 = 9.555\,ft$ Distance between pile fixity and roadway

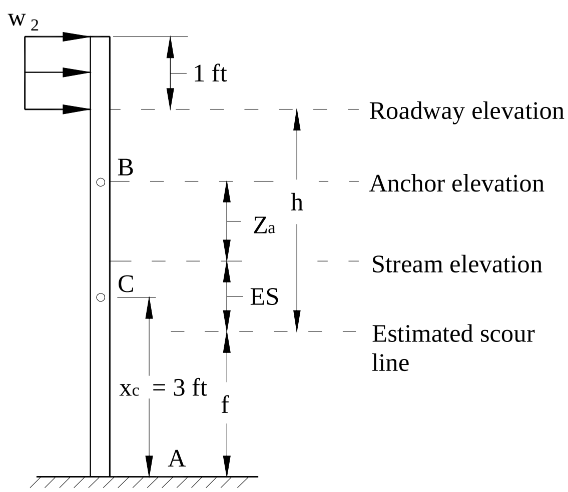
$x_2 := f + ES + Z_a \qquad x_2 = 6.138\,ft$ Distance between pile fixity and anchor elevation

$$M_{A2} := w_3 \cdot \left(x_1 - f \right) \cdot \left[f + \frac{\left(x_1 - f \right)}{2} \right] \qquad M_{A2} = 5.98\,ft \cdot kip$$

$$M_{B2} := w_3 \cdot \frac{\left(x_1 - x_2 \right)^2}{2} \qquad M_{B2} = 0.79\,ft \cdot kip$$

$$M_{C2} := w_3 \cdot \frac{\left(x_1 - x_c \right)^2}{2} \qquad M_{C2} = 2.89\,ft \cdot kip$$

Part b)



$x_1 := f + h$ $\quad$ $x_1 = 9.555\,ft$ $\quad$ Distance between pile fixity and roadway elevation

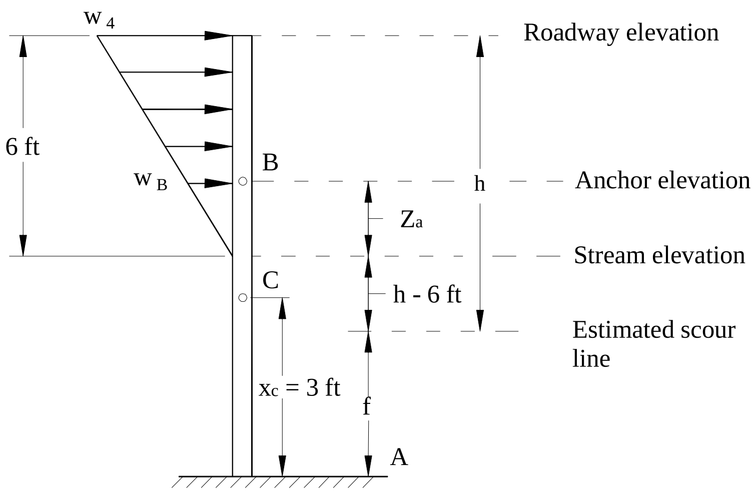
$x_2 := f + ES + Z_a$ $\quad$ $x_2 = 6.138\,ft$ $\quad$ Distance between pile fixity and anchor elevation

$$M_{A3} := w_2 \cdot (1ft) \cdot \left(x_1 + \frac{1ft}{2} \right) \qquad M_{A3} = 9.43\,ft \cdot kip$$

$$M_{B3} := w_2 \cdot (1ft) \cdot \left(x_1 - x_2 + \frac{1ft}{2} \right) \qquad M_{B3} = 3.67\,ft \cdot kip$$

$$M_{C3} := w_2 \cdot (1ft) \cdot \left(x_1 - x_c + \frac{1ft}{2} \right) \qquad M_{C3} = 6.61\,ft \cdot kip$$

Part c)



$w_B := 0.346\text{klf}$

$x_1 := f + h$	$x_1 = 9.555\,\text{ft}$	Distance between pile fixity and roadway elevation
$x_2 := f + ES + Z_a$	$x_2 = 6.138\,\text{ft}$	Distance between pile fixity and anchor elevation
$x_3 := x_1 - 6\text{ft}$	$x_3 = 3.555\,\text{ft}$	Distance between pile fixity and bottom of triangular load

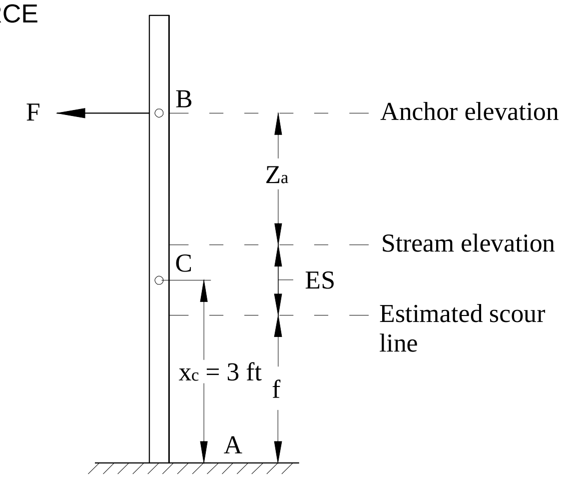
$M_{A4} := \frac{1}{2} \cdot w_4 \cdot (6\text{ft}) \cdot \left[x_3 + \left(\frac{2}{3}\right) \cdot 6\text{ft} \right]$ $\qquad M_{A4} = 18.20\,\text{ft} \cdot \text{kip}$

$$M_{B4} := w_B \cdot \frac{(x_1 - x_2)^2}{2} + \frac{1}{2} \cdot (w_4 - w_B) \cdot (x_1 - x_2)^2 \cdot \left(\frac{2}{3}\right)$$

$M_{B4} = 3.80\,\text{ft} \cdot \text{kip}$

$M_{C4} := \frac{1}{2} \cdot w_4 \cdot (6\text{ft}) \cdot \left[x_3 - x_c + \left(\frac{2}{3}\right) \cdot 6\text{ft} \right]$ $\qquad M_{C4} = 10.97\,\text{ft} \cdot \text{kip}$

ANCHOR FORCE



$f = 1.555\,\text{ft}$

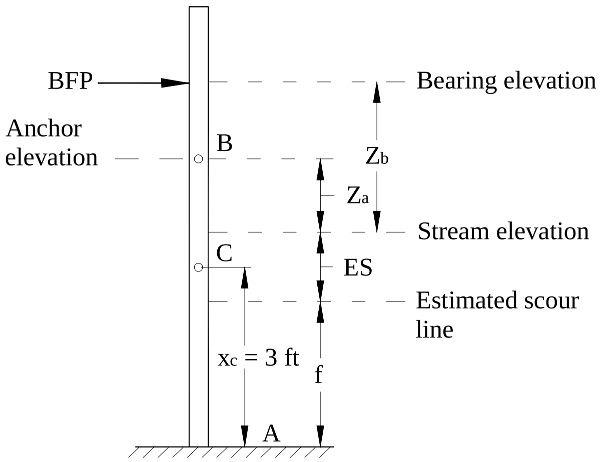
$ES = 2.00\,\text{ft}$

$Z_a = 2.583\,\text{ft}$

$F = 7.63\,\text{kip}$

$x := f + ES + Z_a$	$x = 6.138\,\text{ft}$	Distance between pile fixity and anchor elevation
$M_{A5} := -F \cdot x$	$M_{A5} = -46.82\,\text{ft} \cdot \text{kip}$	
$M_{B5} := 0.00\text{ft} \cdot \text{kip}$	$M_{B5} = 0.00\,\text{ft} \cdot \text{kip}$	
$M_{C5} := -F \cdot (x - x_c)$	$M_{C5} = -23.94\,\text{ft} \cdot \text{kip}$	

BRAKING FORCE



$f = 1.555\,ft$

$ES = 2.00\,ft$

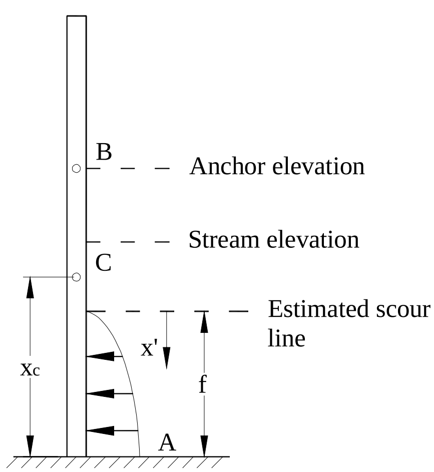
$Z_a = 2.583\,ft$

$Z_b = 3.583\,ft$

$BFP = 0.31\,kip$

$x_1 := f + ES + Z_b$	$x_1 = 7.138\,ft$	Distance between pile fixity and bearing elevation
$x_2 := f + ES + Z_a$	$x_2 = 6.138\,ft$	Distance between pile fixity and anchor elevation
$M_{A6} := BFP \cdot x_1$	$M_{A6} = 2.22\,ft \cdot kip$	
$M_{B6} := BFP \cdot (x_1 - x_2)$	$M_{B6} = 0.31\,ft \cdot kip$	
$M_{C6} := BFP \cdot (x_1 - x_c)$	$M_{C6} = 1.29\,ft \cdot kip$	

PASSIVE EARTH PRESSURE



$f = 1.555\,ft$

$H = 1.41\,kip$

$\alpha := \frac{1.92 \cdot H}{f^2}$ $\alpha = 1.123\,\text{ksf}$

$\xi := 0.12 \cdot \frac{H}{f^3}$ $\xi = 0.045\,\text{kcf}$

Constants in equation of parabolic passive soil reaction distribution

$$w(x') = \alpha * (x') + \xi * (x')^2$$

$$V(x') = \int w(x')dx$$

$$M(x') = \int V(x')dx \qquad \text{for } 0 \le x \le f$$

Derived equation for pile moement as a function of x'

$M_{A7} := \frac{-\alpha \cdot f^3}{6} - \frac{\xi \cdot f^4}{12}$ $M_{A7} = -0.73\,\text{ft} \cdot \text{kip}$ for x' = f

$M_{B7} := 0.00\text{ft} \cdot \text{kip}$ $M_{B7} = 0.00\,\text{ft} \cdot \text{kip}$

$M_{C7} := 0.00\text{ft} \cdot \text{kip}$ $M_{C7} = 0.00\,\text{ft} \cdot \text{kip}$

$M_{AT} := M_{A1} + M_{A2} + M_{A3} + M_{A4} + M_{A5} + M_{A6} + M_{A7}$

$M_{AT} = 6.47\,\text{ft} \cdot \text{kip}$

Total pile moment at point of fixity

$M_{BT} := M_{B1} + M_{B2} + M_{B3} + M_{B4} + M_{B5} + M_{B6} + M_{B7}$

$M_{BT} = 9.46\,\text{ft} \cdot \text{kip}$

Total pile moment at anchor location

$M_{CT} := M_{C1} + M_{C2} + M_{C3} + M_{C4} + M_{C5} + M_{C6} + M_{C7}$

$M_{CT} = 4.15\,\text{ft} \cdot \text{kip}$

Total pile moment halfway between anchor and fixity elevations

$M := 9.46\text{ft} \cdot \text{kip}$

Maximum total pile moment

Transverse Pile Moments

$f = 1.555\,\text{ft}$	$Z_b = 3.583\,\text{ft}$	
$ES = 2.00\,\text{ft}$		
	$WS = 0.60\,\text{kip}$	Wind on superstructure force per pile
	$WL = 0.29\,\text{kip}$	Wind on live load force per pile
$M_{WS} := WS \cdot (f + ES + Z_b)$	$M_{WS} = 4.25\,\text{ft} \cdot \text{kip}$	Wind on superstructure transverse pile moment
$M_{WL} := WL \cdot (f + ES + Z_b)$	$M_{WL} = 2.04\,\text{ft} \cdot \text{kip}$	Wind on live load transverse pile moment

PILE SELF-WEIGHT

For friction piles, the gravity load is disipated as the depth below ground increases. Therefore, only consider pile self-weight for the length above point of pile fixity.

$x := f + ES + Z_b$	$x = 7.138\,\text{ft}$	Distance between point of fixity and bearing elevation
$P_{SW} := 0.033\,\text{klf}$		Pile self-weight per foot
$P_{SWT} := P_{SW} \cdot x$	$P_{SWT} = 0.24\,\text{kip}$	Pile weight

LOAD SUMMARY

	$DL_g = 130.00\,\text{kip}$	Dead load abutment reaction
	$pf = 1.4$	Nominal axial pile factor (Chapter 2, Volume 2)
	$N = 7$	Number of piles
$P_{DL} := \dfrac{DL_g \cdot pf}{N}$	$P_{DL} = 26.00\,\text{kip}$	Pile axial dead load
$P_{LL} := \dfrac{LL_g \cdot pf}{N}$	$P_{LL} = 22.08\,\text{kip}$	Pile axial live load
$P_T := P_{DL} + P_{LL}$	$P_T = 48.08\,\text{kip}$	Pile total axial load

DESIGN CHECKS

Pile Length

$Z_b = 3.583\,ft$ $\quad$ $ES = 2.00\,ft$

$B = 10.99\,in$ — Representative pile diameter

$P_T = 48.08\,kip$ — Total axial pile load

$FB := 0.7 \cdot \frac{ton}{ft}$ — Friction bearing resistance (Iowa DOT FSIC)

$AFB := \frac{B}{10in} \cdot FB$ $\quad$ $AFB = 0.77\,\frac{ton}{ft}$ — Adjusted friction bearing resistance (Iowa DOT FSIC)

$PL := \frac{P_T \cdot \left(\frac{1ton}{2kip}\right)}{AFB}$ $\quad$ $PL = 31.249\,ft$ — Required miminum embedded pile length

$TPL := PL + ES + Z_b$ $\quad$ $TPL = 36.833\,ft$ — Required minimum total pile length

Roundup to nearst 5 ft, 40 ft < 55 ft **OK** (Iowa DOT BDM 6.2.6.3)

Allowable Axial Pile Load

25 tons for piles 30 ft and longer (Iowa DOT BDM 6.2.6.3)
20 tons for piles less than 30 ft

Since required embedded length of 31.2 ft is greater than 30 ft, the 25 ton per pile limit applies.

$P_T \cdot \left(\frac{1ton}{2kip}\right) = 24.04\,ton$ $\quad$ 24.04 tons < 25 tons **OK**

Vertical Bearing Capacity

If the embedded length is greater than or equal to 31.2 ft, then the vertical bearing capacity will be sufficient. Therefore this check is **OK**

Anchor Location

Minimum anchor rod length = 13.48 ft (previously calculated)

Anchor length used = 15 ft 13.48 ft < 15 ft **OK**

Combined Axial and Lateral Loading Check

For combined bending and axial loads, AASHTO recommends the interaction equation from the NDS Manual. Note the x and y axis are assumed to be parallel and perpendicular to the backwall face, respectively (AASHTO 13.7.2)

$$\left(\frac{f_c}{F_c{}^1}\right)^2 + \frac{f_{bx}}{F'_{bx}\left(1-\frac{f_c}{F_{cEx}}\right)} + \frac{f_{by}}{F'_{by}\cdot\left(1-\frac{f_c}{F_{cEy}}-\frac{f_{bx}}{F_{bE}}\right)} < 1.0 \qquad \text{(NDS 3.9)}$$

For the given loads, three different load combinations given in section 6.6.3.1 of the Iowa DOT BDM are applicable.

