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

Verification of Design Methodology

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REPORT

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The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Iowa Department of Transportation.
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 COHESIONLESS 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

RDWY := 24ft

BW := 6ft

ES := 2ft

W21 := 21in

SLab depth = 8in

Z_b := BW – 8in – W21

Z_b = 3.583 ft

SPT := 20

FB := 0.7 \text{ton ft}^{-1}

FB := \text{Pile friction bearing resistance (Iowa DOT FSIC)}

NA := 2

GRAVITY LOADS

Dead Loads

GL := SPAN + 2 \cdot (6in)

GL = 41.00 ft

BL := GL + 2 \cdot (6in)

BL = 42.00 ft

G := 57 plf

W21x57 girder weight per foot (Iowa DOT TR-410)

N_G := 8

Number of girders
**Abutment Cross Section**

- **W 21 x 57**
- **Pile cap**
- **End diaphragm**

### Calculated Crown Weight

\[ \text{Crown weight} = \text{Crown BL} \cdot \gamma_c \cdot \cdot \cdot = \text{Crown BL} \cdot \gamma_c \cdot \cdot \cdot \]

- **Calculated crown weight**

### Cross sectional area of crown

\[ \text{Cross sectional area of crown} = \frac{1}{2} \cdot y \cdot \frac{\text{RDWY}}{2} \cdot \cdot \cdot \]

- **Cross sectional area of crown**

### Calculated Girder Weight

\[ \text{Girder weight} = \text{N}_G \cdot G \cdot \cdot \cdot \]

- **Calculated girder weight**

### Calculated Barrier Rail Weight

\[ \text{Rail weight} = 2 \cdot \text{BR} \cdot \text{BL} \]

- **Calculated barrier rail weight**

### Calculated Future Wearing Surface Weight

\[ \text{FWSwt weight} = \text{FWS} \cdot \text{RDWY} \cdot \text{BL} \]

- **Calculated future wearing surface weight**

### Assumed Future Wearing Surface

- **FWS := 20 psf**
- **Conservatively assumed thrie-beam rail weight per foot**

### Concrete Unit Weight

- **γ_c := 0.150 kcf**
- **Concrete unit weight**

### Slab

\[ \text{Slab} := (8\text{in}) \cdot \text{BL} \cdot \text{RDWY} \cdot \gamma_c \]

- **Slab = 100.80 kip**
- **Calculated slab weight**

### Girder

\[ \text{Girder} := \text{NG} \cdot \text{G} \cdot \text{GL} \]

- **Girder = 18.70 kip**
- **Calculated girder weight**

### Rail

\[ \text{Rail} := 2 \cdot \text{BR} \cdot \text{BL} \]

- **Rail = 4.20 kip**
- **Calculated barrier rail weight**

### Slab

\[ \text{Slab} := 8\text{in()} \cdot \text{BL} \cdot \text{RDWY} \cdot \gamma_c \]

- **Slab = 100.80 kip**
- **Calculated slab weight**

### Concrete Unit Weight

- **γ_c 0.150 kcf**
- **Concrete unit weight**

### Assumed Future Wearing Surface

- **FWS 20 psf**
- **Assumed future wearing surface**

### Cross-sectional area of crown

\[ y := \frac{\text{RDWY}}{2} \cdot 2\% \]

- **y = 0.240 ft**
- **Cross-sectional area of crown**

\[ A := \frac{1}{2} \cdot y \cdot \frac{\text{RDWY}}{2} \cdot 2 \]

- **A = 2.880 ft^2**
- **Cross-sectional area of crown**

### Calculated Crown Weight

\[ \text{Crown := BL} \cdot \text{A} \cdot \gamma_c \]

- **Crown = 18.14 kip**
- **Calculated crown weight**
Diaphragm := (18in)·(21in)·RDWY·γ_c·NA  
Diaphragm = 18.90 kip  
Calculated end diaphragm weight (for conservative weight calculations only)

Cap := (3ft)·(3ft)·RDWY·γ_c·NA  
Cap = 64.80 kip  
Calculated pile cap weight

Wale := 2·(20plf)·RDWY·NA  
Wale = 1.92 kip  
Calculated abutment wale weight (2, 20 plf wales per abutment)

\[ DL_{gb} := \text{Slab + Girder + Rail + FWS}_{wt} + \text{Crown} + \text{Diaphragm} + \text{Cap} + \text{Wale} \]

\[ DL_{gb} = 247.62 \text{ kip} \]  
Total bridge dead load

\[ DL_g := \frac{DL_{gb}}{1.05} \]

\[ DL_g = 130.00 \text{ kip} \]  
Dead load abutment reaction (increased by 5% because standards for nonspecific bridge sites were used)

**Live Load**

AASHTO HS20-44 design truck  
(AASHTO 3.7)

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

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

\[ R_A = 55.20 \text{ kip} \]

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

\[ \frac{\text{RDWY}}{10\cdot\text{ft}} = 2.4 \]  
Number of 10 ft design traffic lanes  
(AASHTO 3.6.1)
LN := 2  
No lane reduction factor needed.  
(AASHTO 3.12.1)

\[ LL_g := LN \cdot R_A \]  
\[ LL_g = 110.40 \text{kip} \]  
Calculated live load abutment reaction

\[ TAR := LL_g + DL_g \]  
\[ TAR = 240.40 \text{kip} \]  
Total abutment reaction

pf := 1.40  
Nominal axial pile factor  
(Volume II, Chapter 2)

\[ FAR := TAR \cdot 1.4 \]  
\[ FAR = 336.56 \text{kip} \]  
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)

\[ N_1 := \frac{FAR}{MPL \cdot \left( \frac{2 \text{kip}}{\text{ton}} \right)} \]  
\[ N_1 = 6.73 \]  
7 piles will work

N := 7  
Use 7 piles

\[ S := \frac{RDWY - 2 \cdot (0.75\text{ft})}{(N - 1)} \]  
\[ S = 3.750 \text{ft} \]  
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  
(lowa DOT BDM 6.6.2.6.1)

\[ EA := (1.75\text{ft} + 8\text{in} + W21) \times \text{SPAN} \]
\[ EA = 166.67 \text{ft}^2 \]
Bridge superstructure elevation surface area

\[ WS := \frac{EA \times (50\text{psf})}{NA \times N} \]
\[ WS = 0.60 \text{kip} \]
Wind on superstructure force per pile

WIND ON LIVE LOAD

\[ LL_w := 100\text{plf} \]
Line load applied to entire bridge length
(lowa DOT BDM 6.6.2.6.2)

\[ WL := LL_w \times SPAN \times \frac{1}{NA \times N} \]
\[ WL = 0.29\text{kip} \]
Wind on live load force per pile

Longitudinal Loads

BRAKING FORCE  
(lowa DOT BDM 6.6.2.4)

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

\[ W := 0.64\text{klf} \]
Total lateral force per pile from active earth pressure

\[ \text{EDL} = \frac{1}{2} w_1 \cdot h \]

\[ \text{EDL} = 4.31 \text{kip} \]

Braking force per pile

\[ F := 18 \text{kip} \]

\[ \text{BFP} := \frac{\text{LN} \cdot (W \cdot \text{SPAN} + F) \cdot 0.05}{\text{NA} \cdot \text{N}} \]

\[ \text{BFP} = 0.31 \text{kip} \]

Dead Load Earth Pressure

\[ P_1 = (35.9 \cdot h) \text{ psf} \]

\[ h := \text{BW} + \text{ES} \]

\[ h = 8.00 \text{ ft} \]

\[ P_1 := (35.9 \text{pcf}) \cdot h \]

\[ P_1 = 287.2 \text{ psf} \]

\[ w_1 := P_1 \cdot S \]

\[ w_1 = 1.077 \text{ klf} \]

Convert \( P_1 \) to a distributed pile line load

\[ \text{Total lateral force per pile from active earth pressure} \]
For this example, an anchor system is used. This requires an interactive consistent deformation process starting with an initial assumption for the anchor force per pile.

\[ F := 5.00 \text{ kip} \]  
\[ H := \text{BFP} + \text{LL}_{\text{sur}} + \text{EDL} - F \]  
\[ B = \text{pile width} \]  
\[ D_b := 13\text{in} \]  
\[ D_t := 10\text{in} \]

\[ w_2 := (250 \text{ psf}) \cdot S \]  
\[ w_3 := (35.9 \text{ psf}) \cdot S \]  
\[ \text{LL}_{\text{sur}} := (1\text{ft}) \cdot w_2 + \frac{1}{2} \left( (w_2 - w_3) \cdot (6\text{ft}) + h \cdot w_3 \right) \]  
\[ w_2 = 0.938 \text{ klf} \]  
\[ w_3 = 0.135 \text{ klf} \]  
\[ \text{LL}_{\text{sur}} = 4.42 \text{ kip} \]  
\[ \text{Total lateral force per pile from live load surcharge} \]

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

For a cohesionless soil

\[ \text{Estimated scour line} \]  
\[ 6 \text{ ft} \]  
\[ h - 6 \text{ ft} \]  
\[ h \]  
\[ \text{Roadway elevation} \]  
\[ \text{Backwall} \]  
\[ \text{(Iowa DOT BDM 6.5.2.2)} \]  
\[ \text{LIVE LOAD SURCHARGE} \]  
\[ 250 \text{ psf} \]  
\[ 6 \text{ ft} \]  
\[ 35.9 \text{ psf} \]  
\[ \text{Estimated scour line} \]
To account for the change in cross section use a representative pile diameter.

\[ B := D_t + 0.33(D_b - D_t) \quad B = 10.99 \text{ in} \quad (\text{AASHTO 13.7.3.4.3}) \]

\[ \phi = 53.881\text{deg} - (27.603\text{deg}) \cdot e^{-0.0147 \cdot \text{SPT}} \quad \phi := 33.309\text{deg} \]

\[ K_p := \frac{1 + \sin(\phi)}{1 - \sin(\phi)} \quad K_p := 3.436 \]

Rankine passive earth pressure coefficient (assume soil surface behind the backwall is horizontal)

\[ \gamma_s := 0.125\text{kcf} \]

Soil unit weight

\[ f := 0.82 \cdot \sqrt{\frac{H}{(\gamma_s \cdot B \cdot K_p)}} \quad f = 2.629 \text{ ft} \]

Depth below estimated scour line to pile fixity

Pile and Anchor Properties

\[ A := \frac{\pi}{4}(B^2) \quad A = 94.86 \text{ in}^2 \]

Representative pile area

\[ \gamma_t := 0.05\text{kcf} \]

Timber unit weight

\[ \text{PSW} := A \cdot \gamma_t \quad \text{PSW} = 0.033 \text{ klf} \]

Pile self-weight per foot

\[ I := \frac{\pi}{64}B^4 \quad I = 716.1 \text{ in}^4 \]

Pile moment of inertia

Vertical Anchor Location

\[ BW = 6.00 \text{ ft} \]

\[ ES = 2.00 \text{ ft} \]

\[ Z_b = 3.583 \text{ ft} \]
\[ Z_a := 2.583 \text{ft} \]

Maximum Lateral Anchor Capacity

\[ 
\begin{align*}
\text{Roadway elevation} & \\
Z_1 & = \text{Distance between roadway elevation and top of anchor block} \\
Z_2 & = \text{Distance between roadway elevation and bottom of anchor block} \\
Z_a & = \text{Anchor height} \\
\end{align*} 
\]

Distance between stream and anchor elevations (1ft below the bearings)

\[ \gamma_s = 0.125 \text{ kcf} \]

\[ b := 3.00 \text{ft} \]

\[ 
\begin{align*}
\text{BA} & := Z_a - \frac{1}{2} \cdot b \\
\text{BA} & = 1.083 \text{ ft} \\
\end{align*} 
\]

Distance between stream elevation and bottom of anchor block

\[ Z_1 := \text{Distance between roadway elevation and top of anchor block} \]

\[ Z_1 = 1.917 \text{ ft} \]

Distance between roadway elevation and top of anchor block

\[ Z_2 := \text{Distance between roadway elevation and bottom of anchor block} \]

\[ Z_2 = 4.917 \text{ ft} \]

Distance between roadway elevation and bottom of anchor block

\[ F_{\text{max}} = \text{maximum lateral anchor capacity} = (\gamma_s \cdot b/2) \cdot (Z_1 + Z_2) \cdot (K_p - K_a) \]

(Bowles 13-8.3)

\[ \phi_b := 33.69 \text{deg} \]

Backfill soil friction angle (Iowa DOT BDM 6.5.2.5)

\[ K_{pa} := \frac{1 + \sin(\phi_b)}{1 - \sin(\phi_b)} \]

\[ K_{pa} := 3.491 \]

Rankine passive earth pressure coefficient for backfill soil

\[ K_{aa} := K_{pa}^{-1} \]

\[ K_{aa} = 0.286 \]

Rankine active earth pressure coefficient for backfill soil
\[ FM := \frac{\gamma_Z (b)}{2} \left( Z_2 + Z_1 \right) \left( K_{pa} - K_{aa} \right) \]

\[ \text{FM} = 4.106 \text{ klf} \quad \text{Maximum lateral anchor capacity per foot} \]

\[ FS := 1.5 \]

\[ S = 3.750 \text{ ft} \quad \text{Pile spacing} \]

\[ FMP := \frac{FM \cdot S}{FS} \]

\[ \text{FMP} = 10.27 \text{ kip} \quad \text{Maximum anchor block capacity per pile} \]

Therefore 5 kips anchor force assumption is \textbf{OK}.