Group I: 1.0(DL)+1.0(LL)+1.0(E)+1.0(BF) using 100% of the allowable stress
Group II: 1.0(DL)+1.0(E)+1.0(WS) using 125% of the allowable stress
Group III: 1.0(DL)+1.0(LL)+1.0(E)+1.0(BF)+0.3(WS)+1.0(WL) using 125% of the allowable stress

DL = Dead load
LL = Live load
E = Earth load
BF = Longitudinal braking force
WS = Wind on superstructure
WL = Wind on live load

APPLIED STRESSES

f_c = axial compressive force

$P_{LL} = 22.08\ \text{kip}$ Pile axial live load

$P_{DL} = 26.00\ \text{kip}$ Pile axial dead load

$P_T = 48.08\ \text{kip}$ Pile total axial load

$A = 94.86\ \text{in}^2$ Representative pile area

Group I and III (with Live Load)

$f_{cT} := \frac{P_T}{A}$ $\quad f_{cT} = 0.507\,\text{ksi}$ Group I and III axial compressive stress

Group II (without Live Load)

$f_{cDL} := \frac{P_{DL}}{A}$ $\quad f_{cDL} = 0.274\,\text{ksi}$ Group II axial compressive stress

When computing the applied x-axis bending stress, the live and dead loads were not separated. Therefore, the live load surcharge and braking force is also included in the second load combination recommended by the Iowa DOT BDM (i.e., Group II) and f_{bx} is the same for all load combinations. However, the pile axial live and dead load can be separtated as demonstrated on the previous page.

$I = 716.1\,\text{in}^4$ Representative moment of inertia

$B = 10.99\,\text{in}$ Representative pile diameter

$SM := \frac{I}{\left(\frac{B}{2}\right)}$ $\quad SM = 130.3\,\text{in}^3$ Section modulus

$M = 9.46\,\text{ft}\cdot\text{kip}$ Maximum pile moment (x-axis bending)

$M_{WS} = 4.25\,\text{ft}\cdot\text{kip}$ Wind on superstructure pile moment (y-axis bending)

$M_{WL} = 2.04\,\text{ft}\cdot\text{kip}$ Wind on live load pile moment (y-axis bending)

$f_{bx} := \frac{M}{SM}$ $\quad f_{bx} = 0.871\,\text{ksi}$ Groups I, II, and III applied x-axis bending stress

$f_{byW} := \frac{M_{WS}}{SM}$ $\quad f_{byW} = 0.391\,\text{ksi}$ Group II and III applied y-axis bending bending stress from wind on superstructure

$f_{byWL} := \frac{M_{WL}}{SM}$ $\quad f_{byWL} = 0.188\,\text{ksi}$ Group III applied y-axis bending stress from wind on live load

ALLOWABLE STRESSES

When necessary, round piles shall be designed as square columns with an equivalent cross sectional area. (AASHTO 13.7.3.5)

$A = 94.86\,in^2$

$d := \sqrt{A}$ $\qquad d = 9.74\,in$ Equivalent square dimension

Allowable Compressive Stress

$F_{c'} = F_c \cdot C_m \cdot C_D \cdot C_F \cdot C_P$ (AASHTO 13.7.3.2)

Use southern pine timber piles, obtain material properties from AASHTO Table 13.5.1A.

$E = 1600\,ksi$ Tabulated timber modulus of elasticity

$F_c := 1100psi$ Tabulated timber compressive stress

$F_b := 1750psi$ Tabulated timber bending stress

$C_M := 1.0$ Wet service compression factor (AASHTO Table 13.5.1A)

$C_F := 1.0$ For sawn lumber only

$C_D := 0.90$ Load duration factor for permanent loading (AASHTO Table 13.5.5A)

$$C_p = \frac{\left(1 + \frac{F_{cEx}}{F_{c'}}\right)}{2 \cdot c} - \sqrt{\frac{\left(1 + \frac{F_{cEx}}{F_{c'}}\right)^2}{(2 \cdot c)^2} - \frac{\left(\frac{F_{cEx}}{F_{c'}}\right)}{c}}$$

Column stability factor (AASHTO 13.7.3.3.5)

$c := 0.85$ For round piles

$K_{cE} := 0.30$ For visually graded lumber

$E' := E \cdot C_M$ | $E' = 1600\,ksi$ | Adjusted modulus of elasticity

X-axis Bending:

l_e = (k) * (length between braced points)

$k_x := 0.7$

$d = 9.74\,in$ | Equivalent square dimension

$f = 1.555\,ft$ $\quad Z_a = 2.583\,ft$

$ES = 2.00\,ft$

$l_x := f + ES + Z_a$ | $l_x = 6.138\,ft$ | Distance between point of fixity and anchor elevation

$l_{ex} := l_x \cdot k_x$ | $l_{ex} = 4.297\,ft$ | Effective pile length for x-axis bending

$$F_{cEx} := \frac{K_{cE} \cdot E'}{\left(\frac{l_{ex}}{d}\right)^2}$$

$F_{cEx} = 17.13\,ksi$ | x-axis buckling stress

Y-axis Bending:

$k_y := 0.7$

$Z_b = 3.583\,ft$

$l_y := f + ES + Z_b$ | $l_y = 7.138\,ft$ | Distance between point of fixity and bearing elevation

$l_{ey} := k_y \cdot l_y$ | $l_{ey} = 4.997\,ft$ | Effective pile length for y-axis bending

$$F_{cEy} := \frac{K_{cE} \cdot E'}{\left(\frac{l_{ey}}{d}\right)^2}$$

$F_{cEy} = 12.66\,ksi$ | y-axis buckling stress

$F_{c'} := F_c \cdot C_M \cdot C_D \cdot C_F$ | $F_{c'} = 0.990\,ksi$ | Allowable axial stress without column stability factor

$$C_{px} := \frac{\left(1 + \frac{F_{cEx}}{F_{c'}}\right)}{2 \cdot c} - \sqrt{\frac{\left(1 + \frac{F_{cEx}}{F_{c'}}\right)^2}{(2 \cdot c)^2} - \frac{\left(\frac{F_{cEx}}{F_{c'}}\right)}{c}}$$ $C_{px} = 0.991$ x-axis column stability factor

$$C_{py} := \frac{\left(1 + \frac{F_{cEy}}{F_{c'}}\right)}{2 \cdot c} - \sqrt{\frac{\left(1 + \frac{F_{cEy}}{F_{c'}}\right)^2}{(2 \cdot c)^2} - \frac{\left(\frac{F_{cEy}}{F_{c'}}\right)}{c}}$$ $C_{py} = 0.988$ y-axis column stability factor

$C_{py} = 0.988$ **Controls**

$F_{c'} := F_c \cdot C_M \cdot C_D \cdot C_F \cdot C_{py}$ $F_{c'} = 0.978\,\text{ksi}$ **Allowable axial stress**

Allowable Bending Stress

$F_{b'} = F_b \cdot C_m \cdot C_D \cdot C_F \cdot C_V \cdot C_L \cdot C_f \cdot C_{fu} \cdot C_r$ Tabulated timber bending stress (AASHTO 13.6.4.1)

$C_M := 1.0$ Wet service bending factor for members 5" x 5" or larger (AASHTO Table 13.5.1A)

$C_D := 0.90$ Load duration factor for permanent loading (AASHTO Table 13.5.5A)

$C_F := 1.0$ For sawn lumber only

$C_V := 1.0$ For glued laminated timber only

$C_L := 1.0$ Equal to 1.0 for members whose depth does not exceed its width (AASHTO 13.6.4.4.2)

$C_f := 1.18$ Round member factor (AASHTO 13.6.4.5)

$C_{fu} := 1.0$ For sawn lumber only

$C_r := 1.0$ For sawn lumber only

$F_{b'} := F_b \cdot C_M \cdot C_D \cdot C_F \cdot C_V \cdot C_L \cdot C_f \cdot C_{fu} \cdot C_r$ $F_{b'} = 1.859\,\text{ksi}$ **Allowable bending stress**

INTERACTION EQUATION VALIDATION CHECK

$$F_{BE} = \frac{K_{bE} \cdot E'}{R_B^{\,2}}$$ Bending buckling stress (NDS 3.9)

$K_{bE} := 0.439$ For visually graded lumber (NDS Manual 3.3.3.6)

$E' = 1600\,\text{ksi}$

$$R_B = \sqrt{\frac{l_e \cdot d}{b^2}}$$ (NDS Manual 3.3.3.6)

b = d = 9.75 in (for square cross section)

F_{BE} is used in the interaction equation with the x-axis bending stress, therefore use the x-axis effective length of the R_B term.

$$R_B := \sqrt{\frac{l_{ex}}{d}}$$ $R_B = 2.30$

$$F_{BE} := \frac{K_{bE} \cdot E'}{R_B^{\,2}}$$ $F_{BE} = 132.69\,\text{ksi}$

To account for secondary bending effects, a P-Δ factor is used. This value must be greater than 1.0.

x-axis bending

<u>Group I and Group III (with Live Load)</u>

$$P\Delta_{x1} := \frac{1}{1 - \frac{f_{cT}}{F_{cEx}}}$$ $P\Delta_{x1} = 1.03$ **>1.0, therefore OK**

Group II (without Live Load)

$$P\Delta_{x2} := \frac{1}{\left(1 - \frac{f_{cDL}}{F_{cEx}}\right)}$$ $P\Delta_{x2} = 1.02$ **> 1.0, therefore OK**

y-axis bending

Group III (with Live Load)

$$P\Delta_{y1} := \frac{1}{1 - \left(\frac{f_{cT}}{F_{cEy}}\right) - \left(\frac{f_{bx}}{F_{BE}}\right)^2}$$ $P\Delta_{y1} = 1.04$ **> 1.0, therefore OK**

Group II (without Live Load)

$$P\Delta_{y2} := \frac{1}{1 - \left(\frac{f_{cDL}}{F_{cEy}}\right) - \left(\frac{f_{bx}}{F_{BE}}\right)^2}$$ $P\Delta_{y2} = 1.02$ **> 1.0, therefore OK**

Group I Interaction equation

$f_{cT} = 0.507\,\text{ksi}$ Applied total axial stress

$F_{c'} = 0.978\,\text{ksi}$ Allowable axial stress

$f_{bx} = 0.871\,\text{ksi}$ Applied x-axis bending stress

$F_{b'} = 1.859\,\text{ksi}$ Allowable bending stress

$P\Delta_{x1} = 1.03$ x-axis secondary moment factor

$$\left(\frac{f_{cT}}{F_{c'}}\right)^2 + \frac{f_{bx} \cdot P\Delta_{x1}}{F_{b'}} = 0.75$$ $0.75 \le 1.0$ **OK**

Group II Interaction equation

$f_{cDL} = 0.274\,\text{ksi}$ Applied dead load axial stress

$P\Delta_{x2} = 1.02$ x-axis secondary moment factor for dead load stress

$f_{byW} = 0.391\,\text{ksi}$ Applied y-axis bending stress from wind on superstructure

$P\Delta_{y2} = 1.02$ y-axis secondary moment factor for dead load stress

$$\left[\left(\frac{f_{cDL}}{F_{c'}}\right)^2 + \frac{f_{bx}\cdot P\Delta_{x2}}{F_{b'}} + \frac{f_{byW}\cdot P\Delta_{y2}}{F_{b'}}\right]\cdot\frac{1}{1.25} = 0.62$$

1.25 allowable overstress factor

$0.62 \le 1.0$ **OK**

Group III Interaction Equation

$P\Delta_{x1} = 1.03$ x-axis secondary moment factor

$f_{byWL} = 0.188\,\text{ksi}$ Applied y-axis bending stress from wind on live load

$P\Delta_{y1} = 1.04$ y-axis secondary moment factor

$$\left[\left(\frac{f_{cT}}{F_{c'}}\right)^2 + \frac{f_{bx}\cdot P\Delta_{x1}}{F_{b'}} + \frac{0.3\cdot f_{byW}\cdot P\Delta_{y1}}{F_{b'}} + \frac{f_{byWL}\cdot P\Delta_{y1}}{F_{b'}}\right]\cdot\frac{1}{1.25} = 0.74$$

1.25 allowable overstress factor

$0.74 \le 1.0$ **OK**

All interaction equations are less than or equal to 1.0. **OK**

Anchor Rod Stress

$\sigma = 24.17\,\text{ksi}$ — Applied anchor rod stress

$f_y := 60 \cdot \text{ksi}$

$\sigma_a := 0.55 \cdot f_y$ — $\sigma_a = 33.00\,\text{ksi}$ — Allowable stress is equal to 55% of the yield stress (AASHTO Table 10.32.1A)

$24.17\text{ksi} < 33\text{ksi}$ — **OK**

Anchor Block Lateral Capacity

$FMP = 10.27\,\text{kip}$ — Maximum allowable anchor force per pile

$F = 7.63\,\text{kip}$ — Applied anchor force per pile

$7.63\text{kip} < 10.27\text{kip}$ — **OK**

Maximum Abutment Displacement

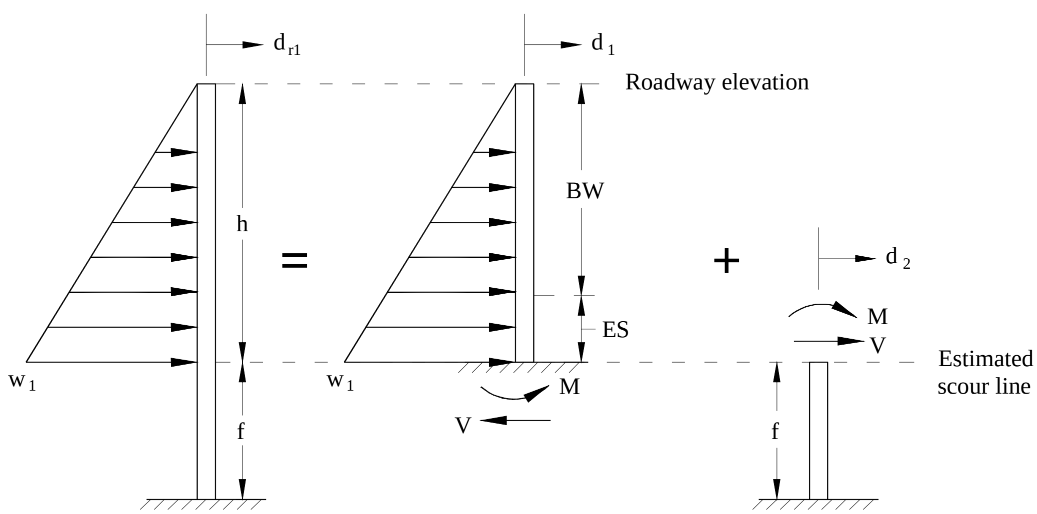
Maximum horizontal displacement = 1.5 in — (AASHTO 4.4.7.2.5 via 4.5.12)

$\delta_{aT} = 0.150\,\text{in}$ — Pile deflection at anchor elevation

0.150 in < 1.50 in — **OK**

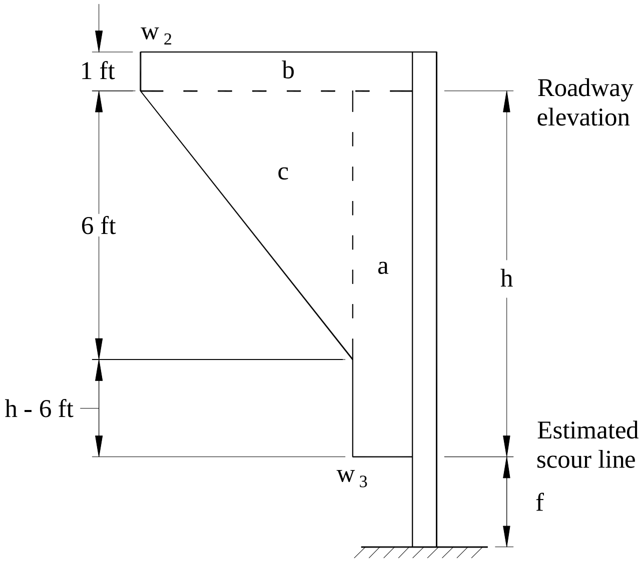
Must check displacement at roadway elevation

1) DEAD LOAD EARTH PRESSURE



	$f = 1.555\,ft$	$h = 8.00\,ft$
	$ES = 2.00\,ft$	$BW = 6.00\,ft$
	$w_1 = 1.077\,klf$	
$x := ES + BW$	$x = 96.0\,in$	Distance between anchor elevation and estimated scour line
	$E = 1600\,ksi$	
	$I = 716.1\,in^4$	
$d_1 := \dfrac{w_1 \cdot (x^2)}{120 \cdot h \cdot E \cdot I} \cdot 4 \cdot x^3$	$d_1 = 0.222\,in$	Pile deflection at roadway elevation
$M := \dfrac{1}{2} \cdot h \cdot w_1 \cdot \left(\dfrac{h}{3}\right)$	$M = 11.49\,ft \cdot kip$	Moment at estimated scour line
$V := \dfrac{1}{2} \cdot h \cdot w_1$	$V = 4.31\,kip$	Shear at estimated scour line
$d_2 := \dfrac{1}{E \cdot I}\left[\dfrac{(M \cdot f^2)}{2} + \dfrac{(V \cdot f^3)}{3}\right]$	$d_2 = 0.029\,in$	Pile deflection at estimated scour line
$\theta := \dfrac{1}{E \cdot I}\left(M \cdot f + \dfrac{V \cdot f^2}{2}\right)$	$\theta = 0.003\,rad$	Pile slope at estimated scour line
$d_{r1} := (d_1 + d_2 + \theta \cdot x)$	$d_{r1} = 0.529\,in$	Total pile deflection at roadway elevation from active earth pressure

2) LIVE LOAD SURCHARGE



$f = 1.555\,ft$

$ES = 2.00\,ft$

$BW = 6.00\,ft$

$h = 8.00\,ft$

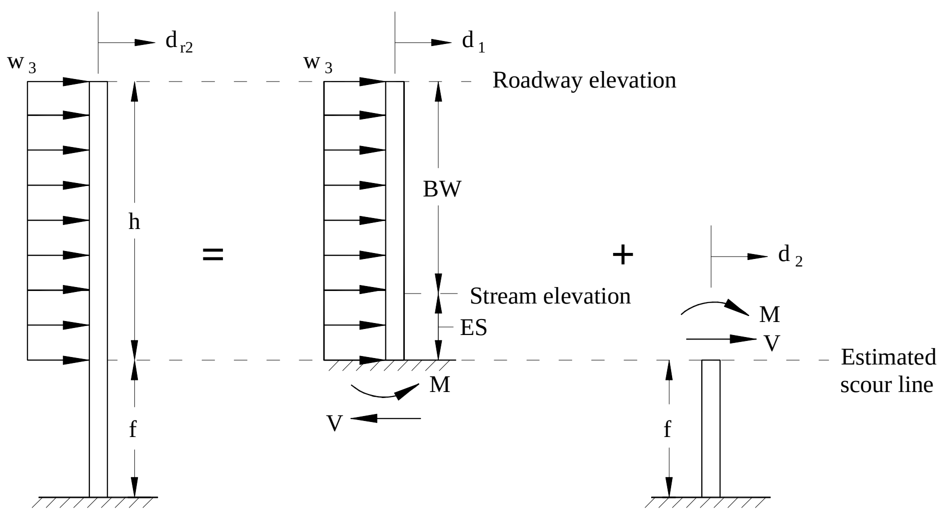
$Z_a = 2.583\,ft$

$w_2 = 0.938\,klf$

$w_3 = 0.135\,klf$

$w_4 = 0.803\,klf$

Part a)



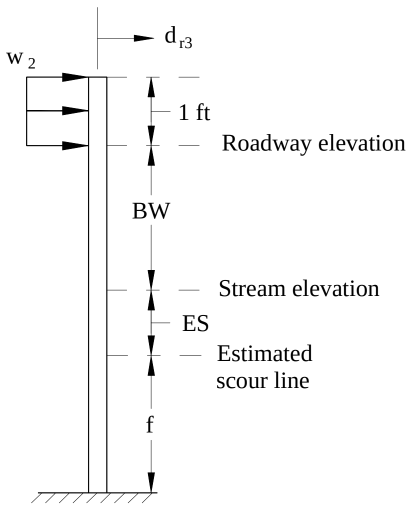
$d_1 := \frac{w_3 \cdot (x^2)}{24 \cdot E \cdot I} \cdot (3 \cdot x^2)$ $d_1 = 0.104\,in$ Pile deflection at roadway elevation

$M := \frac{w_3 \cdot h^2}{2}$ $M = 4.31\,ft \cdot kip$ Moment at estimated scour line

$V := w_3 \cdot h$ $V = 1.08\,kip$ Shear at estimated scour line

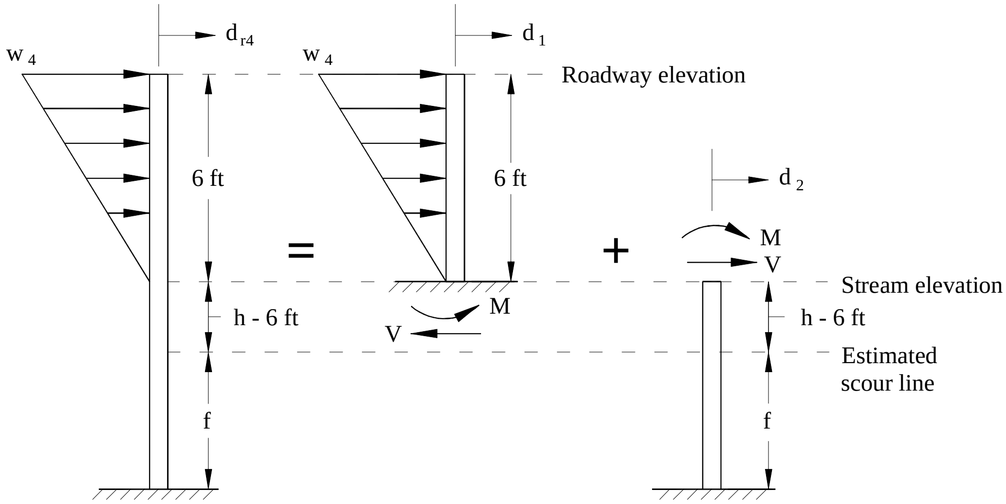
$d_2 := \frac{1}{E \cdot I}\left[\frac{\left(M \cdot f^2\right)}{2} + \frac{\left(V \cdot f^3\right)}{3}\right]$	$d_2 = 0.010\,in$	Pile deflection at estimated scour line
$\theta := \frac{1}{E \cdot I} \cdot \left(M \cdot f + \frac{V \cdot f^2}{2}\right)$	$\theta = 0.001\,rad$	Pile slope at estimated scour line
$d_{r2} := d_1 + d_2 + \theta \cdot x$	$d_{r2} = 0.210\,in$	Total pile deflection at roadway elevation from Part a) of live load surcharge

Part b)



$L := f + ES + BW + 1 \cdot ft$	$L = 10.555\,ft$	Distance between point of fixity and 1 ft above roadway
$x := f + ES + BW$	$x = 9.555\,ft$	Roadway elevation above point of fixity
$d_{r3} := \frac{w_2 \cdot 1 \cdot ft \cdot x^2}{2 \cdot E \cdot I} \cdot \left[\left(\frac{-1}{3}\right) \cdot x - \frac{1}{2} \cdot 1 \cdot ft + L\right]$	$d_{r3} = 0.443\,in$	Pile deflection from Part b) of live load surcharge

Part c)



$d_1 := \frac{w_4 \cdot (6 \cdot ft)^2}{120 \cdot (6 \cdot ft) \cdot E \cdot I} \cdot 11 \cdot [6 \cdot (ft)]^3$ — $d_1 = 0.144\,in$ — Pile deflection at roadway elevation

$V := \frac{1}{2} \cdot w_4 \cdot 6 \cdot ft$ — $V = 2.41\,kip$ — Shear at stream elevation

$M := V \cdot \left(\frac{2}{3}\right) \cdot 6 \cdot ft$ — $M = 9.63\,ft \cdot kip$ — Moment at stream elevation

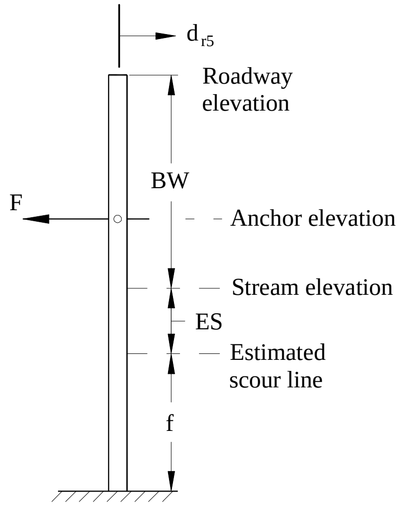
$x := f + ES$ — $x = 3.555\,ft$

$d_2 := \frac{1}{E \cdot I} \cdot \left(\frac{M \cdot x^2}{2} + \frac{V \cdot x^3}{3}\right)$ — $d_2 = 0.146\,in$ — Pile deflection at stream elevation

$\theta := \frac{1}{E \cdot I} \cdot \left(M \cdot x + \frac{V \cdot x^2}{2}\right)$ — $\theta = 0.006\,rad$ — Pile slope at stream elevation

$d_{r4} := d_1 + d_2 + \theta \cdot 6 \cdot ft$ — $d_{r4} = 0.738\,in$ — Total pile deflection from Part c) of live load surcharge

3) ANCHOR FORCE



$f = 1.555\,ft$

$ES = 2.00\,ft$

$BW = 6.00\,ft$

$Z_a = 2.583\,ft$

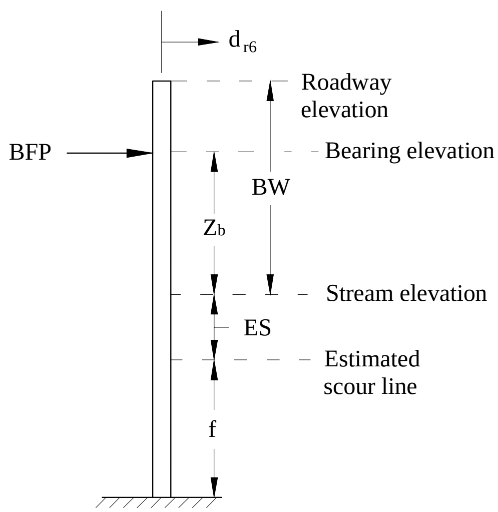
$F = 7.63\,kip$

$x_1 := f + ES + Z_a$ $\quad$ $x_1 = 6.138\,ft$ $\quad$ Distance between pile fixity and anchor elevation

$x_2 := f + ES + BW$ $\quad$ $x_2 = 9.555\,ft$ $\quad$ Distance between pile fixity and roadway elevation

$d_{r5} := \frac{-F \cdot x_1^{\,2}}{6 \cdot E \cdot I} \cdot \left(3 \cdot x_2 - x_1\right)$ $\quad$ $d_{r5} = -1.627\,in$ $\quad$ Total pile deflection from anchor force

4) BRAKING FORCE



$f = 1.555\,ft$

$ES = 2.00\,ft$

$BW = 6.00\,ft$

$Z_b = 3.583\,ft$

$BFP = 0.31\,kip$

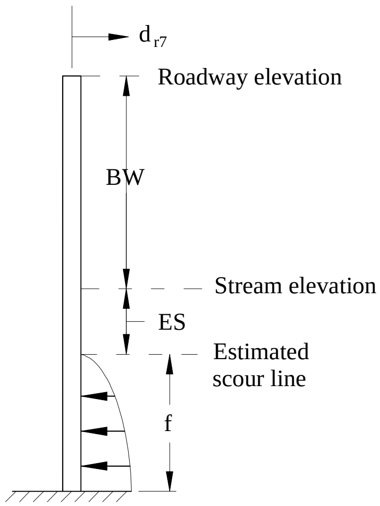
$x_1 := f + ES + Z_b$ $\quad$ $x_1 = 7.138\,ft$ $\quad$ Distance between point of pile fixity and bearing elevation

$x_2 := f + ES + BW$ $\quad x_2 = 9.555\,ft$ $\quad$ Distance between point of pile fixity and roadway elevation

$$d_{r6} := \frac{BFP \cdot x_1^{\,2}}{6E \cdot I} \cdot (3 \cdot x_2 - x_1) \qquad d_{r6} = 0.086\,in$$

Total pile deflection from braking force

5) PASSIVE EARTH PRESSURE



$f = 1.555\,ft$

$ES = 2.00\,ft$

$BW = 6.00\,ft$

$H = 1.41\,kip$

$$\alpha := \frac{1.92 \cdot H}{f^2} \qquad \alpha = 1.123\,ksf$$

$$\xi := \frac{0.12 \cdot H}{f^3} \qquad \xi = 0.045\,kcf$$

Constants in equation of parabolic passive soil reaction distribution

$x := f + ES + BW$ $\quad$ Distance between pile fixity and roadway elevation

$$d_{r7} := \left(\frac{\alpha \cdot f^4 \cdot x}{24} + \frac{\xi \cdot f^5 \cdot x}{60} - \frac{\alpha \cdot f^5}{120} - \frac{\xi \cdot f^6}{120}\right) \cdot \left(\frac{-1}{E \cdot I}\right)$$

$$d_{r7} = -0.004\,in$$

Total pile deflection from passive soil reaction

$$d_{rT} := d_{r1} + d_{r2} + d_{r3} + d_{r4} + d_{r5} + d_{r6} + d_{r7} \qquad d_{rT} = 0.375\,in$$

Total pile deflection at roadway elevation

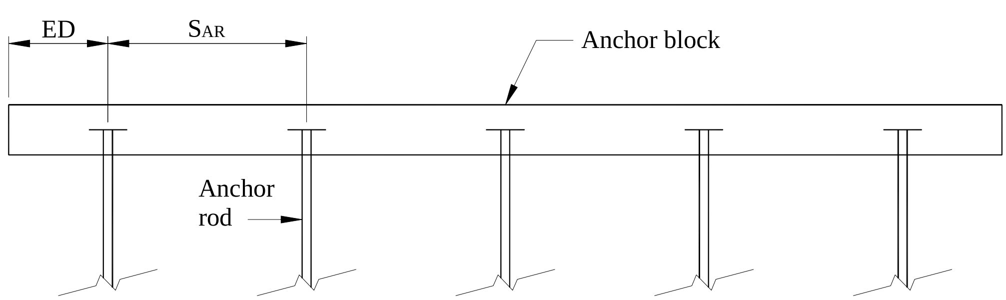
$0.375 \cdot in \le 1.5 \cdot in$ $\quad$ **OK**