\textbf{Compute New Anchor Force Per Pile}

\[ F = \sigma \cdot A \quad \sigma = E \cdot \varepsilon \quad \varepsilon = \frac{\Delta L}{L_0} \quad \Delta L = \text{Pile deflection at anchor elevation} \]

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) \textbf{DEAD LOAD EARTH PRESSURE}
Total pile deflection at anchor elevation from active earth pressure

\[ d_{a1} = d_1 + d_2 + \frac{\theta}{E \cdot l} (Z_a + ES) \]

Distance between anchor elevation and estimated scour line

\[ x = Z_a + ES \]

Pile slope at estimated scour line

\[ \theta = \frac{1}{E \cdot l} \left( M \cdot f + \frac{V \cdot f^3}{2} \right) \]

Timber modulus of elasticity for southern pine (AASHTO Table 13.5.1A)

\[ E = 1600 \text{ ksi} \]

Distance between anchor elevation and estimated scour line

\[ x = 55.0 \text{ in} \]

Pile deflection at anchor elevation

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

Pile deflection at estimated scour line

\[ d_2 = \frac{1}{E \cdot l} \left[ \frac{M \cdot f^2}{2} + \frac{V \cdot f^3}{3} \right] \]

Moment at estimated scour line

\[ M = \frac{1}{2} \cdot h \cdot w_1 \cdot \frac{h}{3} \]

Shear at estimated scour line

\[ V = \frac{1}{2} \cdot h \cdot w_1 \]

Total pile deflection at anchor elevation from active earth pressure

\[ d_{a1} = 0.515 \text{ in} \]

Moment at estimated scour line

\[ M = 11.49 \text{ ft kip} \]

Pile deflection at estimated scour line

\[ d_1 = 0.104 \text{ in} \]

Shear at estimated scour line

\[ V = 4.31 \text{ kip} \]

Pile slope at estimated scour line

\[ \theta = 0.006 \text{ rad} \]
2) LIVE LOAD SURCHARGE

\[ f = 2.629 \text{ ft} \quad ES = 2.00 \text{ ft} \quad BW = 6.00 \text{ ft} \]
\[ h = 8.00 \text{ ft} \quad Z_a = 2.583 \text{ ft} \]
\[ w_2 = 0.938 \text{ klf} \quad w_3 = 0.135 \text{ klf} \]

Part a)

\[ x := Z_a + ES \quad x = 55.00 \text{ in} \]
Distance between anchor elevation and estimated scour line
$$d_1 := \frac{w_3 x^2}{24 \cdot E \cdot I} \left(6 \cdot h^2 - 4 \cdot h \cdot x + x^2\right)$$

\[d_1 = 0.046 \text{ in}\]

Pile deflection at anchor elevation

$$M := \frac{w_3 h^2}{2}$$

\[M = 4.31 \text{ ft} \cdot \text{kip}\]

Moment at estimated scour line

$$V := w_3 \cdot h$$

\[V = 1.08 \text{ kip}\]

Shear at estimated scour line

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

\[d_2 = 0.032 \text{ in}\]

Pile deflection at estimated scour line

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

\[\theta = 0.002 \text{ rad}\]

Pile slope at estimated scour line

$$d_{a2} := d_1 + d_2 + \theta \cdot x$$

\[d_{a2} = 0.182 \text{ in}\]

Total pile deflection at anchor elevation from Part a) of live load surcharge

Part b)

$$L := ES + BW + f + 1 \cdot \text{ft}$$

\[L = 11.629 \text{ ft}\]

Pile length from point of fixity to 1 ft above roadway elevation

$$x := f + ES + Z_a$$

\[x = 7.212 \text{ ft}\]

Distance between anchor elevation and point of fixity

$$d_{a3} := \frac{w_2 \cdot (1\text{ft}) \cdot x^2}{2 \cdot E \cdot I} \left[\left(\frac{-1}{3}\right) \cdot x - \frac{1}{2} \cdot (1\text{ft}) + L\right]$$

\[d_{a3} = 0.321 \text{ in}\]

Total pile deflection at anchor elevation from Part b) of live load surcharge
**Part c)**

\[ w_4 := w_2 - w_3 \quad w_4 = 0.803 \text{ klf} \]

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

\[ d_1 = 0.038 \text{ in} \quad \text{Pile deflection at anchor elevation} \]

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

\[ M := V \cdot \left( \frac{2}{3} \right) \cdot (6 \text{ ft}) \quad M = 9.63 \text{ ftkip} \quad \text{Moment at stream elevation} \]

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

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

\[ d_2 = 0.276 \text{ in} \quad \text{Pile deflection at stream elevation} \]

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

\[ \theta = 0.009 \text{ rad} \quad \text{Pile slope at stream elevation} \]

\[ d_{a4} := d_1 + d_2 + \theta \cdot Z_a \]

\[ d_{a4} = 0.588 \text{ in} \quad \text{Total pile deflection at anchor elevation from Part c) of live load surcharge} \]
3) ANCHOR FORCE

\[ x := f + ES + Z_a \]

\[ d_{a5} := \frac{-F \cdot x^3}{3 \cdot E \cdot I} \]

\[ d_{a5} = -0.943 \text{ in} \]

4) BRAKING FORCE

\[ x_1 := f + ES + Z_a \]

\[ x_1 = 7.212 \text{ ft} \]

\[ x_2 := f + ES + Z_b \]

\[ x_2 = 8.212 \text{ ft} \]

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

\[ d_{a6} = 0.071 \text{ in} \]
5) PASSIVE EARTH PRESSURE

\[ \begin{align*}
\alpha & := \frac{1.92 \cdot H}{f^2} \quad \text{\( \alpha = 1.123 \text{ ksf} \)} \\
\xi & := 0.12 \frac{H}{f^3} \quad \text{\( \xi = 0.027 \text{ kcf} \)} \\
_{\text{distance between point of}} \quad x & := f + ES + Z_a \quad \text{\( x = 7.212 \text{ ft} \)} \\
_{\text{fixity and anchor elevation}} \\
_{\text{total pile deflection at}} \quad 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) \left( \frac{1}{E \cdot I} \right) \\
_{\text{anchor elevation from}} \quad d_{a7} & = -0.023 \text{ in} \\
_{\text{passive soil reaction}} \\
_{\text{total pile deflection at}} \quad d_{aT} & := d_{a1} + d_{a2} + d_{a3} + d_{a4} + d_{a5} + d_{a6} + d_{a7} \quad \text{\( d_{aT} = 0.711 \text{ in} \)} \\
_{\text{anchor elevation}} \\
_{\text{pile deflection at the anchor location}} \quad d_{aT} & = 0.711 \text{ in. with an assumed anchor force of 5 kips per pile.}
\end{align*} \]
Case A) minimum anchor rod length

\[ \alpha = 45 - \phi / 2 \]

\[ h = 8.00 \text{ ft} \]
\[ b = 3.00 \text{ ft} \]
\[ ES = 2.00 \text{ ft} \]
\[ Z_a = 2.583 \text{ ft} \]

\[ y := Z_a + ES - \frac{1}{2} \cdot b \]
\[ y = 3.083 \text{ ft} \]

Two possibilities:
- a) active failure plane controls
- b) passive failure plane controls

Case A: minimum rod length = \( x_1 + x_2 \)

\[ \phi_b := 33.69 \text{ deg} \]
\[ \alpha := 45 \text{ deg} - \frac{\phi_b}{2} \]
\[ \alpha = 28.16 \text{ deg} \]

\[ \tan(\theta) := \frac{x_1}{h} \]
\[ x_1 := \tan(\alpha) \cdot h \]
\[ x_1 := 4.28 \text{ ft} \]

\[ x_2 := \frac{h - y}{\tan(\alpha)} \]
\[ x_2 := 9.20 \text{ ft} \]

\[ x_1 + x_2 = 13.480 \text{ ft} \]

Distance between estimated scour line and anchor elevation

Backfill soil friction angle
(Iowa DOT BDM 6.5.2.4)
Case B:

\[ x_3 := \frac{y + b}{\tan(\alpha)} \quad x_3 := 11.37 \text{ ft} \]

Case B) minimum anchor rod length

13.47 ft > 11.48 ft

Minimum anchor rod length = 13.47 ft.

\[ x_r := 15 \text{ ft} \]

Anchor rod length used for this analysis

\[ d_{aT} = 0.711 \text{ in} \]

Anchor rod elongation and pile deflection at anchor elevation

\[ \varepsilon_r := \frac{d_{aT}}{x_r} \quad \varepsilon_r = 0.0040 \]

Anchor rod strain

\[ f_y := 60 \text{ ksi} \]

Anchor rod yield stress

\[ \varepsilon_y := \frac{f_y}{29000 \text{ ksi}} \quad \varepsilon_y = 0.0021 \]

Anchor rod yield strain

\[ \varepsilon_y < \varepsilon_r \]

Therefore:

\[ \sigma_r := 60 \text{ ksi} \]

Anchor rod stress

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

\[ N_r := 5 \]

Anchor rods per abutment

\[ \phi_r := 0.75 \text{ in} \]

Anchor rod diameter

\[ N = 7 \]

Number of piles

\[ A_{rp} := \frac{\pi}{4} \cdot \phi_r^2 \cdot \frac{N_r}{N} \quad A_{rp} = 0.316 \text{ in}^2 \]

Anchor rod area per pile

\[ F_{ap} := \sigma_r \cdot A_{rp} \quad F_{ap} = 18.93 \text{ kip} \]

Calculated anchor force per pile

\[ F := 9.00 \text{ kip} \]

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

Determine Depth to Pile Fixity

\[ f = \text{depth to fixity} \quad f := 0.82 \cdot \frac{H}{\sqrt{\gamma_c \cdot B \cdot K_p}} \]

\[ BFP = 0.31 \text{ kip} \]

EDL = 4.31 kip

\[ LL_{sur} = 4.42 \text{ kip} \]
Compute the deflection of the pile at the elevation of the anchor rod.