ANCHOR BLOCK DESIGN (AASHTO, Section 8)

$S = 3.75\,ft$	Pile spacing
$F = 7.63\,kip$	Anchor force per pile
$N_r = 5$	Number of anchor rods
$N = 7$	Number of piles
$b = 3.00\,ft$	Anchor height
$h_a := 12in$	Anchor width
$f'_c := 3ksi$	Concrete compressive strength
$f_y = 60\,ksi$	Reinforcing steel yield strength

Determine Anchor Block Loads

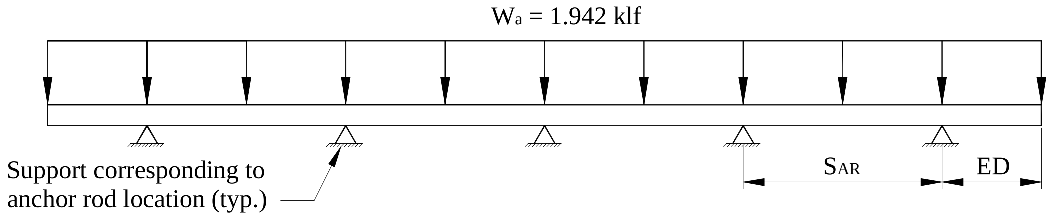
$F_{aT} := N \cdot F$	$F_{aT} = 53.40\,kip$	Total anchor force per abutment
$L_{min} := N \cdot S$	$L_{min} = 26.250\,ft$	Minimum anchor block length
$ABL := 27.00ft$		Anchor block length used for analysis



Anchor block plan view

$ED := 1.5ft$		Distance between end of the anchor block and exterior anchor rod
$S_{AR} := \dfrac{ABL - 2 \cdot ED}{N_r - 1}$	$S_{AR} = 6.000\,ft$	Anchor rod spacing

STRUCTURAL MODEL



$w_a := \dfrac{F_{aT}}{ABL}$ $\quad$ $w_a = 1.978\,\text{klf}$ $\quad$ Passive soil reaction distribution imparted on anchor block

From indeterminate structural analysis:

$M := 7.00\text{ft}\cdot\text{kip}$ $\quad$ **Maximum anchor block moment**

$V := 6.73\text{kip}$ $\quad$ **Maximum anchor block shear**

AASHTO LOAD COMBINATIONS

Since dead and live load anchor forces are not separated, use earth load factors.

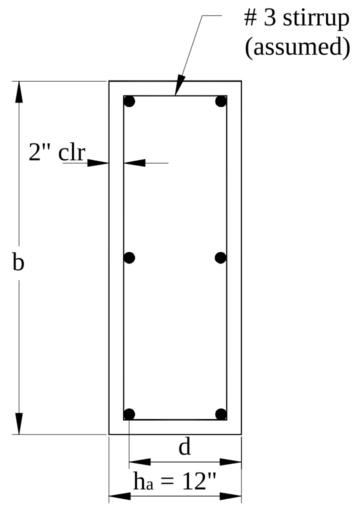
Group I Loading: 1.3(1.3E) = 1.69E $\quad$ (AASHTO 3.22)

$M_u := 1.69 \cdot M$ $\quad$ $M_u = 11.83\,\text{ft}\cdot\text{kip}$ $\quad$ Factored anchor block moment

$V_u := 1.69 \cdot V$ $\quad$ $V_u = 11.37\,\text{kip}$ $\quad$ Factored anchor block shear

$\phi_b := 0.9$ $\quad$ Resistance factor for bending (AASHTO 8.16.1.2.2)

$\phi_v := 0.85$ $\quad$ Resistance factor for shear (AASHTO 8.16.1.2.2)



Design Checks

FLEXURAL CAPACITY

$$\phi M_n = \phi_b \cdot A_s \cdot f_y \cdot \left(d - 0.60 \cdot \frac{A_s \cdot f_y}{f'_c \cdot b} \right)$$

Design flexural capacity (AASHTO 8.16.3.2)

Assume # 3 stirrup is used

$h_a = 12.0\,in$ — Anchor block width

$d_3 := \frac{3}{8} \cdot in$ — Diameter of # 3 stirrup

$d_4 := \frac{4}{8} in$ — Diameter of # 4 bar (assumed)

$d := h_a - 2 \cdot in - d_3 - \frac{1}{2} \cdot d_4$ $\quad d = 9.37\,in$ — Effective concrete depth

$b = 3.00\,ft$ $\quad b = 36.00\,in$ — Width of compression block

$f'_c = 3\,ksi$ — Concrete compressive strength

$f_y = 60\,ksi$ — Reinforcing steel yield strength

Set M_U equal to ϕM_n and determine the area of steel required by solving the resulting quadratic equation.

$A_{sREQ} := 0.28in^2$

Use 3 - # 4 bars on each vertical face

$A_s := 3 \cdot 0.20 \cdot in^2$	$A_s = 0.60\, in^2$	Tension steel area provided
$\phi M_n := \phi_b \cdot A_s \cdot f_y \cdot \left(d - 0.60 \cdot \frac{A_s \cdot f_y}{f'_c \cdot b} \right)$	$\phi M_n = 24.77\, ft \cdot kip$	Flexure design capacity
	$24.77 ft \cdot kip > 11.83 ft \cdot kip$	**OK**

REINFORCEMENT RATIO (AASHTO 8.16.3.2.2)

$\rho := \frac{A_s}{b \cdot d}$	$\rho = 0.0018$	Reinforcement ratio
$\beta_1 := 0.85$		(AASHTO 8.16.2.7)
$\rho_b := \frac{0.85 \cdot \beta_1 \cdot f'_c}{f_y} \cdot \left(\frac{87ksi}{87ksi + f_y} \right)$	$\rho_b = 0.0214$	Balanced reinforcement ratio (AASHTO 8.16.3.2.2)
$0.75 \cdot \rho_b = 0.0160$	$0.75 \cdot \rho_b > \rho = 0.0018$	**OK**

MINIMUM REINFORCEMENT (AASHTO 8.17)

$A_{sREQ} = 0.28\, in^2$		Required area of steel for flexural strength
$A_s = 0.60\, in^2$		Area of steel provided
$\frac{4}{3} \cdot A_{sREQ} = 0.37\, in^2$	$0.39 in^2 < 0.60 in^2$	

Therefore the minimum reinforcement requirement is satisfied. **OK**

SHEAR CAPACITY (AASHTO 8.16.6)

$\phi V_n > V_u$

$\phi V_n = \phi V_c + \phi V_s$

$V_c := \left(2 \cdot \sqrt{f'_c \cdot psi} \cdot b \cdot d\right)$ $V_c = 36.97\,kip$ Concrete shear strength (AASHTO 8.16.6.2.3)

$\phi_v \cdot V_c = 31.43\,kip$ Design concrete shear strength

$31.43kip > 11.37kip$ **OK**

Stirrups not required for strength. Must check minimum reinforcement requirement.

Minimum stirrups required when $\frac{\phi V_c}{2} < V_u$ (AASHTO 8.19.1.1)

$\frac{\phi_v \cdot V_c}{2} = 15.71\,kip$ $15.71kip > 11.37kip$ **OK**

Therefore, no shear reinforcement (i.e., stirrups) required.

County:
Project No:
Description:

computed by:
checked by:
date: 8/30/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

Instructions Worksheet

Return to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	40.00	ft
	2	Roadway width	24.00	ft
	3	Location of exterior pile relative to the edge of the roadway	0.75	ft
		Maximum number of piles	10	piles on 2.50 ft centers
		Minimum number of piles	4	piles on 7.50 ft centers
	4	Number of piles	7	
	5	Backwall height	6.00	ft
	6	Estimated scour depth	2.00	ft
	7	Superstructure system	PCDT	
		Estimated dead load abutment reaction	128.6	kip per abutment (default value)
	8	Dead load abutment reaction for this analysis	128.6	kip per abutment
		Estimated live load abutment reaction	110.0	kip per abutment (default value)
	9	Live load abutment reaction for this analysis	110.0	kip per abutment
Foundation Material Input	10	Soil SPT blow count (N)	20	
		Correlated soil friction angle (ϕ)	33.3	degrees
	11	Soil friction angle for this analysis	33.3	degrees
	12	Estimated friction bearing value for depths less than 30 ft	0.7	tons per ft
	13	Estimated friction bearing value for depths greater than 30 ft	0.7	tons per ft
Pile Input	14	Timber species	southern pine	
	15	Tabulated timber bending stress	1,750	psi
	16	Tabulated timber compressive stress	1,100	psi
	17	Tabulated timber modulus of elasticity	1,600,000	psi
	18	Pile butt diameter	13.0	in.
	19	Pile tip diameter	10.0	in.
Lateral Restraint Input	20	Superstructure bearing elevation	3.58	ft
	21	Type of lateral restraint system	buried concrete anchor block	
	22	Anchor rod steel yield stress	60	ksi
	23	Total number of anchor rods per abutment	5	per abutment
	24	Anchor rod diameter	0.75	in.
	25	Height of anchor block	3.00	ft
	26	Bottom elevation of anchor block	1.08	ft
		Anchor block lateral capacity	10.3	kip per pile
		Computed anchor force per pile	7.6	kip per pile
		Minimum anchor rod length	13.47	ft
	27	Anchor rod length	15.00	ft

Check Pile Design

County:
Project No:
Description:

computed by:
checked by:
date: 8/30/2004

THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

Design Checks	1	Axial pile load	$P \le P_{ALLOWABLE}$	48.0 kip	OK
	2	Pile length	$Length \le 55\ ft$	37 ft	OK
	3	Pile bearing capacity	$Axial\ Pile\ Load \le Capacity$	sufficient if pile is embedded at least	34 ft
	4	Interaction equation validation	$\frac{1}{(1-f_C/F'_e)} > 1.0$	1.04	OK
	5	Combined loading interaction requirement $\left(\frac{f_C}{F'_C}\right)^2 + \frac{f_{bx}}{F'_b\left(1-\frac{f_C}{F'_{ex}}\right)} + \frac{f_{by}}{F'_b\left(1-\frac{f_C}{F'_{ey}}-\left(\frac{f_{bx}}{F'_{bE}}\right)^2\right)} \le 1.0$		0.75	OK
	6	Buried anchor block location	$Anchor\ rod\ length \ge minimum$	15.00 ft	OK
	7	Anchor rod stress	$\sigma \le 0.55\ F_Y$	24.2 ksi	OK
	8	Anchor block capacity	$Total\ Anchor\ Force \le Capacity$	10.3 kip per pile	OK
	9	Maximum displacement	$d_{MAX} \le 1.5\,in.$	0.38 in.	OK

Anchor Design Worksheet

Foundation Summary	1	Roadway width	24.00 ft
	2	Span length	40.00 ft
	3	Distance between superstructure bearings and roadway grade	2.42 ft
	4	Backwall height	6.00 ft
	5	Dead load abutment reaction	128.6 kip per abutment
	6	Live load abutment reaction	110.0 kip per abutment
	7	Number of piles	7
	8	Total axial pile load	24.0 tons
	9	Pile spacing	3.75 ft
	10	Pile size	
		Butt diameter	13.0 in.
		Tip diameter	10.0 in.
	11	Pile material properties	
		Timber species	southern pine
		Tabulated timber compressive stress	1,100 psi
		Tabulated timber bending stress	1,750 psi
		Tabulated timber modulus of elasticity	1,600,000 psi
	12	Minimum total pile length	37 ft

County:
Project No:
Description:

computed by:
checked by:
date: 8/27/2004

THIS WORKSHEET IS ONLY TO BE USED AFTER THE PILE SYSTEM HAS BEEN DESIGNED AND ALL DESIGN REQUIREMENTS HAVE BEEN SATISFIED.

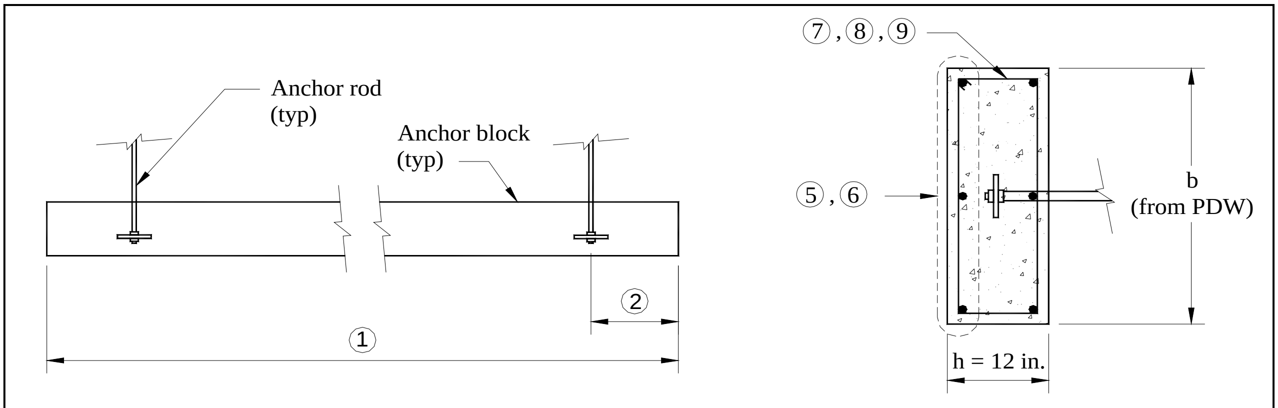
Return to Pile Design Worksheet

Go to Pile and Soil Selection Worksheet

The design in this worksheet is based on Section 8 of the AASHTO Standard Specifications.

Once the instructions on this sheet have been reviewed, proceed to the input section of this worksheet below.

Data required is to be entered in the highlighted cells of the Input Information section; all circled numbers are shown on the figure below.



Anchor block plan view.

Anchor block cross section.

Instructions	Cell No.	Description
	1	Enter the total length of the anchor block.
	2	Enter the distance between the end of the anchor block and the exterior anchor rod.
	3	Enter the anchor block concrete compressive strength.
	4	Use the pull-down menu provided to select the yield strength of the reinforcing steel.
	5	Enter the number of tension steel reinforcing bars on one vertical anchor block face.
	6	Use the pull-down menu provided to select the tension steel bar size.
	7	If applicable, use the pull-down menu provided to select the stirrup bar size.
	8	If applicable, enter the number of stirrup legs per section.
	9	If applicable, enter the stirrup spacing for this analysis. This value must be less than the value in the cell directly above this input cell.

County:
Project No:
Description:

computed by:
checked by:
date: 8/30/2004

THIS WORKSHEET IS ONLY TO BE USED AFTER THE PILE SYSTEM HAS BEEN DESIGNED AND ALL DESIGN REQUIREMENTS HAVE BEEN SATISFIED.

Input Information	1	Anchor block length	27.00	ft
	2	Distance from end of anchor block to exterior anchor rod	1.50	ft
	3	Concrete compressive strength	3.0	ksi
	4	Yield strength of reinforcing steel	60	ksi
		Tension steel area required	0.28	in^2
	5	Number of reinforcing bars per vertical anchor block face	3	bars
	6	Tension steel bar size	4	#
		Tension steel area provided	0.60	in^2
		Are stirrups required?	No	

Design Checks	1	Design flexural capacity	$\boxed{M_U < \phi M_N}$	24.78	ft-kips	OK	{AASHTO 8.16.3.2}
	2	Reinforcement ratio	$\boxed{\rho < 0.75\rho_b}$	0.0018		OK	{AASHTO 8.16.3.2.2}
	3	Minimum reinforcement				OK	{AASHTO 8.17}
	4	Design shear capacity	$\boxed{V_U < \phi V_N}$	31.4	kip	OK	{AASHTO 8.16.6.1.1}

Anchor System Summary	1	Number of anchor rods	5	
	2	Anchor rod steel yield stress	60	ksi
	3	Anchor rod diameter	0.750	in.
	4	Anchor rod length	15.00	ft
	5	Anchor rod spacing	6.00	ft
	6	Vertical distance between bottom of anchor block and roadway grade	4.92	ft
	7	Anchor block length	27.00	ft
	8	Anchor block height	3.0	ft
	9	Anchor block width	12.0	in.
	10	Concrete compressive strength	3.0	ksi
	11	Details of reinforcement on one vertical anchor block face	3	# 4 bars

EXAMPLE 2

STEEL PILE ABUTMENT WITHOUT ANCHORS IN A COHESSIVE SOIL

<u>EXAMPLE 2</u>: STEEL PILE ABUTMENT WITHOUT AN ANCHOR IN A COHESIVE SOIL

BRIDGE INFORMATION

Prestressed concrete girder superstructure (see Iowa DOT H24S-87 County standards)

$SPAN := 60ft$		Span length
$RDWY := 24ft$		Roadway width
$BW := 6ft$		Backwall height
$ES := 2ft$		Estimated depth of scour below stream elevation

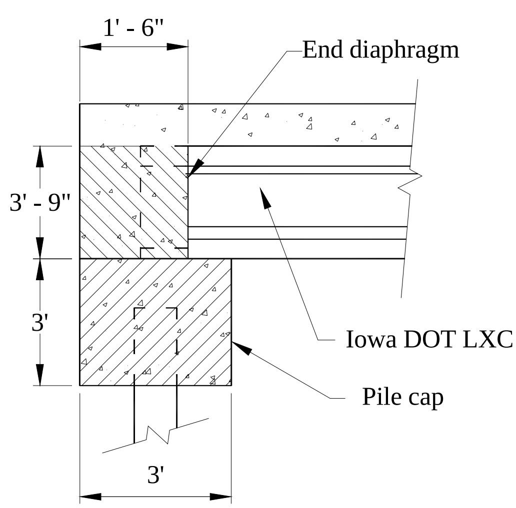
Slab depth = 8in

$LXC := 3.75ft$		Iowa DOT LXC girder depth
$Z_b := BW - 8in - LXC$	$Z_b = 1.583\,ft$	Distance between stream and bearing elevation
$SPT := 11$		Standard penetration test blow count for a soil best described as a firm glacial clay in the Iowa DOT FSIC
$FB_1 := 0.7\cdot\frac{ton}{ft}$		Pile friction bearing resistance for soil within 30 ft of the natural ground line (Iowa DOT FSIC)
$FB_2 := 0.80\cdot\frac{ton}{ft}$		Pile friction bearing resistance for soil not within 30 ft of the natural ground line (Iowa DOT FSIC)
$DB := 40ft$		Depth to bedrock below stream elevation
$N_{rock} := 150$		End bearing SPT blow count
$NA := 2$		Number of abutments

GRAVITY LOADS

Dead Loads

$GL := SPAN + 2\cdot(6in)$	$GL = 61.00\,ft$	Girder length
$BL := GL + 2\cdot(6in)$	$BL = 62.00\,ft$	Bridge length
$G := 447.0plf$		Iowa DOT LXC girder weight per foot
$N_G := 4$		Number of girders
$BR := 50plf$		Conservatively assumed thrie-beam weight per ft
$P_{sw} := 0.042klf$		HP10 x 42 wt. per foot
$FWS := 20psf$		Assumed future wearing surface
$\gamma_c := 0.150kcf$		Concrete unit weight
$Slab := (8in)\cdot BL\cdot RDWY\cdot\gamma_c$	$Slab = 148.80\,kip$	Calculated slab weight
$Girder := N_G\cdot G\cdot GL$	$Girder = 109.07\,kip$	Calculated girder weight
$Rail := 2\cdot BR\cdot BL$	$Rail = 6.20\,kip$	Calculated barrier rail weight
$FWS_{wt} := FWS\cdot RDWY\cdot BL$	$FWS_{wt} = 29.76\,kip$	Calculated future wearing surface weight



Abutment Cross Section

$\text{Diaphragm} := (18in) \cdot LXC \cdot RDWY \cdot \gamma_c \cdot NA$ — $\text{Diaphragm} = 40.50\,kip$ — Calculated end diaphragm weight (for conservative weight calculations only)

$\text{Cap} := (3ft) \cdot (3ft) \cdot RDWY \cdot \gamma_c \cdot NA$ — $\text{Cap} = 64.80\,kip$ — Calculated pile cap wt.

$\text{Wale} := 2 \cdot (20plf) \cdot RDWY \cdot NA$ — $\text{Wale} = 1.92\,kip$ — Calculated abutment wale weight (2, 20 plf wales per abutment)

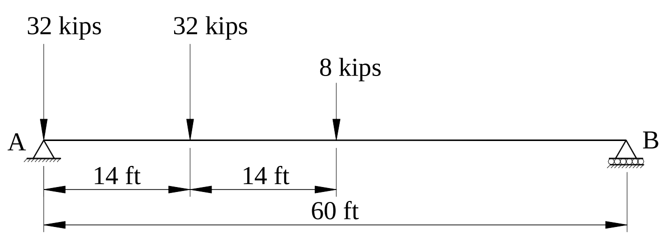
$$DL_{gb} := \text{Slab} + \text{Girder} + \text{Rail} + FWS_{wt} + \text{Diaphragm} + \text{Cap} + \text{Wale}$$

$DL_{gb} = 401.05\,kip$ — Bridge dead load

$DL_g := \dfrac{DL_{gb}}{NA} \cdot 1.05$ — $DL_g = 210.55\,kip$ — Dead load abutment reaction (increased by 5% because standards for nonspecific bridges were used)

Live Load

AASHTO HS20-44 design truck (AASHTO 3.7)



$$\Sigma M_B = 0 = (8kip) \cdot (60ft - 28ft) + (32kip) \cdot (60 \cdot ft - 14 \cdot ft) + (32kip) \cdot (60 \cdot ft) - R_A \cdot (60ft)$$

$$R_A := \frac{8kip \cdot (60ft - 28ft) + 32kip \cdot (60ft - 14ft) + (32kip) \cdot 60ft}{60ft} \qquad R_A = 60.80\,kip \uparrow$$

For 1 traffic lane, maximum live load abutment reaction = 60.80 kips.

$\dfrac{RDWY}{10ft} = 2.4$ — Number of 10 ft design traffic lanes (AASHTO 3.6.1)

$LN := 2$ Round down to 2 traffic lanes

Therefore, no lane reduction factor needed. (AASHTO 3.12.1)

$LL_g := LN \cdot R_A$ $LL_g = 121.60\ kip$ Calculated live load abutment reaction

$N := 8$ Assume 8 piles

$pf := 1.30$ Nominal axial pile factor (Volume 2, Chapter 2)

$P_{swt} := P_{sw} \cdot \left(DB + Z_b + ES\right)$ $P_{swt} = 1.83\ kip$ (per pile)

$P_{DL} := \frac{DL_g}{N} \cdot pf + P_{swt}$ $P_{DL} = 36.04\ kip$ Pile axial dead load

$P_{LL} := \frac{LL_g \cdot pf}{N}$ $P_{LL} = 19.76\ kip$ Pile axial live load

$P_T := P_{DL} + P_{LL}$ $P_T = 55.80\ kip$ Total axial load

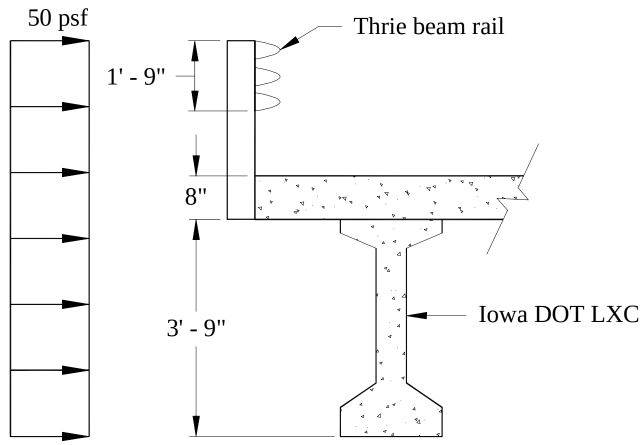
$S := \frac{RDWY - 2 \cdot (0.9167ft)}{(N - 1)}$ $S = 3.167\ ft$ Pile spacing with 11 in. between edge of roadway and first exterior pile

LATERAL LOADS

Transverse Loads

Transverse wind loads are assumed to be divided equally among all piles and are transferred through shear at the bridge bearings.

WIND ON SUPERSTRUCTURE



(Iowa DOT BDM 6.6.2.6.1)

$EA := (1.75ft + 8in + LXC) \cdot SPAN$ $\quad EA = 370.00\,ft^2$ $\quad$ Bridge superstructure elevation surface area

$WS := \dfrac{EA \cdot (50psf)}{NA \cdot N}$ $\quad WS = 1.16\,kip$ $\quad$ Wind on superstructure force per pile

WIND ON LIVE LOAD

$LL_w := 100plf$ $\quad$ Line load applied to entire bridge length (Iowa DOT BDM 6.6.2.6.2)

$WL := LL_w \cdot \dfrac{SPAN}{(NA \cdot N)}$ $\quad WL = 0.38\,kip$ $\quad$ Wind on live load force per pile

Longitudinal Loads

BRAKING FORCE (Iowa DOT BDM 6.6.2.4)

5% of the AASHTO lane gravity loading multiplied by the number of 10 ft design lanes.

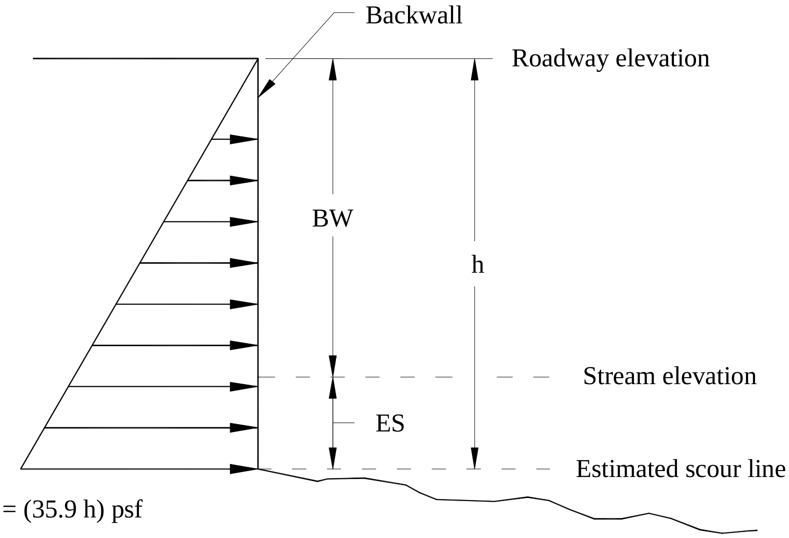
$W := 0.64\text{klf}$

$F := 18\text{kip}$

$BFP := \dfrac{LN\cdot(W\cdot SPAN + F)\cdot 0.05}{NA\cdot N}$ $\quad BFP = 0.35\,\text{kip}$ Braking force per pile

DEAD LOAD EARTH PRESSURE

(Iowa DOT BDM 6.5.2.5)



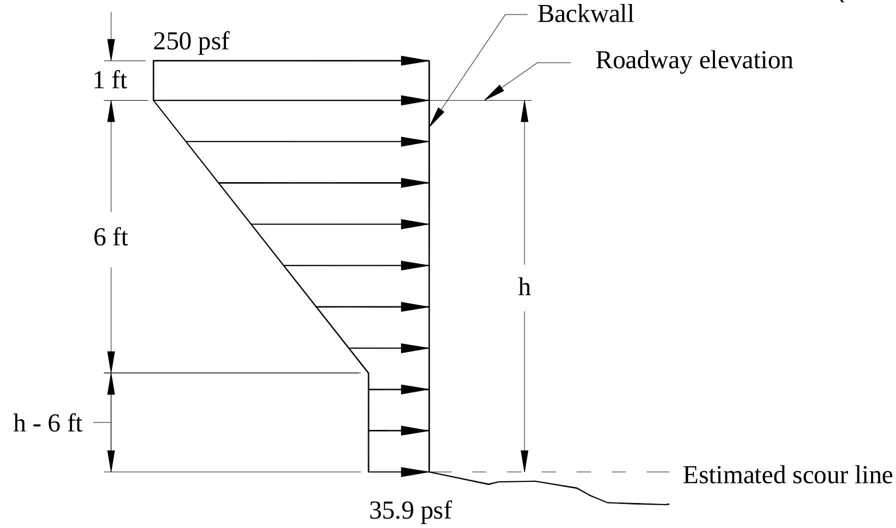
$h := BW + ES$ $\quad h = 8.00\,\text{ft}$

$P_1 := (35.9\text{pcf})\cdot h$ $\quad P_1 = 287.20\,\text{psf}$

$w_1 := P_1\cdot S$ $\quad w_1 = 0.909\,\text{klf}$ Convert P_1 to a distributed pile line load

$EDL := \dfrac{1}{2}\cdot w_1\cdot h$ $\quad EDL = 3.64\,\text{kip}$ Total lateral force per pile from active earth pressure

LIVE LOAD SURCHARGE (Iowa DOT BDM 6.5.2.5)



$w_2 := (250psf)\cdot S$ $\quad w_2 = 0.792\,klf$ Convert soil pressures into distributed loads

$w_3 := (35.9psf)\cdot S$ $\quad w_3 = 0.114\,klf$

$w_4 := w_2 - w_3$ $\quad w_4 = 0.678\,klf$

$LL_{sur} := (1ft)\cdot w_2 + \frac{1}{2}\cdot\left(w_4\right)\cdot BW + h\cdot w_3$ $\quad LL_{sur} = 3.74\,kip$ Total lateral force per pile from live load surcharge

DETERMINE DEPTH TO PILE FIXITY

$L_f = f + 1.5\cdot B$

$f = \dfrac{H}{9\cdot C_u\cdot B}$ For a cohesive soil (Broms, 1964)

$H := BFP + LL_{sur} + EDL$ $\quad H = 7.73\,kip$ Total lateral force per pile

$P_{ATM} := 14.69psi$ Atmospheric pressure

$C_u := 0.06\cdot SPT\cdot P_{ATM}$ $\quad C_u = 1396\,psf$ Undrained soil shear strength

$B := 10.1in$ HP 10x42 pile width

$f := \dfrac{H}{9\cdot C_u\cdot B}$ $\quad f = 8.8\,in$

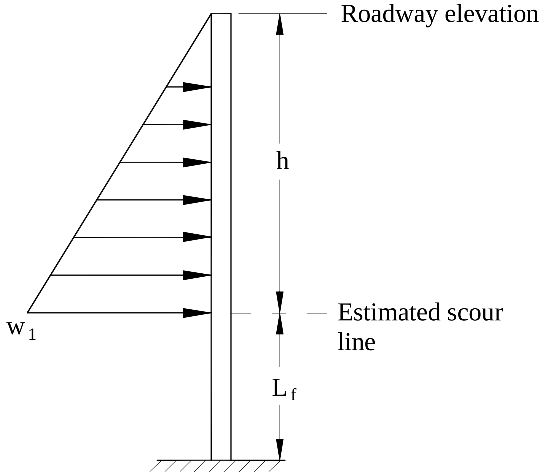
$L_f := f + 1.5 \cdot B$ $\qquad$ $L_f = 1.993\ ft$ $\qquad$ Depth below estimated scour line to pile fixity

DETERMINE MAXIMUM PILE MOMENT

For cantilever system, the maximum moment will occur at the point of fixity.