1) DEAD LOAD EARTH PRESSURE

\[ \begin{align*}
H &:= BFP + LL_{sur} + EDL - F \\
f &:= 0.82 \sqrt{\frac{H}{\gamma_s \cdot B \cdot K_p}}
\end{align*} \]

\[ \begin{align*}
H &= 0.04 \text{ kip} \\
f &= 0.270 \text{ ft}
\end{align*} \]

\[ \begin{align*}
x &:= Z_a + ES \\
x &= 55.00 \text{ in}
\end{align*} \]

\[ \begin{align*}
E &= 1600 \text{ ksi} \\
I &= 716.1 \text{ in}^4
\end{align*} \]

\[ \begin{align*}
d_1 &= \frac{w_1 (x^2)}{120 \cdot h \cdot E \cdot I} \left[ (10 \cdot h^3 - 10 \cdot h^2 \cdot x) + 5 \cdot h \cdot x^2 - x^3 \right] \\
d_1 &= 0.104 \text{ in}
\end{align*} \]
\[ M := \frac{1}{2} \cdot h \cdot w_1 \cdot \left( \frac{h}{3} \right) \]

Moments at estimated scour line

\[ V := \frac{1}{2} \cdot h \cdot w_1 \]

Shear at estimated scour line

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

Pile deflection at estimated scour line

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

Pile slope at estimated scour line

\[ d_{a1} := d_1 + d_2 + \theta \cdot (Z_a + ES) \]

Total pile deflection at anchor elevation from active earth pressure

2) LIVE LOAD SURCHARGE

\[ w_2 = 0.938 \text{ klf} \]

\[ w_3 = 0.135 \text{ klf} \]

\[ w_4 = 0.803 \text{ klf} \]

\[ f = 0.270 \text{ ft} \]

\[ ES = 2.00 \text{ ft} \]

\[ BW = 6.00 \text{ ft} \]

\[ h = 8.00 \text{ ft} \]

\[ Z_a = 2.583 \text{ ft} \]
Part a)

\[
\begin{align*}
\text{d}_1 &= \frac{w_3 \left( x^2 \right)}{24 \cdot E \cdot I} \left( 6 \cdot h^2 - 4 \cdot h \cdot x + x^2 \right) & \text{d}_1 &= 0.046 \text{ in} \\
\text{M} &= \frac{w_3 \cdot h^2}{2} & \text{M} &= 4.31 \text{ ft}-\text{kip} \\
\text{V} &= w_3 \cdot h & \text{V} &= 1.08 \text{ kip} \\
\text{d}_2 &= \frac{1}{E \cdot I} \left[ \frac{(M \cdot f^2)}{2} + \frac{(V \cdot f^3)}{3} \right] & \text{d}_2 &= 0.0002 \text{ in} \\
\theta &= \frac{1}{E \cdot I} \left( M \cdot f + \frac{V \cdot f^2}{2} \right) & \theta &= 0.0002 \\
\text{d}_{a2} &= \text{d}_1 + \text{d}_2 + \theta \cdot x & \text{d}_{a2} &= 0.054 \text{ in}
\end{align*}
\]

x := Z_a + ES 

x = 55.00 in Distance between estimated scour line and anchor elevation

\[
\begin{align*}
\text{d}_1 &= \frac{w_3 \left( x^2 \right)}{24 \cdot E \cdot I} \left( 6 \cdot h^2 - 4 \cdot h \cdot x + x^2 \right) & \text{d}_1 &= 0.046 \text{ in} \\
\text{M} &= \frac{w_3 \cdot h^2}{2} & \text{M} &= 4.31 \text{ ft}-\text{kip} \\
\text{V} &= w_3 \cdot h & \text{V} &= 1.08 \text{ kip} \\
\text{d}_2 &= \frac{1}{E \cdot I} \left[ \frac{(M \cdot f^2)}{2} + \frac{(V \cdot f^3)}{3} \right] & \text{d}_2 &= 0.0002 \text{ in} \\
\theta &= \frac{1}{E \cdot I} \left( M \cdot f + \frac{V \cdot f^2}{2} \right) & \theta &= 0.0002 \\
\text{d}_{a2} &= \text{d}_1 + \text{d}_2 + \theta \cdot x & \text{d}_{a2} &= 0.054 \text{ in}
\end{align*}
\]
Part b)

\[ L := ES + BW + f + 1 \text{ft} \]

\[ L = 9.270 \text{ ft} \]

\[ x := f + ES + Z_a \]

\[ x = 4.853 \text{ ft} \]

\[ d_{a3} := \frac{w_2 \cdot (1 \text{ft}) \cdot x^2}{2 \cdot E \cdot I} \left[ \left( \frac{-1}{3} \right) \cdot x - \frac{1}{2} \left( 1 \text{ft} \right) + L \right] \]

\[ d_{a3} = 0.119 \text{ in} \]

Distance between point of fixity and 1 ft above roadway elevation

Distance between and anchor elevation and point of fixity

Total pile deflection at anchor elevation from Part b) of live load surcharge

Part c)
\[
\begin{align*}
d_1 &= \frac{w_4 \cdot Z_a^2}{120 \cdot (6\text{ft}) \cdot E \cdot I} \left[ 20 \cdot (6\text{ft})^3 - 10 \cdot (6\text{ft})^2 \cdot Z_a + Z_a^3 \right]
\end{align*}
\]

\[d_1 = 0.038 \text{ in}\]

Pile deflection at anchor elevation

\[
V := \frac{1}{2} \cdot w_4 \cdot (6\text{ft})
\]

\[V = 2.41 \text{ kip}\]

Shear at stream elevation

\[
M := V \cdot \left( \frac{2}{3} \right) \cdot 6\text{ft}
\]

\[M = 9.63 \text{ ft-kip}\]

Moment at stream elevation

\[x := f + ES\]

\[x = 2.270 \text{ ft}\]

Distance between pile fixity and stream elevation

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

\[d_2 = 0.052 \text{ in}\]

Pile deflection at stream elevation

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

\[\theta = 0.004 \text{ rad}\]

Pile slope at stream elevation

\[
d_{a4} := d_1 + d_2 + \theta \cdot Z_a\]

\[d_{a4} = 0.199 \text{ in}\]

Total pile deflection at anchor elevation from Part c) of live load surcharge

3) ANCHOR FORCE

\[d_{a5} = \frac{-F \cdot x^3}{3 \cdot E \cdot I}\]

\[d_{a5} = -0.517 \text{ in}\]

Total pile deflection at anchor elevation from assumed anchor force

\[x := f + ES + Z_a\]

\[x = 4.853 \text{ ft}\]

Distance between pile fixity and anchor elevation

\[f = 0.270 \text{ ft}\]

\[ES = 2.00 \text{ ft}\]

\[Z_a = 2.583 \text{ ft}\]

\[F = 9.00 \text{ kip}\]
4) BRAKING FORCE

\[ x_1 := f + ES + Z_a \]
\[ x_1 = 4.853 \text{ ft} \]  
Distance between point of pile fixity and anchor elevation

\[ x_2 := f + ES + Z_b \]
\[ x_2 = 5.853 \text{ ft} \]  
Distance between point of pile fixity and bearing elevation

\[ d_{a6} := \frac{BFP \cdot x_1^2}{6 \cdot E \cdot I} \left[ 3 \cdot (x_2) - x_1 \right] \]
\[ d_{a6} = 0.023 \text{ in} \]  
Pile deflection at anchor elevation due to braking force

5) PASSIVE EARTH PRESSURE

\[ f = 0.270 \text{ ft} \]
\[ ES = 2.000 \text{ ft} \]
\[ Z_a = 2.583 \text{ ft} \]
\[ H = 0.043 \text{ kip} \]
Final pile deflection and anchor rod elongation

\[ \delta_{aT} = 0.1500 \text{ in} \]

Final depth to fixity below estimated scour line

\[ f = 1.5549 \text{ ft} \]

Final anchor force per pile

\[ F = 7.628 \text{ kip} \]

After several iterations:

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.

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.

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

\[ \varepsilon_r = 3.56 \times 10^{-5} \text{ Anchor rod strain} \]

\[ \sigma_r = 1.032 \text{ ksi} \text{ Anchor rod stress} \]

\[ F = \sigma_r A_{rp} \text{ Calculated anchor force per pile} \]

The constants in the equation of parabolic passive soil reaction distribution are:

\[ \alpha := \frac{1.92 \cdot H}{f^2} \quad \alpha = 1.123 \text{ ksf} \]

\[ \xi := \frac{0.12 \cdot H}{f^3} \quad \xi = 0.260 \text{ kcf} \]

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

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

\[ d_{aT} = d_{a1} + d_{a2} + d_{a3} + d_{a4} + d_{a5} + d_{a6} + d_{a7} \]

\[ d_{aT} = 0.006 \text{ in} \text{ Total pile deflection at anchor elevation} \]
\[ \varepsilon_r := \frac{\delta_a T}{15 \text{ ft}} \]
\[ \varepsilon_r = 0.001 \]
Final anchor rod strain

\[ \sigma := 29000 \text{ksi} \left( \varepsilon_r \right) \]
\[ \sigma = 24.17 \text{ksi} \]
Final anchor rod stress
(OK if 60 ksi steel is used)

\[ H := \text{BFP} + \text{LL}_{\text{sur}} + \text{EDL} - F \]
\[ H = 1.41 \text{kip} \]
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)

\[ ES = 2.00 \text{ ft} \]
\[ Z_a = 2.583 \text{ ft} \]

\[ x_b := f + ES + Z_a \]
\[ x_B = 6.138 \text{ ft} \]
Distance between pile fixity and anchor elevation

\[ x_c := \frac{x_b}{2} \]
\[ x_C = 3.069 \text{ ft} \]
Halfway between pile fixity and anchor elevation

\[ x_c := 3.00 \text{ ft} \]
Use \( x_c = 3 \text{ ft} \)

**EARTH DEAD LOAD**

\[ w_1 = 1.077 \text{ klf} \]
\[ h = 8.00 \text{ ft} \]
\[ Z_a = 2.583 \text{ ft} \]

\[ w_B := 0.460 \text{ klf} \]
\[ f = 1.555 \text{ ft} \]
\[ ES = 2.00 \text{ ft} \]

\[ w_C := 0.883 \text{ klf} \]

Roadway elevation
Anchor elevation
Stream elevation
Estimated scour line

\( \delta_a \) - Anchor displacement
\( T \) - Anchor force
\( f \) - Pile fixity
\( x \) - Distance from pile shaft
\[ M_{A1} := \frac{1}{2} w_C \cdot h \cdot \left( f + \frac{h}{3} \right) \]
\[ M_{A1} = 18.19 \text{ ft.kip} \]
\[ M_{B1} := \frac{1}{2} w_B \cdot (h - ES - Z_a)^2 \cdot \frac{1}{3} \]
\[ M_{B1} = 0.895 \text{ ft.kip} \]
\[ M_{C1} := \frac{1}{2} w_C \cdot (h + f - x_c)^2 \cdot \frac{1}{3} \]
\[ M_{C1} = 6.32 \text{ ft.kip} \]

**LIVE LOAD SURCHARGE**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>( f )</td>
<td>1.555 ft</td>
</tr>
<tr>
<td>( Z_a )</td>
<td>2.583 ft</td>
</tr>
<tr>
<td>( w_4 )</td>
<td>0.803 klf</td>
</tr>
<tr>
<td>( ES )</td>
<td>2.00 ft</td>
</tr>
<tr>
<td>( w_2 )</td>
<td>0.938 klf</td>
</tr>
<tr>
<td>( h )</td>
<td>8.00 ft</td>
</tr>
<tr>
<td>( w_3 )</td>
<td>0.135 klf</td>
</tr>
</tbody>
</table>

**Part a)**

\[ x_1 := f + h \]
\[ x_1 = 9.555 \text{ ft} \]
Distance between pile fixity and roadway

\[ x_2 := f + ES + Z_a \]
\[ x_2 = 6.138 \text{ ft} \]
Distance between pile fixity and anchor elevation

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

\[ M_{B2} := w_3 \frac{(x_1 - x_2)^2}{2} \]
\[ M_{B2} = 0.79 \text{ ft.kip} \]

\[ M_{C2} := w_3 \frac{(x_1 - x_c)^2}{2} \]
\[ M_{C2} = 2.89 \text{ ft.kip} \]
Part b)

\[ x_1 := f + h \quad x_1 = 9.555 \text{ ft} \]
Distance between pile fixity and roadway elevation

\[ x_2 := f + ES + Z_a \quad x_2 = 6.138 \text{ ft} \]
Distance between pile fixity and anchor elevation

\[ M_{A3} := w_2 \cdot (1\text{ft}) \left( x_1 + \frac{1\text{ft}}{2} \right) \quad M_{A3} = 9.43 \text{ ft.kip} \]

\[ M_{B3} := w_2 \cdot (1\text{ft}) \left( x_1 - x_2 + \frac{1\text{ft}}{2} \right) \quad M_{B3} = 3.67 \text{ ft.kip} \]

\[ M_{C3} := w_2 \cdot (1\text{ft}) \left( x_1 - x_c + \frac{1\text{ft}}{2} \right) \quad M_{C3} = 6.61 \text{ ft.kip} \]

Part c)
\[ w_B := 0.346 \text{ klf} \]

\[ x_1 := f + h \quad x_1 = 9.555 \text{ ft} \quad \text{Distance between pile fixity and roadway elevation} \]

\[ x_2 := f + ES + Z_a \quad x_2 = 6.138 \text{ ft} \quad \text{Distance between pile fixity and anchor elevation} \]

\[ x_3 := x_1 - 6\text{ft} \quad x_3 = 3.555 \text{ ft} \quad \text{Distance between pile fixity and bottom of triangular load} \]

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

\[ M_{B4} := w_B \left( \frac{x_1 - x_2}{2} \right)^2 + \frac{1}{2} \left( w_4 - w_B \right) \left( x_1 - x_2 \right)^2 \left( \frac{2}{3} \right) \quad M_{B4} = 3.80 \text{ ft-kip} \]