Longitudinal Bending Moment

EARTH DEAD LOAD



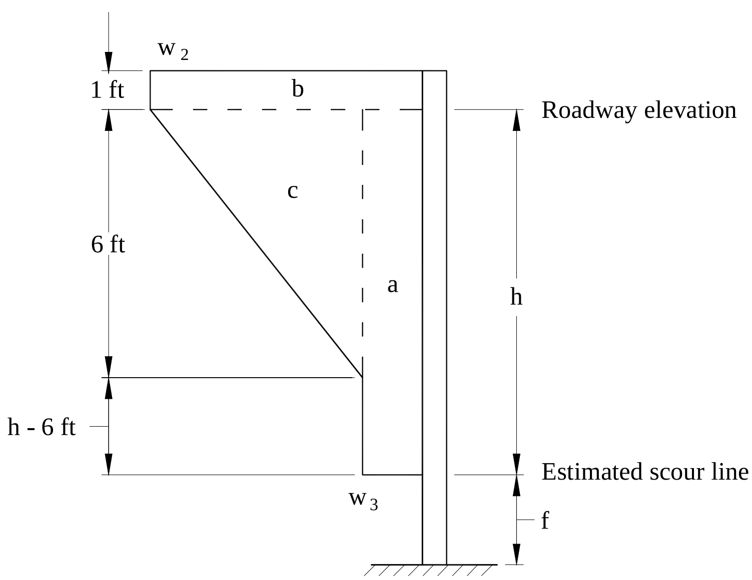
$h = 8.000\ ft$

$L_f = 1.993\ ft$

$w_1 = 0.909\ klf$

$$M_{EDL} := \frac{1}{2} \cdot h \cdot w_1 \cdot \left(L_f + \frac{1}{3} \cdot h\right)$$ $\qquad$ $M_{EDL} = 16.95\ ft \cdot kip$

LIVE LOAD SURCHARGE



$w_2 = 0.792\ klf$

$w_3 = 0.114\ klf$

$w_4 = 0.678\ klf$

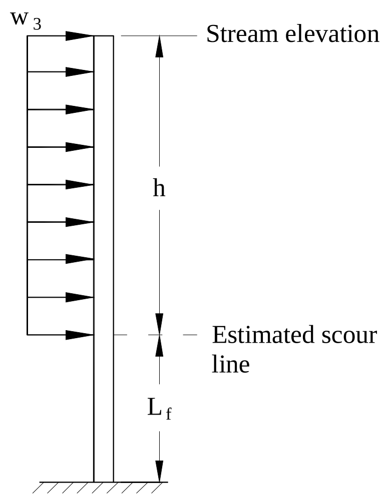
$L_f = 1.993\ ft$

$ES = 2.0\ ft$

$BW = 6.0\ ft$

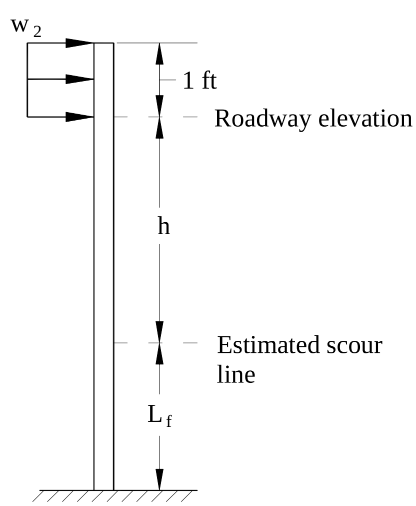
$h = 8.0\ ft$

Part a)



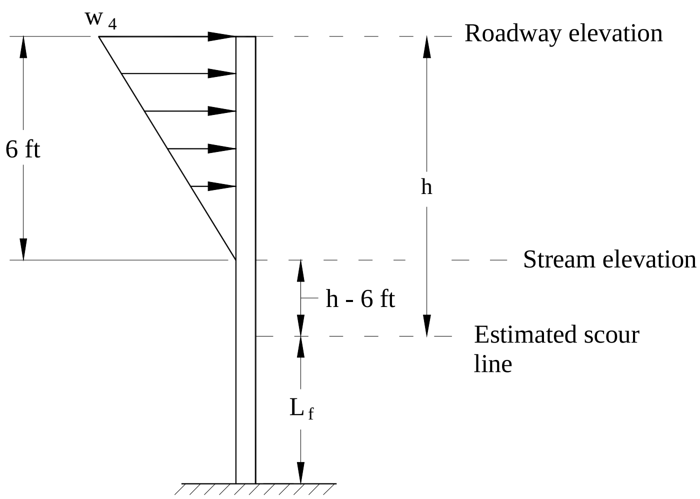
$$M_{LLA} := w_3 \cdot (h) \cdot \left(L_f + \frac{h}{2} \right) \qquad M_{LLA} = 5.45\,\text{ft} \cdot \text{kip}$$

Part b)



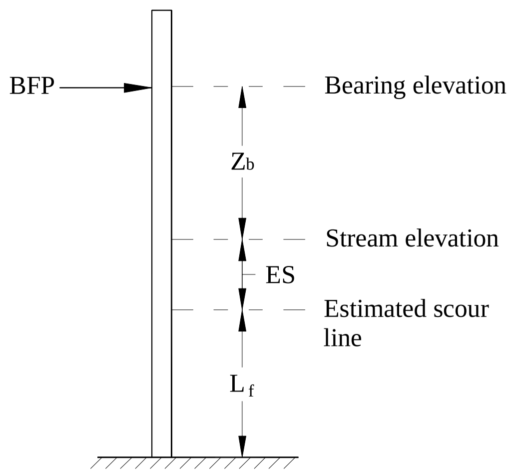
$$M_{LLB} := w_2 \cdot (1\text{ft}) \cdot \left(h + \frac{1\text{ft}}{2} + L_f \right) \qquad M_{LLB} = 8.31\,\text{ft} \cdot \text{kip}$$

Part c)



$$M_{LLC} := \frac{1}{2} \cdot w_4 \cdot (6ft) \cdot \left[h - 6ft + L_f + \left(\frac{2}{3}\right) \cdot 6ft \right] \qquad M_{LLC} = 16.26\, ft \cdot kip$$

BRAKING FORCE



$BFP = 0.35\, kip$

$L_f = 1.993\, ft$

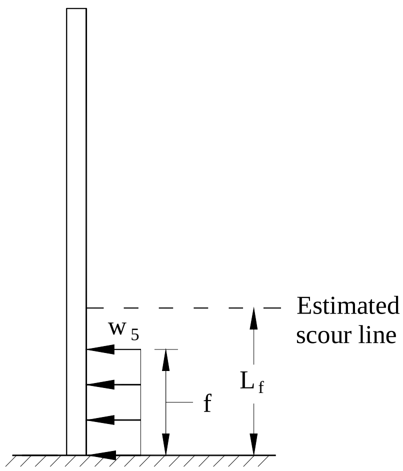
$ES = 2.0\, ft$

$Z_b = 1.583\, ft$

$x_4 := L_f + ES + Z_b \qquad x_4 = 5.576\, ft$ Distance between pile fixity and bearing elevation

$M_{BF} := BFP \cdot x_4 \qquad M_{BF} = 1.97\, ft \cdot kip$

PASSIVE EARTH PRESSURE



$f = 0.730\ ft$

$L_f = 1.993\ ft$

$C_u = 1396\ psf$

$B = 10.1\ in$

$w_5 := 9 \cdot C_u \cdot B$ $\quad w_5 = 10.576\ klf$ — Passive soil pressure

$$M_{PE} := \frac{-w_5 \cdot f^2}{2}$$ $\quad M_{PE} = -2.82\ ft \cdot kip$

$M := M_{EDL} + M_{LLA} + M_{LLB} + M_{LLC} + M_{BF} + M_{PE}$

$M = 46.11\ ft \cdot kip$ — **Total longitudinal pile moment**

Transverse Pile Moments

$WS = 1.16\ kip$ — Wind on superstructure force per pile

$WL = 0.38\ kip$ — Wind on live load force per pile

$M_{WS} := WS \cdot (L_f + ES + Z_b)$ $\quad M_{WS} = 6.45\ ft \cdot kip$ — Wind on superstructure transverse pile moment

$M_{WL} := WL \cdot (L_f + ES + Z_b)$ $\quad M_{WL} = 2.09\ ft \cdot kip$ — Wind on live load transvers pile moment

Load Summary

$P_{DL} = 36.04\ kip$ — Pile axial dead load

$P_{LL} = 19.760\ kip$ — Pile axial live load

$P_T := P_{DL} + P_{LL}$ $\quad P_T = 55.80\ kip$ — Pile total axial load

DESIGN CHECKS

Allowable Axial Pile Stress

For combination friction and end bearing piles, the maximum allowable axial stress = 9 ksi for steel piles seated in bedrock with an estimated SPT blow count between 100 and 200.

(Iowa DOT BDM 6.2.6.1)

$A := 12.4\mathrm{in}^2$ HP10 x 42 area

$fa := \frac{P_T}{A}$ $fa = 4.50\,\mathrm{ksi}$ Total axial pile stress

$4.50\mathrm{ksi} < 9\mathrm{ksi}$ **OK**

Pile Bearing Capacity

Allowable end bearing stress = 9ksi (Iowa DOT FSIC)

Maximum pile load = (9ksi)* A = 111.6kips/pile

$111.6\,\frac{\mathrm{kips}}{\mathrm{pile}} > 55.80\cdot\frac{\mathrm{kips}}{\mathrm{pile}}$ **OK**

Combined Axial and Lateral Loading Check

Two interaction equations are cited in AASHTO.

$$\frac{f_a}{F_a} + \frac{C_{mx}\cdot\left(f_{bx}\right)}{F_{bx}\cdot\left(1 - \frac{f_a}{F'_{ex}}\right)} + \frac{C_{my}\cdot f_{by}}{F'_{by}\cdot\left(1 - \frac{f_c}{F_{cEy}}\right)} \le 1.0$$

(AASHTO 10.36})

$$\frac{f_a}{0.472\cdot F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0$$

(AASHTO 10.36)

Note: The x-axis for the pile is assumed to be parallel to the backwall face. Additionally, the Iowa DOT specifies three group loading combinations that apply to this application. (Iowa DOT BDM 6.6.3.1)

For the given loads, three different load combinations given in Section 6.6.3.1 of the Iowa DOT BDM are applicable.

Group I: 1.0(DL)+1.0(LL)+1.0(E)+1.0(BF) using 100% of the allowable stress
Group II: 1.0(DL)+1.0(E)+1.0(WS) using 125% of the allowable stress
Group III: 1.0(DL)+1.0(LL)+1.0(E)+1.0(LF)+0.3(WS)+1.0(WL) using 125% of the allowable stress

DL = Dead load
LL = Live load
E = Earth load
BF = Longitudinal braking force
WS = Wind on superstructure
WL = Wind on live load

$P_{LL} = 19.76\ \text{kip}$ Axial pile live load

$P_{DL} = 36.04\ \text{kip}$ Axial pile dead load

$P_T = 55.80\ \text{kip}$ Axial pile total load

$A = 12.40\ \text{in}^2$ Pile area

GROUP I AND III (with live load)

$f_a := \frac{P_T}{A}$ $f_a = 4.50\ \text{ksi}$ Group I and III axial compressive stress

GROUP II (without live load)

$f_{aDL} := \frac{P_{DL}}{A}$ $f_{aDL} = 2.91\ \text{ksi}$ Group II axial compressive stress

When computing to applied x-axis applied bending stress, the live and dead loads were not separated. Therefore, the live load surcharge and braking force is also included in the second load combination recommended by the Iowa DOT BDM (i.e., Group II) and f_{bx} is the same for all load combinations. However, the pile axial live and dead load can be separated.

Pile properties

$F_y := 36ksi$		Pile steel yield stress
$E := 29000ksi$		Modulus of elasticity
$d := 9.7in$		HP10 x 47 depth
$t_f := 0.420in$		Flange thickness
$B := 10.1in$		Pile width
$D := d - 2 \cdot t_f$	$D = 8.860\,in$	Depth of web
$t_w := 0.415in$		Web thickness
$I := 210in^4$		Pile moment of inertia
$SM_x := 43.4in^3$		Strong axis section modulus
$SM_y := 14.2in^3$		Weak axis section modulus
$r_x := 4.13in$		Strong axis radius of gyration
$r_y := 2.41in$		Weak axis radius of gyration
$M = 46.11\,ft \cdot kip$		Maximum pile moment (x-axis bending)
$M_{WS} = 6.45\,ft \cdot kip$		Wind on superstructure pile moment (y-axis bending)
$M_{WL} = 2.09\,ft \cdot kip$		Wind on live load pile moment (y-axis bending)
$f_{bx} := \frac{M}{SM_x}$	$f_{bx} = 12.75\,ksi$	Groups I, II, and III applied x-axis bending stress
$f_{byWS} := \frac{M_{WS}}{SM_y}$	$f_{byWS} = 5.45\,ksi$	Group II and III applied y-axis bending bending stress from wind on superstructure
$f_{byWL} := \frac{M_{WL}}{SM_y}$	$f_{byWL} = 1.77\,ksi$	Group III applied y-axis bending stress from wind on live load

$$F'_e = \frac{\pi \cdot E}{2.12\left(\frac{k \cdot l}{r}\right)^2}$$

If no lateral pile restraint is used, consider the pile cap a braced point for y-axis bending.

$L_f = 1.993\,\text{ft}$ $\quad$ $h = 8.0\,\text{ft}$

$ES = 2.0\,\text{ft}$ $\quad$ $Z_b = 1.583\,\text{ft}$

$k_x := 2.0$

$k_y := 0.7$

$l_x := L_f + h$ $\quad$ $l_x = 9.993\,\text{ft}$ $\quad$ Distance between point of fixity and roadway elevation

$l_y := L_f + ES + Z_b$ $\quad$ $l_y = 5.576\,\text{ft}$ $\quad$ Distance between point of fixity and pile cap

$SR_x := \frac{k_x \cdot l_x}{r_x}$ $\quad$ $SR_x = 58.07$ $\quad$ x-axis slenderness ratio

$SR_y := \frac{k_y \cdot l_y}{r_y}$ $\quad$ $SR_y = 19.44$ $\quad$ y-axis slenderness ratio

$F'_{ex} := \frac{\pi^2 \cdot E}{2.12 SR_x^{\ 2}}$ $\quad$ $F'_{ex} = 40.04\,\text{ksi}$ $\quad$ x-axis buckling stress

$F'_{ey} := \frac{\pi^2 \cdot E}{2.12 \cdot SR_y^{\ 2}}$ $\quad$ $F'_{ey} = 357.39\,\text{ksi}$ $\quad$ y-axis buckling stress

Interaction Equation Validation Check

To account for secondary moment effects, a P-Δ factor is used. These values must be greater than 1.0.

X-AXIS BENDING

Group I and Group III Loading (with Live Load)

$$P\Delta_{x1} := \frac{1}{1 - \frac{f_a}{F'_{ex}}}$$

$P\Delta_{x1} = 1.13$

$1.13 > 1.0$ **OK**

Group II Loading (without Live Load)

$$P\Delta_{x2} := \frac{1}{\left(1 - \frac{f_{aDL}}{F'_{ex}}\right)}$$

$P\Delta_{x2} = 1.08$

$1.08 > 1.0$ **OK**

Y-AXIS BENDING

Group I and III Loading (with Live Load)

$$P\Delta_{y1} := \frac{1}{1 - \left(\frac{f_a}{F'_{ey}}\right)}$$

$P\Delta_{y1} = 1.01$

$1.01 > 1.0$ **OK**

Group II loading

$$P\Delta_{y2} := \frac{1}{1 - \left(\frac{f_{aDL}}{F'_{ey}}\right)}$$

$P\Delta_{y2} = 1.01$

$1.01 > 1.0$ **OK**

$C_{mx} := 0.85$

$C_{my} := 0.85$

For beam-columns with transverse loading (AASHTO Table 10.36A)

Allowable Compressive Stress (AASHTO Table 10.32.1A)

$$C_c := \sqrt{\frac{2\cdot\pi^2\cdot E}{F_y}}$$ $C_c = 126.1$ Column buckling coefficient

$SR_{max} := 58.973$ Max. slenderness ratio

$58.973 \le 126.1$ Therefore the following equation is used to determine the allowable compressive stress.

$$F_a := \frac{F_y}{2.12}\cdot\left(1 - \frac{SR_{max}^2\cdot F_y}{4\cdot\pi^2\cdot E}\right)$$ $F_a = 15.12\ \text{ksi}$ Allowable compressive stress

Allowable Bending Stress (AASHTO Table 10.32.1A)

$$F_b = \frac{50\cdot 10^6\cdot C_b}{SM_x}\cdot\frac{I_{yc}}{\zeta}\cdot\sqrt{0.772\cdot\frac{J}{I_{yc}} + 9.87\cdot\left(\frac{d}{\zeta}\right)^2} < 0.55\cdot F_y$$ Allowable bending stress

$C_b := 1.0$ Conservatively assumed bending coefficient

$$I_{yc} := \frac{1}{12}t_f\cdot B^3$$ $I_{yc} = 36.06\ \text{in}^4$ Moment of inertia for compression flange about vertical axis in the plane of the web

$\zeta := L_f + ES + Z_b$ $\zeta = 66.92\ \text{in}$ Length of unsuported flange (distance between pile fixity and bearing elevation

$$J := \frac{\left(2\cdot B\cdot t_f^3 + D\cdot t_w^3\right)}{3}$$ $J = 0.710\ \text{in}^4$ Pile torsional constant

$d = 9.70\ \text{in}$ Pile depth

$$F_b := \frac{50\cdot 10^6\cdot C_b}{SM_x}\cdot\frac{I_{yc}}{\zeta}\cdot\sqrt{0.772\cdot\frac{J}{I_{yc}} + 9.87\cdot\left(\frac{d}{\zeta}\right)^2}$$ $F_b = 292916$ psi = 292.9 ksi

$0.55 \cdot F_y = 19.80\ \text{ksi}$ **controls**

$F_b := 0.55 \cdot F_y$ **Allowable bending stress**

Interaction Equation #1

GROUP I INTERACTION LOADING

$f_a = 4.50\ \text{ksi}$ Applied total axial stress

$F_a = 15.12\ \text{ksi}$ Allowable axial stress

$f_{bx} = 12.75\ \text{ksi}$ Applied x-axis bending stress

$F_b = 19.80\ \text{ksi}$ Allowable bending stress

$P\Delta_{x1} = 1.13$ x-axis secondary moment factor

$$\frac{f_a}{F_a} + \frac{C_{mx} \cdot f_{bx} \cdot P\Delta_{x1}}{F_b} = 0.91 \qquad 0.91 \le 1.0$$ **OK**

GROUP II INTERACTION LOADING

$f_{aDL} = 2.91\ \text{ksi}$ Applied dead load axial stress

$P\Delta_{x2} = 1.08$ x-axis secondary moment factor for dead load stress

$f_{byWS} = 5.45\ \text{ksi}$ Applied y-axis bending stress from wind on superstructure

$P\Delta_{y2} = 1.01$ y-axis secondary moment factor for dead load stress

$$\left(\frac{f_{aDL}}{F_a} + \frac{C_{mx} \cdot f_{bx} \cdot P\Delta_{x2}}{F_b} + \frac{C_{my} \cdot f_{byWS} \cdot P\Delta_{y2}}{F_b}\right) \cdot \left(\frac{1}{1.25}\right) = 0.81$$ 1.25 allowable overstress factor

$0.81 \le 1.0$ **OK**

GROUP III INTERACTION LOADING

$P\Delta_{x1} = 1.13$ — x-axis secondary moment factor

$f_{byWL} = 1.767\ \text{ksi}$ — Applied y-axis bending stress from wind on live load

$P\Delta_{y1} = 1.013$ — y-axis secondary moment factor

$$\left(\frac{f_a}{F_a} + \frac{C_{mx} \cdot f_{bx} \cdot P\Delta_{x1}}{F_b} + \frac{0.3 \cdot C_{mx} \cdot f_{byWS} \cdot P\Delta_{y1}}{F_b} + \frac{C_{my} \cdot f_{byWL} \cdot P\Delta_{y1}}{F_b}\right) \cdot \left(\frac{1}{1.25}\right) = 0.850$$

1.25 allowable overstress factor

$0.85 \le 1.0$ **OK**

Interaction Equation #2

GROUP I LOADING

$$\frac{f_a}{0.472 \cdot F_y} + \frac{f_{bx}}{F_b} = 0.91$$

$0.91 \le 1.0$ **OK**

GROUP II LOADING

$$\left(\frac{f_{aDL}}{0.472 \cdot F_y} + \frac{f_{bx}}{F_b} + \frac{f_{byWS}}{F_b}\right) \cdot \left(\frac{1}{1.25}\right) = 0.87$$

$0.87 \le 1.0$ **OK**

GROUP III LOADING

$$\left(\frac{f_a}{0.472 \cdot F_y} + \frac{f_{bx}}{F_b} + \frac{0.3 \cdot f_{byWS}}{F_b} + \frac{f_{byWL}}{F_b}\right) \cdot \left(\frac{1}{1.25}\right) = 0.86$$

$0.86 \le 1.0$ **OK**

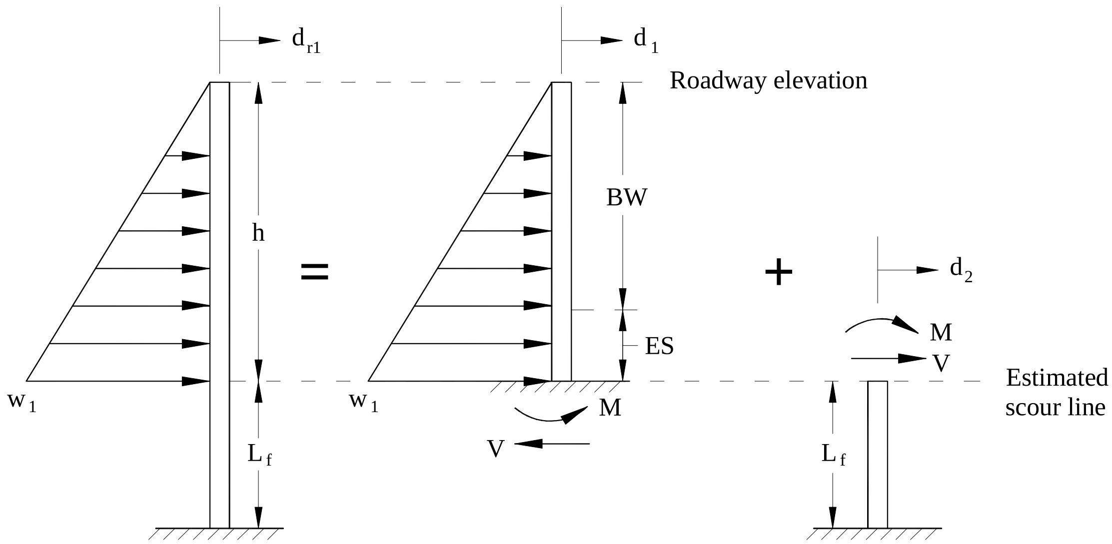
All interaction equations are less than or equal to 1.0 **OK**

Maximum Abutment Displacement

Maximum horizontal displacement = 1.5 in (AASHTO 4.4.7.2.5 via 4.5.12)

Must check displacement at roadway elevation

DEAD LOAD EARTH PRESSURE



$E = 29000\ \text{ksi}$ $I = 210.0\ \text{in}^4$

$w_1 = 0.909\ \text{klf}$ $L_f = 1.993\ \text{ft}$

$h = 8.00\ \text{ft}$ Distance between estimated scour line and roadway elevation

$d_1 := \dfrac{4w_1 \cdot h^4}{120 \cdot E \cdot I}$ $d_1 = 0.035\ \text{in}$ Pile deflection at roadway elevation

$M := \dfrac{1}{2} \cdot h \cdot w_1 \cdot \left(\dfrac{h}{3}\right)$ $M = 9.70\ \text{ft} \cdot \text{kip}$ Moment at estimated scour line

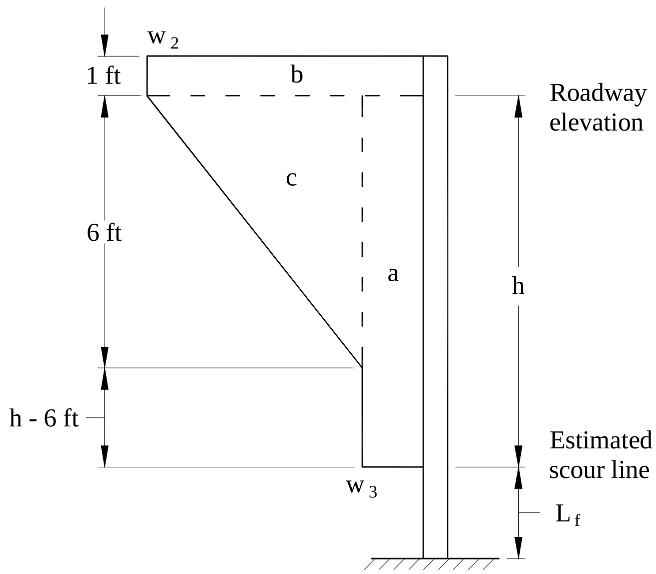
$V := \dfrac{1}{2} \cdot h \cdot w_1$ $V = 3.64\ \text{kip}$ Shear at estimated scour line

$d_2 := \dfrac{1}{E \cdot I}\left[\dfrac{\left(M \cdot L_f^{\,2}\right)}{2} + \dfrac{\left(V \cdot L_f^{\,3}\right)}{3}\right]$ $d_2 = 0.008\ \text{in}$ Pile deflection at estimated scour line

$$\theta := \frac{1}{E \cdot I}\left[M \cdot L_f + \frac{V \cdot \left(L_f^{\,2}\right)}{2}\right]$$ $\theta = 6.280 \times 10^{-4}\,\text{rad}$ Pile slope at estimated scour line

$$d_{r1} := \left(d_1 + d_2 + \theta \cdot h\right)$$ $d_{r1} = 0.104\,\text{in}$ Total pile deflection at roadway elevation from active earth pressure

LIVE LOAD SURCHARGE



$w_2 = 0.792\,\text{klf}$

$w_3 = 0.114\,\text{klf}$

$w_4 = 0.678\,\text{klf}$

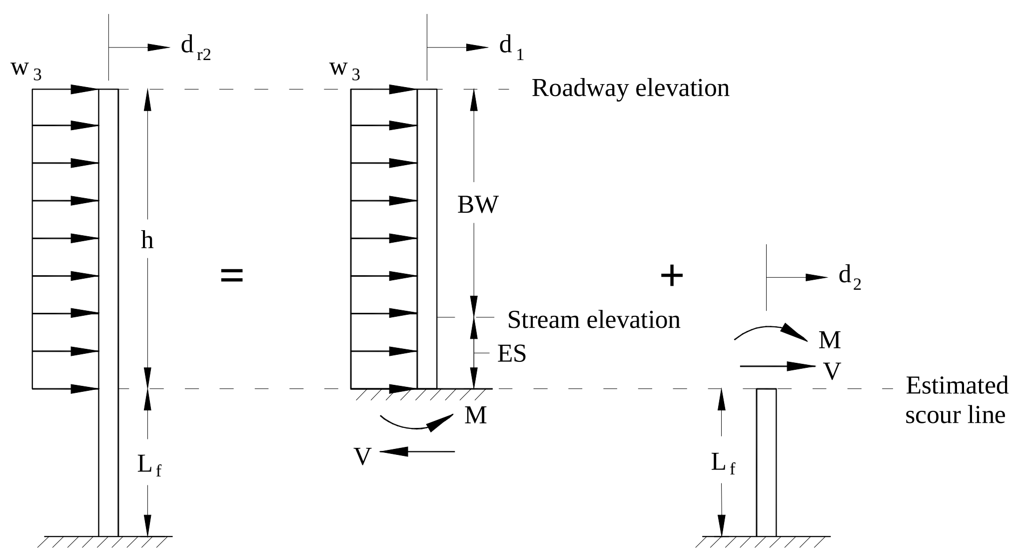
$L_f = 1.993\,\text{ft}$

$ES = 2.0\,\text{ft}$

$BW = 6.0\,\text{ft}$

$h = 8.00\,\text{ft}$

Part a)



$$d_1 := \frac{w_3 \cdot \left(h^4\right)}{8 \cdot E \cdot I}$$ $d_1 = 0.017\,\text{in}$ Pile deflection at roadway elevation

$$M := \frac{w_3 \cdot h^2}{2}$$ $M = 3.64\,\text{ft} \cdot \text{kip}$ Moment at estimated scour line