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

**ANCHOR FORCE**

\[ x := f + ES + Z_a \quad x = 6.138 \text{ ft} \quad \text{Distance between pile fixity and anchor elevation} \]

\[ M_{A5} := -F \cdot x \quad M_{A5} = -46.82 \text{ ft-kip} \]

\[ M_{B5} := 0.00 \text{ ft-kip} \quad M_{B5} = 0.00 \text{ ft-kip} \]

\[ M_{C5} := -F \left( x - x_c \right) \quad M_{C5} = -23.94 \text{ ft-kip} \]
BRAKING FORCE

\[ x_1 := f + ES + Z_b \]
\[ x_1 = 7.138 \text{ ft} \]
Distance between pile fixity and bearing elevation

\[ x_2 := f + ES + Z_a \]
\[ x_2 = 6.138 \text{ ft} \]
Distance between pile fixity and anchor elevation

\[ M_{A6} := BFP \cdot x_1 \]
\[ M_{A6} = 2.22 \text{ ft·kip} \]

\[ M_{B6} := BFP \left( x_1 - x_2 \right) \]
\[ M_{B6} = 0.31 \text{ ft·kip} \]

\[ M_{C6} := BFP \left( x_1 - x_c \right) \]
\[ M_{C6} = 1.29 \text{ ft·kip} \]

PASSIVE EARTH PRESSURE

\[ f = 1.555 \text{ ft} \]

\[ H = 1.41 \text{ kip} \]
Maximum total pile moment

\[ M := 9.46 \text{ ft·kip} \]

\[ M_{CT} := M_{C1} + M_{C2} + M_{C3} + M_{C4} + M_{C5} + M_{C6} + M_{C7} \]
\[ M_{CT} = 4.15 \text{ ft·kip} \]

\[ M_{BT} := M_{B1} + M_{B2} + M_{B3} + M_{B4} + M_{B5} + M_{B6} + M_{B7} \]
\[ M_{BT} = 9.46 \text{ ft·kip} \]

\[ M_{AT} := M_{A1} + M_{A2} + M_{A3} + M_{A4} + M_{A5} + M_{A6} + M_{A7} \]
\[ M_{AT} = 6.47 \text{ ft·kip} \]

\[ w(x') = \alpha \cdot (x') + \zeta \cdot (x')^2 \]

\[ V(x') = \int w(x')dx \]
\[ M(x') = \int V(x')dx \]
\[ \text{for } 0 \leq x \leq f' \]

\[ M_{A7} := \frac{-\alpha \cdot f^3}{6} - \frac{\zeta \cdot f^4}{12} \]
\[ M_{A7} = -0.73 \text{ ft·kip} \]
\[ M_{B7} := 0.00 \text{ ft·kip} \]
\[ M_{C7} := 0.00 \text{ ft·kip} \]

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

\[ \zeta := 0.12 \cdot \frac{H}{f^3} \]
\[ \zeta = 0.045 \text{ kcf} \]

\[ \text{Derived equation for pile moment as a function of } x' \]

\[ \text{for } x' = f \]

\[ \text{Constants in equation of parabolic passive soil reaction distribution} \]
Transverse Pile Moments

\[ f = 1.555 \text{ ft} \quad Z_b = 3.583 \text{ ft} \]
\[ ES = 2.00 \text{ ft} \]
\[ WS = 0.60 \text{ kip} \quad \text{Wind on superstructure force per pile} \]
\[ WL = 0.29 \text{ kip} \quad \text{Wind on live load force per pile} \]

\[ M_{WS} := WS \left( f + ES + Z_b \right) \quad M_{WS} = 4.25 \text{ ft-kip} \quad \text{Wind on superstructure transverse pile moment} \]

\[ M_{WL} := WL \left( f + ES + Z_b \right) \quad M_{WL} = 2.04 \text{ ft-kip} \quad \text{Wind on live load transverse pile moment} \]

PILE SELF-WEIGHT

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

\[ x := f + ES + Z_b \quad x = 7.138 \text{ ft} \quad \text{Distance between point of fixity and bearing elevation} \]

\[ P_{SW} := 0.033 \text{ klf} \quad \text{Pile self-weight per foot} \]

\[ P_{SWT} := P_{SW} \cdot x \quad P_{SWT} = 0.24 \text{ kip} \quad \text{Pile weight} \]

LOAD SUMMARY

\[ DL_g = 130.00 \text{ kip} \quad \text{Dead load abutment reaction} \]
\[ pf = 1.4 \quad \text{Nominal axial pile factor} \quad \text{(Chapter 2, Volume 2)} \]

\[ N = 7 \quad \text{Number of piles} \]

\[ P_{DL} := \frac{DL_g \cdot pf}{N} \quad P_{DL} = 26.00 \text{ kip} \quad \text{Pile axial dead load} \]

\[ P_{LL} := \frac{LL_g \cdot pf}{N} \quad P_{LL} = 22.08 \text{ kip} \quad \text{Pile axial live load} \]

\[ P_T := P_{DL} + P_{LL} \quad P_T = 48.08 \text{ kip} \quad \text{Pile total axial load} \]
DESIGN CHECKS

Pile Length

\[ Z_b = 3.583 \text{ ft} \quad \text{ES} = 2.00 \text{ ft} \]

\[ B = 10.99 \text{ in} \quad \text{Representative pile diameter} \]

\[ P_T = 48.08 \text{ kip} \quad \text{Total axial pile load} \]

\[ FB := 0.7 \frac{\text{ton}}{\text{ft}} \quad \text{Friction bearing resistance} \]

\[ AFB := \frac{B}{10 \text{ in}} \cdot FB \]

\[ AFB = 0.77 \frac{\text{ton}}{\text{ft}} \quad \text{Adjusted friction bearing resistance} \]

\[ PL := \frac{P_T \left( \frac{1 \text{ton}}{2 \text{kip}} \right)}{AFB} \]

\[ PL = 31.249 \text{ ft} \quad \text{Required minimum embedded pile length} \]

\[ TPL := PL + ES + Z_b \]

\[ TPL = 36.833 \text{ ft} \quad \text{Required minimum total pile length} \]

Roundup to nearest 5 ft, 40 ft < 55 ft \textbf{OK} (Iowa DOT BDM 6.2.6.3)

Allowable Axial Pile Load

25 tons for piles 30 ft and longer \textbf{(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.

\[ P_T \left( \frac{1 \text{ton}}{2 \text{kip}} \right) = 24.04 \text{ ton} \quad 24.04 \text{ tons} < 25 \text{ tons} \quad \textbf{OK} \]

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 \textbf{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)

\[
\left(\frac{f_c}{F_c}\right)^2 + \frac{f_{bx}}{F'_{bx}} \left(1 - \frac{f_c}{F_{cEx}}\right) + \frac{f_{by}}{F_{by}} \left(1 - \frac{f_c}{F_{cEy}} - \frac{f_{bx}}{F_{bE}}\right) < 1.0
\]

(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)

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

Group II (without Live Load)

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

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 separated as demonstrated on the previous page.

\[ I = 716.1 \text{ in}^4 \quad \text{Representative moment of inertia} \]
\[ B = 10.99 \text{ in} \quad \text{Representative pile diameter} \]
\[ SM := \frac{1}{\left(\frac{B}{2}\right)} \quad SM = 130.3 \text{ in}^3 \quad \text{Section modulus} \]
\[ M = 9.46 \text{ ft-kip} \quad \text{Maximum pile moment (x-axis bending)} \]
\[ M_{WS} = 4.25 \text{ ft-kip} \quad \text{Wind on superstructure pile moment (y-axis bending)} \]
\[ M_{WL} = 2.04 \text{ ft-kip} \quad \text{Wind on live load pile moment (y-axis bending)} \]
\[ f_{bx} := \frac{M}{SM} \quad f_{bx} = 0.871 \text{ ksi} \quad \text{Groups I, II, and III applied x-axis bending stress} \]
\[ f_{byW} := \frac{M_{WS}}{SM} \quad f_{byW} = 0.391 \text{ ksi} \quad \text{Group II and III applied y-axis bending stress from wind on superstructure} \]
\[ f_{byWL} := \frac{M_{WL}}{SM} \quad f_{byWL} = 0.188 \text{ ksi} \quad \text{Group III applied y-axis bending stress from wind on live load} \]
ALLOWABLE STRESSES

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

\[ A = 94.86 \text{ in}^2 \]

\[ d := \sqrt{A} \]

\[ d = 9.74 \text{ in} \quad \text{Equivalent square dimension} \]

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.

\[ E = 1600 \text{ ksi} \quad \text{Tabulated timber modulus of elasticity} \]

\[ F_c := 1100 \text{ psi} \quad \text{Tabulated timber compressive stress} \]

\[ F_b := 1750 \text{ psi} \quad \text{Tabulated timber bending stress} \]

\[ C_M := 1.0 \quad \text{Wet service compression factor} \] (AASHTO Table 13.5.1A)

\[ C_F := 1.0 \quad \text{For sawn lumber only} \]

\[ C_D := 0.90 \quad \text{Load duration factor for permanent loading} \] (AASHTO Table 13.5.5A)

\[ C_p = \frac{1 + \frac{F_cE}{F_c'}}{2 \cdot c} - \sqrt{\frac{1 + \frac{F_cE}{F_c'}}{2 \cdot c}} - \frac{\frac{F_cE}{F_c'}}{c} \quad \text{Column stability factor} \] (AASHTO 13.7.3.3.5)

\[ c := 0.85 \quad \text{For round piles} \]

\[ K_{cE} := 0.30 \quad \text{For visually graded lumber} \]
\[ E' := E \cdot C_M \quad E' = 1600 \text{ ksi} \quad \text{Adjusted modulus of elasticity} \]

**X-axis Bending:**

\[ l_e = (k) \cdot (\text{length between braced points}) \]

\[ k_x := 0.7 \]

\[ d = 9.74 \text{ in} \quad \text{Equivalent square dimension} \]

\[ f = 1.555 \text{ ft} \quad Z_a = 2.583 \text{ ft} \]

\[ \text{ES} = 2.00 \text{ ft} \]

\[ I_X := f + \text{ES} + Z_a \quad I_x = 6.138 \text{ ft} \quad \text{Distance between point of fixity and anchor elevation} \]

\[ l_{ex} := I_x \cdot k_x \quad I_{ex} = 4.297 \text{ ft} \quad \text{Effective pile length for x-axis bending} \]

\[ F_{cEx} := \frac{K_{cE} \cdot E'}{(l_{ex})^2} \left( \frac{l_{ex}}{d} \right) \quad F_{cEx} = 17.13 \text{ ksi} \quad \text{x-axis buckling stress} \]

**Y-axis Bending:**

\[ k_y := 0.7 \]

\[ Z_b = 3.583 \text{ ft} \]

\[ I_y := f + \text{ES} + Z_b \quad I_y = 7.138 \text{ ft} \quad \text{Distance between point of fixity and bearing elevation} \]

\[ l_{ey} := k_y \cdot l_y \quad I_{ey} = 4.997 \text{ ft} \quad \text{Effective pile length for y-axis bending} \]

\[ F_{cEy} := \frac{K_{cE} \cdot E'}{(l_{ey})^2} \left( \frac{l_{ey}}{d} \right) \quad F_{cEy} = 12.66 \text{ ksi} \quad \text{y-axis buckling stress} \]

\[ F_c' := F_c \cdot C_M \cdot C_D \cdot C_F \quad F_c' = 0.990 \text{ ksi} \quad \text{Allowable axial stress without column stability factor} \]
Cf := 1.18
Round member factor (AASHTO 13.6.4.5)

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

CV := 1.0
For sawn lumber only

CF := 1.0
For glued laminated timber only

CM := 1.0
Tabulated timber bending stress (AASHTO 13.6.4.1)

CD := 0.90
Wet service bending factor for members 5" x 5" or larger (AASHTO Table 13.5.1A)

CF := 1.0
For sawn lumber only

CV := 1.0
For glued laminated timber only

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

Cf := 1.18
Round member factor (AASHTO 13.6.4.5)

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

CV := 1.0
For sawn lumber only

CF := 1.0
For glued laminated timber only

CM := 1.0
Tabulated timber bending stress (AASHTO 13.6.4.1)

CD := 0.90
Wet service bending factor for members 5" x 5" or larger (AASHTO Table 13.5.1A)

CF := 1.0
For sawn lumber only

CV := 1.0
For glued laminated timber only

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

Cf := 1.18
Round member factor (AASHTO 13.6.4.5)

\[
F_c' = F_c \cdot C_M \cdot C_D \cdot C_F \cdot C_V \cdot C_L \cdot C_f \cdot C_{fu} \cdot C_r
\]
Allowable axial 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)