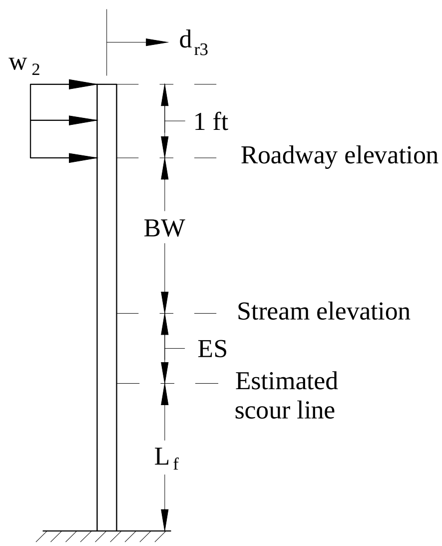
$$V := w_3 \cdot h$$ $V = 0.91\,\text{kip}$ Shear at estimated scour line

$$d_2 := \frac{1}{E \cdot I}\left[\frac{\left(M \cdot L_f^{\,2}\right)}{2} + \frac{\left(V \cdot L_f^{\,3}\right)}{3}\right]$$ $d_2 = 0.003\,\text{in}$ Pile deflection at estimated scour line

$$\theta := \frac{1}{E \cdot I}\left[(M) \cdot L_f + \frac{V \cdot L_f^{\,2}}{2}\right]$$ $\theta = 2.141 \times 10^{-4}\,\text{rad}$ Pile slope at estimated scour line

$$d_{r2} := d_1 + d_2 + \theta \cdot h$$ $d_{r2} = 0.040\,\text{in}$ Total pile deflection at roadway elevation from <u>Part a)</u> of live load surcharge

<u>Part b)</u>



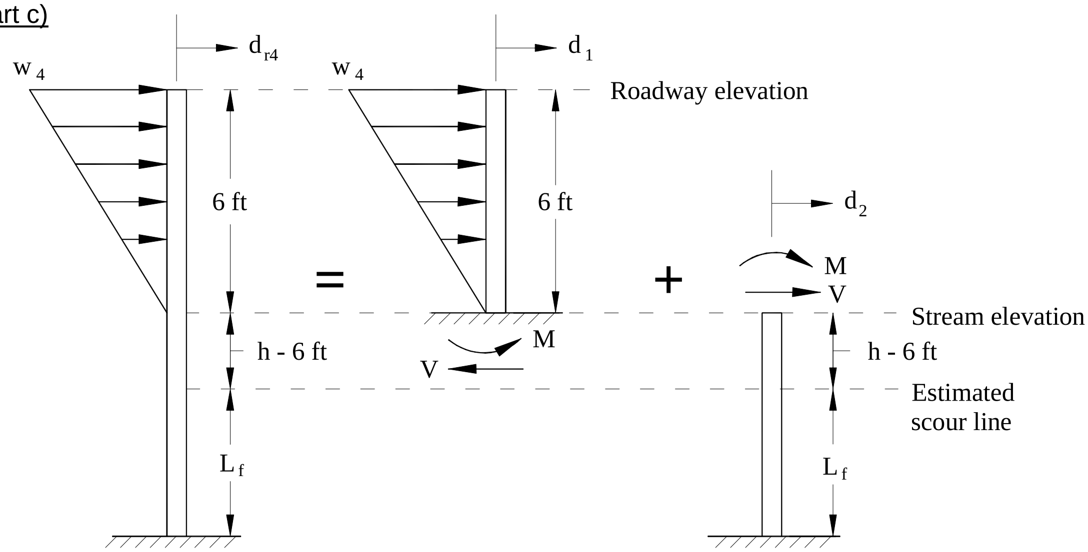
$$L := L_f + ES + BW + 1 \cdot \text{ft}$$ $L = 10.993\,\text{ft}$ Distance between point of fixity and 1 ft above roadway elevation

$x := L_f + ES + BW$ | $x = 9.993\ ft$ | Distance between roadway elevation and point of fixity

$$d_{r3} := \frac{w_2 \cdot 1 \cdot ft \cdot x^2}{2 \cdot E \cdot I} \cdot \left[\left(\frac{-1}{3}\right) \cdot x - \frac{1}{2} \cdot (1ft) + L\right]$$

$d_{r3} = 0.080\ in$ | Total pile deflection from Part b) of live load surcharge

Part c)



$$d_1 := \frac{w_4 \cdot (6ft)^2}{120 \cdot (6ft) \cdot E \cdot I} \cdot 11 \cdot (6ft)^3$$

$d_1 = 0.023\ in$ | Pile deflection at roadway elevation

$V := \frac{1}{2} \cdot w_4 \cdot (6ft)$ | $V = 2.03\ kip$ | Shear at stream elevation

$M := V \cdot \left(\frac{2}{3}\right) \cdot 6ft$ | $M = 8.14\ ft \cdot kip$ | Moment at stream elevation

$x := L_f + ES$ | $x = 3.993\ ft$ | Distance between point of pile fixity and roadway elevation

$$d_2 := \frac{1}{E \cdot I}\left(\frac{M \cdot x^2}{2} + \frac{V \cdot x^3}{3}\right)$$

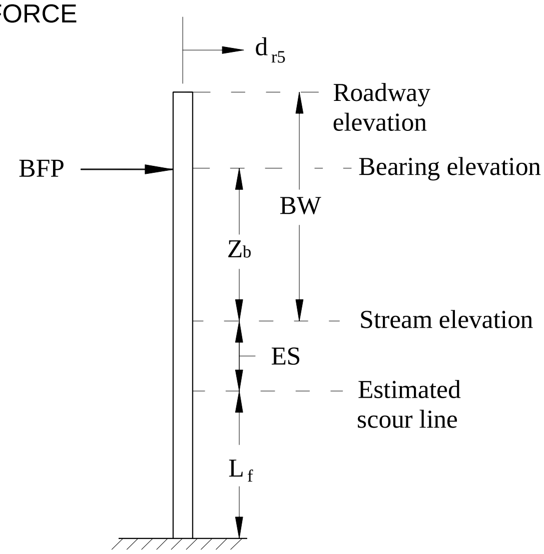
$d_2 = 0.031\ in$ | Pile deflection at stream elevation

$$\theta := \frac{1}{E \cdot I}\left[M \cdot (x) + \frac{V \cdot x^2}{2}\right]$$

$\theta = 1.152 \times 10^{-3}\ rad$ | Pile slope at stream elevation

$d_{r4} := d_1 + d_2 + \theta \cdot (6ft)$ | $d_{r4} = 0.136\ in$ | Total pile deflection from Part c) of live load surcharge

BRAKING FORCE



$BFP = 0.353\,kip$

$L_f = 1.993\,ft$

$ES = 2.0\,ft$

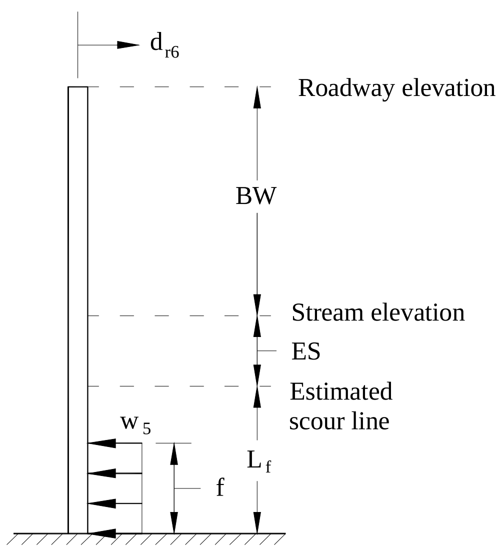
$BW = 6.0\,ft$

$Z_b = 1.583\,ft$

$h = 8.0\,ft$

$x_1 := L_f + ES + Z_b$	$x_1 = 5.576\,ft$	Distance between point of pile fixity and bearing elevation
$x_2 := h + L_f$	$x_2 = 9.993\,ft$	Distance between point of pile fixity and roadway elevation
$d_{r5} := \dfrac{BFP \cdot x_1^{\,2}}{6E \cdot I} \cdot (3 \cdot x_2 - x_1)$	$d_{r5} = 0.013\,in$	Total pile deflection from braking force

PASSIVE EARTH PRESSURE



$w_5 := 9 \cdot C_u \cdot B$ $w_5 = 10.576\,\text{klf}$

$d_{r6} := \frac{-w_5 \cdot f^3}{24 \cdot E \cdot I} \cdot \left[4 \cdot \left(L_f + h\right) - \left(L_f + h - f\right)\right]$ $d_{r6} = -0.001\,\text{in}$ Total pile deflection from passive soil reaction

$d_{rT} := d_{r1} + d_{r2} + d_{r3} + d_{r4} + d_{r5} + d_{r6}$ $d_{rT} = 0.371\,\text{in}$ Total pile deflection at roadway elevation

$0.371\text{in} \le 1.50\text{in}$ **OK**

County:
Project No:
Description:

computed by:
checked by:
date: 8/30/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

Instructions Worksheet

Go to Pile and Soil Selection Worksheet

General Bridge Input	1	Span length	60.00	ft		
	2	Roadway width	24.00	ft		
	3	Location of exterior pile relative to the edge of the roadway	0.92	ft		
		Maximum number of piles	9	piles on	2.77	ft centers
		Minimum number of piles	4	piles on	7.39	ft centers
	4	Number of piles	8			
	5	Backwall height	6.00	ft		
	6	Estimated scour depth	2.00	ft		
	7	Superstructure system	prestressed girder			
		Estimated dead load abutment reaction	210.7	kip per abutment (default value)		
	8	Dead load abutment reaction for this analysis	210.7	kip per abutment		
		Estimated live load abutment reaction	121.5	kip per abutment (default value)		
	9	Live load abutment reaction for this analysis	121.5	kip per abutment		
Foundation Material Input	10	Soil SPT blow count (N)	11			
		Correlated soil un-drained shear strength (C_u)	1,397	psf		
	11	Soil undrained shear strength for this analysis	1,397	psf		
	12	Type of vertical pile bearing resistance	friction & end bearing			
	13	Estimated friction bearing value for depths < 30 ft	0.7	tons per ft		
	14	Estimated friction bearing value for depths > 30 ft	0.8	tons per ft		
	15	Depth to adequate end bearing foundation material	40	ft		
	16	SPT blow count for end bearing foundation material	100 < N < 200			
Pile Input	17	Pile steel yield stress	36	ksi		
	18	Select pile type	HP10x42			
	19	Pile cross sectional area	12.4	in^2		
	20	Pile depth	9.70	in.		
	21	Pile web thickness	0.415	in.		
	22	Pile flange width	10.1	in.		
	23	Pile flange thickness	0.420	in.		
	24	Pile moment of inertia (strong axis)	210	in^4		
	25	Pile section modulus (strong axis)	43.4	in^3		
	26	Pile section modulus (weak axis)	14.2	in^3		
	27	Pile radius of gyration (strong axis)	4.13	in.		
	28	Pile radius of gyration (weak axis)	2.41	in.		
Lateral Restraint Input	29	Superstructure bearing elevation	1.58	ft		
	30	Type of lateral restraint system	no lateral restraint system			
	31					
	32					
	33					
	34					
	35					
	36					

Check Pile Design

County:
Project No:
Description:

computed by:
checked by:
date: 8/30/2004

THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.

Geotechnical, Structural and Serviceability Requirements

Design Checks					
	1	Axial pile stress	$P/A \le \sigma_{ALL}$	4.49 ksi	OK
	2	Pile bearing capacity	Axial Pile Load ≤ Capacity	111.6 kip	OK
	3	Interaction equation validation	$\frac{1}{(1-f_a/F'_e)} > 1.0$	1.13	OK
	4	Combined loading interaction requirement # 1	$\frac{f_a}{F_a} + \frac{C_{mx}\, f_{bx}}{\left(1-\frac{f_a}{F'_{ex}}\right)F_b} + \frac{C_{my}\, f_{by}}{\left(1-\frac{f_a}{F'_{ey}}\right)F_b} \le 1.0$	0.91	OK
	5	Combined loading interaction requirement # 2	$\frac{f_a}{0.472\,F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \le 1.0$	0.91	OK
	6	Buried anchor block location	Anchor rod length ≥ minimum		OK
	7	Anchor rod stress	$\sigma \le 0.55\ F_Y$	N/A	OK
	8	Anchor block capacity	Total Anchor Force ≤ Capacity	N/A	OK
	9	Maximum displacement	$d_{MAX} \le 1.5\,\text{in.}$	0.371 in.	OK

Anchor Design Worksheet

Not applicable, buried concrete anchor option not selected

Foundation Summary			
	1	Roadway width	24.00 ft
	2	Span length	60.00 ft
	3	Distance between superstructure bearings and roadway grade	4.42 ft
	4	Backwall height	6.00 ft
	5	Dead load abutment reaction	210.7 kip per abutment
	6	Live load abutment reaction	121.5 kip per abutment
	7	Number of piles	8
	8	Total axial pile load	27.9 tons
	9	Pile spacing	3.17 ft
	10	Pile size	HP10x42
	11	Pile steel yield stress	36 ksi
	12	Minimum total pile length	42 ft

SAMPLE FOUNDATION DETAILS FOR A PCDT SUPERSTRUCTURE

Table 1. Foundation details for a 40 ft pre-cast double tee (i.e. steel beam) girder bridge.

Superstructure System: Pre-cast double tee Span Length: 40 ft											
General Bridge Information				Pre-designed Foundation Information							
Roadway width (ft)	Soil type (N-value)	Backwall plus scour height (ft)	Pile type	Number of piles	Pile spacing (ft)	*Pile size	Axial pile load (ton)	Minimum embedded length (ft)	Anchor rod detail	Anchor block detail	Anchor elevation above stream (ft)
24	Cohesionless N=20	8	Steel	7	3' - 8"	HP 10x57	24.1	43	-	-	-
			Timber	7	3' - 10"	13", 10"	24.0	39	1	a	1' - 4"
		10	Steel	8	3' - 2"	HP 12x53	21.1	38	-	-	-
			Timber	7	3' - 10"	13", 10"	24.0	39	1	b	3' - 1"
	Cohesive N=10	8	Steel	8	3' - 2"	HP 10x42	21.0	38	-	-	-
			Timber	7	3' - 10"	13", 10"	24.0	39	1	a	1' - 4"
		10	Steel	8	3' - 2"	HP 10x57	21.1	38	-	-	-
			Timber	7	3' - 10"	13", 10"	24.0	39	1	b	3' - 1"
30	Cohesionless N=20	8	Steel	9	3' - 8"	HP 10x57	24.4	44	-	-	-
			Timber	9	3' - 10"	13", 10"	24.3	40	2	a	1' - 4"
		10	Steel	10	3' - 2"	HP 12x53	22.0	40	-	-	-
			Timber	9	3' - 10"	13", 10"	24.3	40	2	b	3' - 1"
	Cohesive N=10	8	Steel	10	3' - 2"	HP 10x42	21.9	40	-	-	-
			Timber	9	3' - 10"	13", 10"	24.3	40	2	a	1' - 4"
		10	Steel	10	3' - 2"	HP 10x57	22.0	40	-	-	-
			Timber	9	3' - 10"	13", 10"	24.3	40	2	b	3' - 1"

* For timber piles, the two values provided refer to the pile butt and tip diameter, respectively.

Table 2. Anchor rod details.

Detail	Number of Rods	Rod Diameter (in.)
1	5	0.75
2	7	0.75

Table 3. Anchor block details.

Detail	Height (ft)	Width (in.)	Flexural Steel (for each anchor block face)	
			Quantity	Bar Size
a	2.50	12	3	5
b	3.00	12	3	5