Cpx := \left(\frac{1 + \frac{F_{cEx}}{F_c'}}{2 \cdot c} \right) \sqrt{\left(\frac{1 + \frac{F_{cEx}}{F_c' \cdot c}}{(2 \cdot c)^2}ight)^2 - \frac{\left(\frac{F_{cEx}}{F_c'} \cdot c\right)^2}{c}}
x-axis column stability factor

Cpy := \left(\frac{1 + \frac{F_{cEy}}{F_c'}}{2 \cdot c} \right) \sqrt{\left(\frac{1 + \frac{F_{cEy}}{F_c' \cdot c}}{(2 \cdot c)^2}ight)^2 - \frac{\left(\frac{F_{cEy}}{F_c'} \cdot c\right)^2}{c}}
y-axis column stability factor

Cpy = 0.988

Controls

F_c' = 0.978 ksi

Cpx = 0.991
x-axis column stability factor

Cpy = 0.988
y-axis column stability factor

\[
C_p = \frac{0.978}{0.988} = 0.991
\]
Controls

F_c' = 0.978 ksi
Allowable axial stress
\( C_{fu} := 1.0 \)  
For sawn lumber only

\( C_r := 1.0 \)  
For sawn lumber only

\[ F_{b'} := F_b' \cdot C_M \cdot C_D \cdot C_F \cdot C_Y \cdot C_L \cdot C_r \cdot C_{fu} \cdot C_r \]  
\[ F_{b'} = 1.859 \, \text{ksi} \]  
\textbf{Allowable bending stress}

\textbf{INTERACTION EQUATION VALIDATION CHECK}

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

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

\( E' = 1600 \, \text{ksi} \)

\[ R_B = \sqrt{\frac{\frac{1}{e \cdot d}}{\frac{b^2}{2}}} \]  
\text{(NDS Manual 3.3.3.6)}

\[ b = d = 9.75 \, \text{in} \]  
\text{(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 \( P-\Delta \) factor is used. This value must be greater than 1.0.

\textbf{x-axis bending}

\textbf{Group I and Group III (with Live Load)}

\[ P_{\Delta x1} := \frac{1}{f_{cT}} \cdot \frac{1 - \frac{f_{cT}}{F_{cEx}}}{P_{\Delta x1} = 1.03} \]  
\[ >1.0, \text{therefore OK} \]
Group II (without Live Load)

\[ P_{\Delta x2} := \frac{1}{1 - \frac{f_{cDL}}{F_{cEx}}} \]

\[ P_{\Delta x2} = 1.02 \quad > 1.0, \text{ therefore OK} \]

y-axis bending

Group III (with Live Load)

\[ P_{\Delta y1} := \frac{1}{1 - \left( \frac{f_{cT}}{F_{cEy}} \right)^2 - \left( \frac{f_{bx}}{F_{BE}} \right)^2} \]

\[ P_{\Delta y1} = 1.04 \quad > 1.0, \text{ therefore OK} \]

Group II (without Live Load)

\[ P_{\Delta y2} := \frac{1}{1 - \left( \frac{f_{cDL}}{F_{cEy}} \right)^2 - \left( \frac{f_{bx}}{F_{BE}} \right)^2} \]

\[ P_{\Delta y2} = 1.02 \quad > 1.0, \text{ therefore OK} \]

Group I Interaction equation

\[ f_{cT} = 0.507 \text{ ksi} \quad \text{Applied total axial stress} \]

\[ F_{c'} = 0.978 \text{ ksi} \quad \text{Allowable axial stress} \]

\[ f_{bx} = 0.871 \text{ ksi} \quad \text{Applied x-axis bending stress} \]

\[ F_{b'} = 1.859 \text{ ksi} \quad \text{Allowable bending stress} \]

\[ P_{\Delta x1} = 1.03 \quad \text{x-axis secondary moment factor} \]

\[ \left( \frac{f_{cT}}{F_{c'}} \right)^2 + \frac{f_{bx} \cdot P_{\Delta x1}}{F_{b'}} = 0.75 \]

\[ 0.75 \leq 1.0 \quad \text{OK} \]
Group II Interaction equation

\[
f_{\text{cDL}} = 0.274 \text{ ksi} \quad \text{Applied dead load axial stress}
\]

\[
P_{\Delta x2} = 1.02 \quad \text{x-axis secondary moment factor for dead load stress}
\]

\[
f_{\text{byW}} = 0.391 \text{ ksi} \quad \text{Applied y-axis bending stress from wind on superstructure}
\]

\[
P_{\Delta y2} = 1.02 \quad \text{y-axis secondary moment factor for dead load stress}
\]

\[
\left[ \left( \frac{f_{\text{cDL}}}{F_{c'}} \right)^2 + \frac{f_{\text{bX}} \cdot P_{\Delta x2}}{F_{b'}} + \frac{f_{\text{byW}} \cdot P_{\Delta y2}}{F_{b'}} \right] \cdot \frac{1}{1.25} = 0.62
\]

1.25 allowable overstress factor

\[
0.62 \leq 1.0 \quad \text{OK}
\]

Group III Interaction Equation

\[
P_{\Delta x1} = 1.03 \quad \text{x-axis secondary moment factor}
\]

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

\[
P_{\Delta y1} = 1.04 \quad \text{y-axis secondary moment factor}
\]

\[
\left[ \left( \frac{f_{\text{cT}}}{F_{c'}} \right)^2 + \frac{f_{\text{bX}} \cdot P_{\Delta x1}}{F_{b'}} + \frac{0.3 \cdot f_{\text{byW}} \cdot P_{\Delta y1}}{F_{b'}} + \frac{f_{\text{byWL}} \cdot P_{\Delta y1}}{F_{b'}} \right] \cdot \frac{1}{1.25} = 0.74
\]

1.25 allowable overstress factor

\[
0.74 \leq 1.0 \quad \text{OK}
\]

All interaction equations are less than or equal to 1.0. \quad \text{OK}
Anchor Rod Stress

\[ f_y := 60 \text{ ksi} \]

\[ \sigma_a := 0.55 \cdot f_y \]

\[ \sigma_a = 33.00 \text{ ksi} \]

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

\[ F = 7.63 \text{ kip} \]

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

\[ 0.150 \text{ in} < 1.50 \text{ in} \]

OK

Must check displacement at roadway elevation

1) DEAD LOAD EARTH PRESSURE
Total pile deflection at roadway elevation from active earth pressure 

\[ dr1 = 0.529 \text{ in} \]

Pile slope at estimated scour line 

\[ \theta = 0.003 \text{ rad} \]

Pile deflection at estimated scour line 

\[ d2 = 0.029 \text{ in} \]

Moment at estimated scour line 

\[ M = 11.49 \text{ ft kip} \]

Shear at estimated scour line 

\[ V = 4.31 \text{ kip} \]

Pile deflection at roadway elevation 

\[ d1 = 0.222 \text{ in} \]

Distance between anchor elevation and estimated scour line 

\[ x = 96.0 \text{ in} \]

E = 1600 ksi

I = 716.1 in^4

\[ f = 1.555 \text{ ft} \]

\[ h = 8.00 \text{ ft} \]

ES = 2.00 ft

BW = 6.00 ft

\[ w_1 = 1.077 \text{ klf} \]

\[ x := ES + BW \]

\[ d1 := \frac{w_1(x^2)}{120 \cdot h \cdot E \cdot I} \cdot 4 \cdot x^3 \]

\[ d1 = 0.222 \text{ in} \]

\[ M := \frac{1}{2} \cdot h \cdot w_1 \cdot \left( \frac{h}{3} \right) \]

\[ M = 11.49 \text{ ft kip} \]

\[ V := \frac{1}{2} \cdot h \cdot w_1 \]

\[ V = 4.31 \text{ kip} \]

\[ d2 := \frac{1}{E \cdot I} \left[ \frac{(M \cdot f^2)}{2} + \frac{(V \cdot f^3)}{3} \right] \]

\[ d2 = 0.029 \text{ in} \]

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

\[ \theta = 0.003 \text{ rad} \]

\[ d_{r1} := (d1 + d2 + \theta \cdot x) \]

\[ d_{r1} = 0.529 \text{ in} \]

Distance between anchor elevation and estimated scour line

E = 1600 ksi

I = 716.1 in^4

\[ f = 1.555 \text{ ft} \]

\[ h = 8.00 \text{ ft} \]

ES = 2.00 ft

BW = 6.00 ft

\[ w_1 = 1.077 \text{ klf} \]

\[ x := ES + BW \]

\[ d1 = 0.222 \text{ in} \]

\[ M = 11.49 \text{ ft kip} \]

\[ V = 4.31 \text{ kip} \]

\[ d2 = 0.029 \text{ in} \]

\[ \theta = 0.003 \text{ rad} \]

\[ d_{r1} = 0.529 \text{ in} \]
2) LIVE LOAD SURCHARGE

Part a)

\[ d_1 := \frac{w_3(x^2)}{24E I} (3x^2) \]

\[ d_1 = 0.104 \text{ in} \quad \text{Pile deflection at roadway elevation} \]

\[ M := \frac{w_3h^2}{2} \]

\[ M = 4.31 \text{ ft.kip} \quad \text{Moment at estimated scour line} \]

\[ V := w_3h \]

\[ V = 1.08 \text{ kip} \quad \text{Shear at estimated scour line} \]
Part b)

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

\[ d_2 = 0.010 \text{ in} \]

Pile deflection at estimated scour line

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

\[ \theta = 0.001 \text{ rad} \]

Pile slope at estimated scour line

\[ d_{r2} := d_1 + d_2 + \theta \cdot x \]

\[ d_{r2} = 0.210 \text{ in} \]

Total pile deflection at roadway elevation from Part a) of live load surcharge

\[ L := f + ES + BW + 1 \cdot \text{ft} \]

\[ L = 10.555 \text{ ft} \]

Distance between point of fixity and 1 ft above roadway

\[ x := f + ES + BW \]

\[ x = 9.555 \text{ ft} \]

Roadway elevation above point of fixity

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

\[ d_{r3} = 0.443 \text{ in} \]

Pile deflection from Part b) of live load surcharge
Part c)

\[ d_1 := \frac{w_4 \cdot (6 \text{ ft})^2}{120 \cdot (6 \text{ ft}) \cdot E \cdot I} \cdot 121 \cdot [6 \text{ (ft)}]^3 \]
\[ d_1 = 0.144 \text{ in} \quad \text{Pile deflection at roadway elevation} \]

\[ V := \frac{1}{2} \cdot w_4 \cdot 6 \text{ ft} \]
\[ V = 2.41 \text{ kip} \quad \text{Shear at stream elevation} \]

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

\[ x := f + ES \]
\[ x = 3.555 \text{ ft} \]

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

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

\[ d_{14} := d_1 + d_2 + \theta \cdot 6 \text{ ft} \]
\[ d_{14} = 0.738 \text{ in} \quad \text{Total pile deflection from Part c) of live load surcharge} \]
3) ANCHOR FORCE

Distance between pile fixity and anchor elevation

\[ x_1 := f + ES + Z_a \]
\[ x_1 = 6.138 \text{ ft} \]
Distance between pile fixity and roadway elevation

\[ x_2 := f + ES + BW \]
\[ x_2 = 9.555 \text{ ft} \]
Total pile deflection from anchor force

\[ d_{f5} := \frac{-F}{6E} \cdot \left(3 \cdot x_2 - x_1\right) \]
\[ d_{f5} = -1.627 \text{ in} \]

4) BRAKING FORCE

Distance between point of pile fixity and bearing elevation

\[ x_1 := f + ES + Z_b \]
\[ x_1 = 7.138 \text{ ft} \]
\[ x_2 := f + ES + BW \quad x_2 = 9.555 \text{ ft} \]

\[ d_{r6} := \frac{BFP \cdot x_1^2}{6E \cdot I} \left(3 \cdot x_2 - x_1\right) \]

\[ d_{r6} = 0.086 \text{ in} \]

Total pile deflection from braking force

5) PASSIVE EARTH PRESSURE

\[ f = 1.555 \text{ ft} \]

\[ ES = 2.00 \text{ ft} \]

\[ BW = 6.00 \text{ ft} \]

\[ H = 1.41 \text{ kip} \]

\[ \alpha := \frac{1.92 \cdot H}{f^2} \]

\[ \alpha = 1.123 \text{ ksf} \]

\[ \xi := \frac{0.12 \cdot H}{f^3} \]

\[ \xi = 0.045 \text{ kcf} \]

\[ x := f + ES + BW \]

Distance between pile fixity and roadway elevation

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

\[ d_{r7} = -0.004 \text{ in} \]

Total pile deflection from passive soil reaction

\[ d_{rT} := d_{r1} + d_{r2} + d_{r3} + d_{r4} + d_{r5} + d_{r6} + d_{r7} \]

\[ d_{rT} = 0.375 \text{ in} \]

Total pile deflection at roadway elevation

\[ 0.375 \text{ in} \leq 1.5 \text{ in} \]

OK
ANCHOR BLOCK DESIGN

S = 3.75 ft
F = 7.63 kip
Nr = 5
N = 7
b = 3.00 ft
ha := 12 in
f'_c := 3 ksi
f_y = 60 ksi

Determine Anchor Block Loads

\[ F_aT := N \cdot F \]
\[ L_{\text{min}} := N \cdot S \]
ABL := 27.00 ft

ED := 1.5 ft
S_{AR} := \frac{ABL - 2 \cdot ED}{Nr - 1} = 6.00 ft

(ANCHOR BLOCK DESIGN) (AASHTO, Section 8)

Pile spacing
Anchor force per pile
Number of anchor rods
Number of piles
Anchor height
Anchor width
Concrete compressive strength
Reinforcing steel yield strength

Total anchor force per abutment
Minimum anchor block length
Anchor block length used for analysis

Distance between end of the anchor block and exterior anchor rod
Anchor rod spacing
Resistance factor for shear (AASHTO 8.16.1.2.2)

\[ \phi_v = 0.85 \]

Resistance factor for bending (AASHTO 8.16.1.2.2)

\[ \phi_b = 0.9 \]

Resistance factor for shear (AASHTO 8.16.1.2.2)

\[ \phi_v = 0.85 \]

Factored anchor block shear

\[ V_u = 11.37 \text{ kip} \]

Factored anchor block moment

\[ M_u = 11.83 \text{ ft kip} \]

Maximum anchor block shear

\[ V = 6.73 \text{ kip} \]

Maximum anchor block moment

\[ M = 7.00 \text{ ft kip} \]

From indeterminate structural analysis:

\[ w_a = \frac{F_a T}{A_B L} \]

Passive soil reaction distribution imparted on anchor block

\[ w_a = 1.978 \text{ klf} \]

Structural Model

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

Group I Loading: 1.3(1.3E) = 1.69E  
(AASHTO 3.22)
Design Checks

FLEXURAL CAPACITY

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

Design flexural capacity (AASHTO 8.16.3.2)

Assume # 3 stirrup is used

$h_a = 12.0$ in  

$\overline{d_3} := \frac{3}{8}$ in  

$\overline{d_4} := \frac{4}{8}$ in  

$\overline{d} := h_a - 2 \cdot \overline{d_3} - \frac{1}{2} \cdot \overline{d_4}$  

$d = 9.37$ in  

$b = 3.00$ ft  

$f_c' = 3$ ksi  

$f_y = 60$ ksi

Set $M_{df}$ equal to $\phi M_n$ and determine the area of steel required by solving the resulting quadratic equation.
Therefore the minimum reinforcement requirement is satisfied.

\[ A_{sREQ} = 0.28\text{in}^2 \]

Use 3 - # 4 bars on each vertical face

\[ A_s := 3 \times 0.20\cdot\text{in}^2 \quad A_s = 0.60\text{in}^2 \quad \text{Tension steel area provided} \]

\[ \phi\!M_n := \phi_b \cdot A_s \cdot f_y \left( d - 0.60 \cdot \frac{A_s \cdot f_y}{f'c \cdot b} \right) \quad \phi\!M_n = 24.77\text{ft-kip} \quad \text{Flexure design capacity} \]

\[ 24.77\text{ft-kip} > 11.83\text{ft-kip} \quad \text{OK} \]

**REINFORCEMENT RATIO**

\[ \rho := \frac{A_s}{b \cdot d} \quad \rho = 0.0018 \quad \text{Reinforcement ratio} \]

\[ \beta_1 := 0.85 \quad (\text{AASHTO 8.16.2.7}) \]

\[ \rho_b := \frac{0.85 \cdot \beta_1 \cdot f'c}{f_y} \left( \frac{87\text{ksi}}{87\text{ksi} + f_y} \right) \quad \rho_b = 0.0214 \quad \text{Balanced reinforcement ratio} \quad (\text{AASHTO 8.16.3.2.2}) \]

\[ 0.75 \cdot \rho_b = 0.0160 \quad 0.75 \cdot \rho_b > \rho = 0.0018 \quad \text{OK} \]

**MINIMUM REINFORCEMENT**

\[ A_{sREQ} = 0.28\text{in}^2 \quad (\text{AASHTO 8.17}) \]

\[ A_s = 0.60\text{in}^2 \quad \text{Area of steel provided} \]

\[ \frac{4}{3} \cdot A_{sREQ} = 0.37\text{in}^2 \quad 0.39\text{in}^2 < 0.60\text{in}^2 \]

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

\( \phi V_n > V_u \)

\( \phi V_n = \phi V_c + \phi V_s \)

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

\( \phi V \cdot V_c = 31.43 \text{kip} \quad \text{Design concrete shear strength} \)

\( 31.43 \text{kip} > 11.37 \text{kip} \quad \text{OK} \)

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

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

\( \frac{\phi V \cdot V_c}{2} = 15.71 \text{kip} \quad 15.71 \text{kip} > 11.37 \text{kip} \quad \text{OK} \)

Therefore, no shear reinforcement (i.e., stirrups) required.
THIS WORKSHEET IS ONLY FOR TIMBER PILES IN A COHESIONLESS SOIL.

<table>
<thead>
<tr>
<th>General Bridge Input</th>
<th>1</th>
<th>Span length</th>
<th>40.00 ft</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>2</td>
<td>Roadway width</td>
<td>24.00 ft</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Location of exterior pile relative to the edge of the roadway</td>
<td>0.75 ft</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Maximum number of piles</td>
<td>10 piles on 2.50 ft centers</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Minimum number of piles</td>
<td>4 piles on 7.50 ft centers</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>Number of piles</td>
<td>7</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Backwall height</td>
<td>6.00 ft</td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>Estimated scour depth</td>
<td>2.00 ft</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>Superstructure system</td>
<td>PCDT</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>Estimated dead load abutment reaction</td>
<td>128.6 kip per abutment (default value)</td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>Dead load abutment reaction for this analysis</td>
<td>128.6 kip per abutment</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>Estimated live load abutment reaction</td>
<td>110.0 kip per abutment (default value)</td>
</tr>
<tr>
<td></td>
<td>11</td>
<td>Live load abutment reaction for this analysis</td>
<td>110.0 kip per abutment</td>
</tr>
<tr>
<td>Foundation Material Input</td>
<td>12</td>
<td>Soil SPT blow count (N)</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td>13</td>
<td>Correlated soil friction angle ($\phi$)</td>
<td>33.3 degrees</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>Soil friction angle for this analysis</td>
<td>33.3 degrees</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>Estimated friction bearing value for depths less than 30 ft</td>
<td>0.7 tons per ft</td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>Estimated friction bearing value for depths greater than 30 ft</td>
<td>0.7 tons per ft</td>
</tr>
<tr>
<td>Pile Input</td>
<td>17</td>
<td>Timber species</td>
<td>southern pine</td>
</tr>
<tr>
<td></td>
<td>18</td>
<td>Tabulated timber bending stress</td>
<td>1,750 psi</td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>Tabulated timber compressive stress</td>
<td>1,100 psi</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>Tabulated timber modulus of elasticity</td>
<td>1,600,000 psi</td>
</tr>
<tr>
<td></td>
<td>21</td>
<td>Pile butt diameter</td>
<td>13.0 in.</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>Pile tip diameter</td>
<td>10.0 in.</td>
</tr>
<tr>
<td>Lateral Restraint Input</td>
<td>23</td>
<td>Superstructure bearing elevation</td>
<td>3.58 ft</td>
</tr>
<tr>
<td></td>
<td>24</td>
<td>Type of lateral restraint system</td>
<td>buried concrete anchor block</td>
</tr>
<tr>
<td></td>
<td>25</td>
<td>Total number of anchor rods per abutment</td>
<td>5 per abutment</td>
</tr>
<tr>
<td></td>
<td>26</td>
<td>Anchor rod steel yield stress</td>
<td>60 ksi</td>
</tr>
<tr>
<td></td>
<td>27</td>
<td>Height of anchor block</td>
<td>3.00 ft</td>
</tr>
<tr>
<td></td>
<td>28</td>
<td>Bottom elevation of anchor block</td>
<td>1.08 ft</td>
</tr>
<tr>
<td></td>
<td>29</td>
<td>Anchor block lateral capacity</td>
<td>10.3 kip per pile</td>
</tr>
<tr>
<td></td>
<td>30</td>
<td>Computed anchor force per pile</td>
<td>7.6 kip per pile</td>
</tr>
<tr>
<td></td>
<td>31</td>
<td>Minimum anchor rod length</td>
<td>13.47 ft</td>
</tr>
<tr>
<td></td>
<td>32</td>
<td>Anchor rod length</td>
<td>15.00 ft</td>
</tr>
</tbody>
</table>
This worksheet is only for timber piles in a cohesionless soil.

### Design Checks

<table>
<thead>
<tr>
<th>Design Checks</th>
<th>Description</th>
<th>Value</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Axial pile load</td>
<td>$P \leq P_{ALLOWABLE}$</td>
<td>48.0 kip</td>
<td>OK</td>
</tr>
<tr>
<td>2 Pile length</td>
<td>Length $\leq 55$ ft</td>
<td>37 ft</td>
<td>OK</td>
</tr>
<tr>
<td>3 Pile bearing capacity</td>
<td>Axial Pile Load $\leq$ Capacity</td>
<td>sufficient if pile is embedded at least 34 ft</td>
<td></td>
</tr>
<tr>
<td>4 Interaction equation validation</td>
<td>$\left( \frac{1}{1 - \frac{f_c}{F'_c}} \right) &gt; 1.0$</td>
<td>1.04</td>
<td>OK</td>
</tr>
<tr>
<td>5 Combined loading interaction requirement</td>
<td>$\left( \frac{f_c}{F'<em>c} \right) + \frac{f</em>{mx}}{F'_b} \left( 1 - \frac{f_c}{F'<em>e} \right) + \frac{f</em>{my}}{F'_b} \left( 1 - \frac{f_c}{F'<em>e} - \left( \frac{f</em>{mx}}{F'_e} \right)^2 \right) \leq 1.0$</td>
<td>0.75</td>
<td>OK</td>
</tr>
<tr>
<td>6 Buried anchor block location</td>
<td>Anchor rod length $\geq$ minimum</td>
<td>15.00 ft</td>
<td>OK</td>
</tr>
<tr>
<td>7 Anchor rod stress</td>
<td>$\sigma \leq 0.55 F_Y$</td>
<td>24.2 ksi</td>
<td>OK</td>
</tr>
<tr>
<td>8 Anchor block capacity</td>
<td>Total Anchor Force $\leq$ Capacity</td>
<td>10.3 kip per pile</td>
<td>OK</td>
</tr>
<tr>
<td>9 Maximum displacement</td>
<td>$\delta_{MAX} \leq 1.5$ in.</td>
<td>0.38 in.</td>
<td>OK</td>
</tr>
</tbody>
</table>

### Foundation Summary

<table>
<thead>
<tr>
<th>Foundation Summary</th>
<th>Description</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Roadway width</td>
<td>24.00 ft</td>
<td></td>
</tr>
<tr>
<td>2 Span length</td>
<td>40.00 ft</td>
<td></td>
</tr>
<tr>
<td>3 Distance between superstructure bearings and roadway grade</td>
<td>2.42 ft</td>
<td></td>
</tr>
<tr>
<td>4 Backwall height</td>
<td>6.00 ft</td>
<td></td>
</tr>
<tr>
<td>5 Dead load abutment reaction</td>
<td>128.6 kip per abutment</td>
<td></td>
</tr>
<tr>
<td>6 Live load abutment reaction</td>
<td>110.0 kip per abutment</td>
<td></td>
</tr>
<tr>
<td>7 Number of piles</td>
<td>7</td>
<td></td>
</tr>
<tr>
<td>8 Total axial pile load</td>
<td>24.00 tons</td>
<td></td>
</tr>
<tr>
<td>9 Pile spacing</td>
<td>3.75 ft</td>
<td></td>
</tr>
<tr>
<td>10 Pile size</td>
<td>Butt diameter 13.0 in.  Tip diameter 10.0 in.</td>
<td></td>
</tr>
<tr>
<td>11 Pile material properties</td>
<td>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</td>
<td></td>
</tr>
<tr>
<td>12 Minimum total pile length</td>
<td>37 ft</td>
<td></td>
</tr>
</tbody>
</table>
THIS WORKSHEET IS ONLY TO BE USED AFTER THE PILE SYSTEM HAS BEEN DESIGNED AND ALL DESIGN REQUIREMENTS HAVE BEEN SATISFIED.

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

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

<table>
<thead>
<tr>
<th>Input Information</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Anchor block length</td>
</tr>
<tr>
<td>2</td>
<td>Distance from end of anchor block to exterior anchor rod</td>
</tr>
<tr>
<td>3</td>
<td>Concrete compressive strength</td>
</tr>
<tr>
<td>4</td>
<td>Yield strength of reinforcing steel</td>
</tr>
<tr>
<td>5</td>
<td>Tension steel area required</td>
</tr>
<tr>
<td>6</td>
<td>Number of reinforcing bars per vertical anchor block face</td>
</tr>
<tr>
<td>7</td>
<td>Tension steel bar size</td>
</tr>
<tr>
<td>8</td>
<td>Tension steel area provided</td>
</tr>
<tr>
<td>9</td>
<td>Are stirrups required?</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Design Checks</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Design flexural capacity $M_U &lt; \phi M_N$</td>
</tr>
<tr>
<td>2</td>
<td>Reinforcement ratio $\rho &lt; 0.75 \rho_r$</td>
</tr>
<tr>
<td>3</td>
<td>Minimum reinforcement</td>
</tr>
<tr>
<td>4</td>
<td>Design shear capacity $V_U &lt; \phi V_N$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Anchor System Summary</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Number of anchor rods</td>
</tr>
<tr>
<td>2</td>
<td>Anchor rod steel yield stress</td>
</tr>
<tr>
<td>3</td>
<td>Anchor rod diameter</td>
</tr>
<tr>
<td>4</td>
<td>Anchor rod length</td>
</tr>
<tr>
<td>5</td>
<td>Anchor rod spacing</td>
</tr>
<tr>
<td>6</td>
<td>Vertical distance between bottom of anchor block and roadway grade</td>
</tr>
<tr>
<td>7</td>
<td>Anchor block length</td>
</tr>
<tr>
<td>8</td>
<td>Anchor block height</td>
</tr>
<tr>
<td>9</td>
<td>Anchor block width</td>
</tr>
<tr>
<td>10</td>
<td>Concrete compressive strength</td>
</tr>
<tr>
<td>11</td>
<td>Details of reinforcement on one vertical anchor block face</td>
</tr>
</tbody>
</table>
EXAMPLE 2

STEEL PILE ABUTMENT WITHOUT ANCHORS IN A COHESIVE SOIL
EXAMPLE 2: 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

RDWy := 24ft

BW := 6ft

ES := 2ft

Slab depth = 8in

LXC := 3.75ft

Z_b := BW − 8in − LXC

Z_b = 1.583 ft

SPT := 11

FB1 := 0.7 \( \frac{\text{ton}}{\text{ft}} \)

FB2 := 0.80 \( \frac{\text{ton}}{\text{ft}} \)

DB := 40ft

N_{\text{rock}} := 150

NA := 2
GRAVITY LOADS

Dead Loads

GL := SPAN + 2·(6in)  \quad GL = 61.00 \text{ ft}  \quad \text{Girder length}

BL := GL + 2·(6in)  \quad BL = 62.00 \text{ ft}  \quad \text{Bridge length}

G := 447.0\text{plf}  \quad \text{Concrete unit weight}

N\text{\textsubscript{G}} := 4  \quad \text{Number of girders}

BR := 50\text{plf}  \quad \text{Conservatively assumed thrie-beam weight per ft}

P\text{\textsubscript{sw}} := 0.042\text{klf}  \quad \text{HP10 x 42 wt. per foot}

FWS := 20\text{psf}  \quad \text{Assumed future wearing surface}

\gamma\text{\textsubscript{c}} := 0.150\text{pcf}  \quad \text{Concrete unit weight}

\text{Slab} := (8\text{in}) \cdot BL \cdot \text{RDWY} \cdot \gamma\text{\textsubscript{c}}  \quad \text{Slab} = 148.80 \text{kip}  \quad \text{Calculated slab weight}

\text{Girder} := N\text{\textsubscript{G}} \cdot G \cdot GL  \quad \text{Girder} = 109.07 \text{kip}  \quad \text{Calculated girder weight}

\text{Rail} := 2 \cdot BR \cdot BL  \quad \text{Rail} = 6.20 \text{kip}  \quad \text{Calculated barrier rail weight}

\text{FWS}\text{\textsubscript{wt}} := \text{FWS} \cdot \text{RDWY} \cdot BL  \quad \text{FWS}\text{\textsubscript{wt}} = 29.76 \text{kip}  \quad \text{Calculated future wearing surface weight}
Diaphragm := (18in)·LXC·RDWY·γ_c·NA  
Diaphragm = 40.50 kip  
Calculated end diaphragm weight (for conservative weight calculations only)

Cap := (3ft)·(3ft)·RDWY·γ_c·NA  
Cap = 64.80 kip  
Calculated pile cap wt.

Wale := 2·(20plf)·RDWY·NA  
Wale = 1.92 kip  
Calculated abutment wale weight (2, 20 plf wales per abutment)

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

DL_{gb} = 401.05 kip  
Bridge dead load

DL_g := \frac{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)

\[ \Sigma M_B = 0 = (8\text{kips}) \cdot (60\text{ft} - 28\text{ft}) + (32\text{kips}) \cdot (60\text{ft} - 14\text{ft}) + (32\text{kips}) \cdot (60\text{ft} - 14\text{ft}) - R_A \cdot (60\text{ft}) \]

\[ R_A := \frac{8\text{kips} \cdot (60\text{ft} - 28\text{ft}) + 32\text{kips} \cdot (60\text{ft} - 14\text{ft}) + (32\text{kips}) \cdot 60\text{ft}}{60\text{ft}} \]

\[ R_A = 60.80 \text{ kip} \]

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

\[ \frac{\text{RDWY}}{10\text{ft}} = 2.4 \]

Number of 10 ft design traffic lanes  
(AASHTO 3.6.1)
LN := 2

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 (DB + Z_b + ES)

P_{swt} = 1.83 kip (per pile)

P_{DL} := \frac{DL_g \cdot pf}{N} + 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 (0.9167\text{ft})}{(N - 1)}

S = 3.167 \text{ 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

\[ \text{EA} := (1.75\text{ft} + 8\text{in} + \text{LXC}) \cdot \text{SPAN} \]
\[ \text{EA} = 370.00\text{ft}^2 \]
Bridge superstructure elevation surface area

\[ \text{WS} := \frac{\text{EA} \cdot (50\text{psf})}{\text{NA} \cdot \text{N}} \]
\[ \text{WS} = 1.16\text{kip} \]
Wind on superstructure force per pile

WIND ON LIVE LOAD

\[ \text{LL}_w := 100\text{psf} \]
Line load applied to entire bridge length

\[ \text{WL} := \frac{\text{LL}_w \cdot \text{SPAN}}{(\text{NA} \cdot \text{N})} \]
\[ \text{WL} = 0.38\text{kip} \]
Wind on live load force per pile
Longitudinal Loads

BRAKING FORCE

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

\[ W := 0.64 \text{klf} \]

\[ F := 18 \text{kip} \]

\[ \text{BFP} := \frac{\ln(W \cdot \text{SPAN} + F) \cdot 0.05}{\text{NA} \cdot N} \]

\[ \text{BFP} = 0.35 \text{kip} \quad \text{Braking force per pile} \]

DEAD LOAD EARTH PRESSURE

\[ P_1 = (35.9 \text{ h}) \text{ psf} \]

\[ h := BW + ES \]

\[ P_1 := (35.9 \text{pcf}) \cdot h \]

\[ P_1 = 287.20 \text{ psf} \]

\[ w_1 := P_1 \cdot S \]

\[ w_1 = 0.909 \text{ klf} \]

\[ \text{EDL} := \frac{1}{2} w_1 \cdot h \]

\[ \text{EDL} = 3.64 \text{ kip} \quad \text{Total lateral force per pile from active earth pressure} \]
LIVE LOAD SURCHARGE

Backwall

Roadway elevation

Estimated scour line

1 ft

6 ft

h - 6 ft

250 psf

35.9 psf

w_2 := (250 \text{psf}) \cdot S

w_2 = 0.792 \text{ klf}

Convert soil pressures into distributed loads

w_3 := (35.9 \text{psf}) \cdot S

w_3 = 0.114 \text{ klf}

w_4 := w_2 - w_3

w_4 = 0.678 \text{ klf}

LL_{sur} := (1 \text{ft}) \cdot w_2 + \frac{1}{2} (w_4) \cdot BW + h \cdot w_3

LL_{sur} = 3.74 \text{ kip}

Total lateral force per pile from live load surcharge

DETERMINE DEPTH TO PILE FIXITY

L_f = f + 1.5 \cdot B

f = \frac{H}{9 \cdot C_u \cdot B}

For a cohesive soil
(Broms, 1964)

H := BFP + LL_{sur} + EDL

H = 7.73 \text{ kip}

Total lateral force per pile

P_{ATM} := 14.69 \text{psi}

Atmospheric pressure

C_u := 0.06 \cdot \text{SPT} \cdot P_{ATM}

C_u = 1396 \text{psf}

Undrained soil shear strength

B := 10.1 \text{in}

HP 10x42 pile width

f := \frac{H}{9 \cdot C_u \cdot B}

f = 8.8 \text{in}
\[ L_f := f + 1.5 \cdot B \quad \text{or} \quad L_f = 1.993 \text{ ft} \]

**DETERMINE MAXIMUM PILE MOMENT**

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

**Longitudinal Bending Moment**

**EARTH DEAD LOAD**

\[ M_{EDL} := \frac{1}{2} \cdot h \cdot w_1 \left( L_f + \frac{1}{3} \cdot h \right) \]

\[ M_{EDL} = 16.95 \text{ ft kip} \]

\[ h = 8.000 \text{ ft} \]

\[ L_f = 1.993 \text{ ft} \]

\[ w_1 = 0.909 \text{ klf} \]

**LIVE LOAD SURCHARGE**

\[ w_2 = 0.792 \text{ klf} \]

\[ w_3 = 0.114 \text{ klf} \]

\[ w_4 = 0.678 \text{ klf} \]

\[ L_f = 1.993 \text{ ft} \]

\[ ES = 2.0 \text{ ft} \]

\[ BW = 6.0 \text{ ft} \]

\[ h = 8.0 \text{ ft} \]
Part a)

\[ M_{LLA} := w_3 \left( h \left( L_f + \frac{h}{2} \right) \right) \]

\[ M_{LLA} = 5.45 \text{ ft·kip} \]

Part b)

\[ M_{LLB} := w_2 \left( 1 \text{ ft} \right) \left( h + \frac{1\text{ ft}}{2} + L_f \right) \]

\[ M_{LLB} = 8.31 \text{ ft·kip} \]
Part c)

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

\[ M_{LLC} = 16.26 \text{ ft.kip} \]

**BRAKING FORCE**

\[ x_4 := L_f + ES + Z_b \]

\[ x_4 = 5.576 \text{ ft} \]

\[ M_{BF} := BFP \cdot x_4 \]

\[ M_{BF} = 1.97 \text{ ft.kip} \]

- BFP = 0.35 kip
- \( L_f = 1.993 \text{ ft} \)
- ES = 2.0 ft
- \( Z_b = 1.583 \text{ ft} \)

Distance between pile fixity and bearing elevation
Pile total axial load $PT = PT_{PDL} + PT_{PLL}$

- $PT_{PDL} = 36.04$ kip
- $PT_{PLL} = 19.760$ kip

Load Summary

- $P_T = P_{DL} + P_{LL}$
- $P_T = 55.80$ kip
- $P_{DL} = 36.04$ kip
- $P_{LL} = 19.760$ kip

Transverse Pile Moments

- $W_S = 10.576$ klf
- $M_{PE} = -2.82$ ft·kip
- $M = 46.11$ ft·kip

- $w_5 = 9 \cdot C_u \cdot B$
- $w_5 = 0.730$ ft
- $L_f = 1.993$ ft
- $C_u = 1396$ psf
- $B = 10.1$ in

Estimated scour line $f_0 = 0.730$ ft

Total longitudinal pile moment

- $M := M_{EDL} + M_{LLA} + M_{LLB} + M_{LLC} + M_{BF} + M_{PE}$
- $M = 46.11$ ft·kip

- $w_5 := 9 \cdot C_u \cdot B$
- $w_5 = 0.730$ ft
- $L_f = 1.993$ ft
- $C_u = 1396$ psf
- $B = 10.1$ in

Passive soil pressure

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

Wind on live load force per pile

- $W_S = 1.16$ kip

Wind on superstructure force per pile

- $W_L = 0.38$ kip

Wind on superstructure transverse pile moment

- $M_{WS} := W_S \cdot (L_f + ES + Z_b)$
- $M_{WS} = 6.45$ ft·kip

Wind on live load transverse pile moment

- $M_{WL} := W_L \cdot (L_f + ES + Z_b)$
- $M_{WL} = 2.09$ ft·kip
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)

\[ A := 12.4 \text{in}^2 \]
\[ f_a := \frac{P_T}{A} \]
\[ f_a = 4.50 \text{ ksi} \]

Total axial pile stress

\[ 4.50 \text{ksi} < 9 \text{ksi} \]  OK

Pile Bearing Capacity

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

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

\[ 111.6 \frac{\text{kips}}{\text{pile}} > 55.80 \frac{\text{kips}}{\text{pile}} \]  OK

Combined Axial and Lateral Loading Check

Two interaction equations are cited in AASHTO.

\[ \frac{f_a}{F_a} + \frac{C_{mx} (f_{bx})}{F_{bx}} + \frac{C_{my} f_{by}}{F'_{by}} \leq 1.0 \]  (AASHTO 10.36)

\[ \frac{f_a}{0.472 F_a} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \]  (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

$P_{LL} = 19.76$ kip Axial pile live load
$P_{DL} = 36.04$ kip Axial pile dead load
$P_T = 55.80$ kip Axial pile total load
$A = 12.40 \text{ in}^2$ Pile area

GROUP I AND III (with live load)

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

GROUP II (without live load)

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

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**

- $F_y := 36 \text{ksi}$
- $E := 29000 \text{ksi}$
- $d := 9.7 \text{in}$
- $t_f := 0.420 \text{in}$
- $B := 10.1 \text{in}$
- $D := d - 2 \cdot t_f$
- $D = 8.860 \text{ in}$
- $t_w := 0.415 \text{in}$
- $I := 210 \text{in}^4$
- $SM_x := 43.4 \text{in}^3$
- $SM_y := 14.2 \text{in}^3$
- $r_x := 4.13 \text{in}$
- $r_y := 2.41 \text{in}$
- $M = 46.11 \text{ ft-kip}$
- $M_{WS} = 6.45 \text{ ft-kip}$
- $M_{WL} = 2.09 \text{ ft-kip}$

- $f_{bx} := \frac{M}{SM_x}$
  \[ f_{bx} = 12.75 \text{ ksi} \]
  - Groups I, II, and III applied $x$-axis bending stress

- $f_{byWS} := \frac{M_{WS}}{SM_y}$
  \[ f_{byWS} = 5.45 \text{ ksi} \]
  - Group II and III applied $y$-axis bending stress from wind on superstructure

- $f_{byWL} := \frac{M_{WL}}{SM_y}$
  \[ f_{byWL} = 1.77 \text{ ksi} \]
  - Group III applied $y$-axis bending stress from wind on live load

$Pile steel yield stress$

$Modulus of elasticity$

$HP10 \times 47$ depth

$Flange thickness$

$Pile width$

$Depth of web$

$Web thickness$

$Pile moment of inertia$

$Strong axis section modulus$

$Weak axis section modulus$

$Strong axis radius of gyration$

$Weak axis radius of gyration$

$Maximum pile moment$ ($x$-axis bending)$

$Wind on superstructure pile moment$ ($y$-axis bending)$

$Wind on live load pile moment$ ($y$-axis bending)$

$Groups I, II, and III applied$ $x$-axis bending stress$

$Group II and III applied$ $y$-axis bending stress$ $from wind on superstructure$

$Group III applied$ $y$-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.

\[
L_f = 1.993 \text{ ft} \quad h = 8.0 \text{ ft}
\]
\[
ES = 2.0 \text{ ft} \quad Z_b = 1.583 \text{ ft}
\]
\[
k_x := 2.0
\]
\[
k_y := 0.7
\]
\[
l_x := L_f + h \quad l_x = 9.993 \text{ ft} \quad \text{Distance between point of fixity and roadway elevation}
\]
\[
l_y := L_f + ES + Z_b \quad l_y = 5.576 \text{ ft} \quad \text{Distance between point of fixity and pile cap}
\]
\[
SR_x := \frac{k_x l_x}{r_x} \quad SR_x = 58.07 \quad \text{x-axis slenderness ratio}
\]
\[
SR_y := \frac{k_y l_y}{r_y} \quad SR_y = 19.44 \quad \text{y-axis slenderness ratio}
\]
\[
F'_{ex} := \frac{\pi^2 \cdot E}{2.12 \cdot SR_x^2} \quad F'_{ex} = 40.04 \text{ ksi} \quad \text{x-axis buckling stress}
\]
\[
F'_{ey} := \frac{\pi^2 \cdot E}{2.12 \cdot SR_y^2} \quad F'_{ey} = 357.39 \text{ ksi} \quad \text{y-axis buckling stress}
\]
Interaction Equation Validation Check

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

**X-AXIS BENDING**

*Group I and Group III Loading (with Live Load)*

\[
P\Delta_{x1} := \frac{1}{f_a - \frac{F_{ex}}{F_{sy}}} \quad \text{PA}_{x1} = 1.13
\]

\[
P\Delta_{x1} = 1.13 \quad 1.13 > 1.0 \quad \text{OK}
\]

*Group II Loading (without Live Load)*

\[
P\Delta_{x2} := \frac{1}{1 - \frac{f_{aDL}}{F_{ex}}} \quad \text{PA}_{x2} = 1.08
\]

\[
P\Delta_{x2} = 1.08 \quad 1.08 > 1.0 \quad \text{OK}
\]

**Y-AXIS BENDING**

*Group I and III Loading (with Live Load)*

\[
P\Delta_{y1} := \frac{1}{f_a - \frac{F_{ex}}{F_{sy}}} \quad \text{PA}_{y1} = 1.01
\]

\[
P\Delta_{y1} = 1.01 \quad 1.01 > 1.0 \quad \text{OK}
\]

*Group II loading*

\[
P\Delta_{y2} := \frac{1}{1 - \frac{f_{aDL}}{F_{ey}}} \quad \text{PA}_{y2} = 1.01
\]

\[
P\Delta_{y2} = 1.01 \quad 1.01 > 1.0 \quad \text{OK}
\]

\[
C_{mx} := 0.85 \quad \text{For beam-columns with transverse loading (AASHTO Table 10.36A)}
\]

\[
C_{my} := 0.85
\]
Allowable Compressive Stress

\[ C_c := \sqrt{\frac{2 \cdot \pi^2 \cdot E}{F_y}} \]

\[ C_c = 126.1 \]  
Column buckling coefficient

\[ \text{SR}_{\text{max}} := 58.973 \]  
Max. slenderness ratio

\[ 58.973 \leq 126.1 \]  
Therefore the following equation is used to determine the allowable compressive stress.

\[ F_a := \frac{F_y}{2.12} \left( 1 - \frac{\text{SR}_{\text{max}}}{4} \cdot \frac{F_y}{\pi^2 \cdot E} \right) \]

\[ F_a = 15.12 \text{ ksi} \]  
Allowable compressive stress

Allowable Bending Stress

\[ F_b := \frac{50 \cdot 10^6 \cdot C_b \cdot I_{yc}}{\text{SM}_x} \cdot \frac{1}{\zeta} \sqrt{0.772 \cdot \frac{J}{I_{yc}} + 9.87 \left( \frac{d}{\zeta} \right)^2} < 0.55 \cdot F_y \]

\[ F_b = 292916 \text{ psi } = 292.9 \text{ ksi} \]  
Conservatively assumed bending coefficient

\[ C_b := 1.0 \]  
Moment of inertia for compression flange about vertical axis in the plane of the web

\[ I_{yc} := \frac{1}{12} t_f \cdot B^3 \]

\[ I_{yc} = 36.06 \text{ in}^4 \]  
Length of unsupported flange (distance between pile fixity and bearing elevation)

\[ \zeta := L_f + ES + Z_b \]

\[ \zeta = 66.92 \text{ in} \]  
Pile torsional constant

\[ d = 9.70 \text{ in} \]  
Pile depth
0.55 $F_y = 19.80$ ksi \textbf{controls}

$F_b := 0.55 \cdot F_y$

\textbf{Interaction Equation #1}

\textbf{GROUP I INTERACTION LOADING}

$f_a = 4.50$ ksi

$F_a = 15.12$ ksi

$f_{bx} = 12.75$ ksi

$F_b = 19.80$ ksi

$P\Delta x_1 = 1.13$

\[
\frac{f_a}{F_a} + \frac{C_{mx} \cdot f_{bx} \cdot P\Delta x_1}{F_b} = 0.91
\]

$0.91 \leq 1.0$ \textbf{OK}

\textbf{GROUP II INTERACTION LOADING}

$f_{aDL} = 2.91$ ksi

$P\Delta x_2 = 1.08$

$f_{byWS} = 5.45$ ksi

$P\Delta y_2 = 1.01$

\[
\left( \frac{f_{aDL}}{F_a} + \frac{C_{mx} \cdot f_{bx} \cdot P\Delta x_2}{F_b} + \frac{C_{my} \cdot f_{byWS} \cdot P\Delta y_2}{F_b} \right) \left( \frac{1}{1.25} \right) = 0.81
\]

$0.81 \leq 1.0$ \textbf{OK}
GROUP III INTERACTION LOADING

\[ P_{\Delta x_1} = 1.13 \]

\[ f_{byWL} = 1.767 \text{ ksi} \]

\[ P_{\Delta y_1} = 1.013 \]

\[
\left( \frac{f_a}{F_a} + \frac{C_{mx} f_{bx} P_{\Delta x_1}}{F_b} + \frac{0.3 C_{mx} f_{byWS} P_{\Delta y_1}}{F_b} + \frac{C_{my} f_{byWL} P_{\Delta y_1}}{F_b} \right) \left( \frac{1}{1.25} \right) = 0.850
\]

1.25 allowable over-stress factor

\[ 0.85 \leq 1.0 \quad \text{OK} \]

Interaction Equation #2

GROUP I LOADING

\[ \frac{f_a}{0.472 F_y} + \frac{f_{bx}}{F_b} = 0.91 \]

\[ 0.91 \leq 1.0 \quad \text{OK} \]

GROUP II LOADING

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

\[ 0.87 \leq 1.0 \quad \text{OK} \]

GROUP III LOADING

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

\[ 0.86 \leq 1.0 \quad \text{OK} \]

All interaction equations are less than or equal to 1.0 \quad \text{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

\[
\begin{align*}
\text{E} &= 29000 \text{ ksi} \\
\text{I} &= 210.0 \text{ in}^4 \\
\text{w}_1 &= 0.909 \text{ klf} \\
\text{L}_f &= 1.993 \text{ ft} \\
\text{h} &= 8.00 \text{ ft}
\end{align*}
\]

\[
\begin{align*}
d_1 &= \frac{4 \text{w}_1 \cdot \text{h}^4}{120 \cdot \text{E} \cdot \text{I}} \\
&= \frac{4 \cdot 0.909 \cdot 8^4}{120 \cdot 29000 \cdot 210} \\
&= 0.035 \text{ in} \\
\text{M} &= \frac{1}{2} \cdot \text{h} \cdot \text{w}_1 \left( \frac{\text{h}}{3} \right) \\
&= \frac{1}{2} \cdot 8 \cdot 0.909 \cdot \frac{8}{3} \\
&= 9.70 \text{ ft-kip} \\
\text{V} &= \frac{1}{2} \cdot \text{h} \cdot \text{w}_1 \\
&= \frac{1}{2} \cdot 8 \cdot 0.909 \\
&= 3.64 \text{ kip} \\
d_2 &= \frac{1}{\text{E} \cdot \text{I}} \left[ \frac{\left( \text{M} \cdot \text{L}_f \right)^2}{2} + \frac{\left( \text{V} \cdot \text{L}_f \right)^3}{3} \right] \\
&= \frac{1}{29000 \cdot 300} \left[ \frac{9.70^2}{2} + \frac{3.64^3}{3} \right] \\
&= 0.008 \text{ in}
\end{align*}
\]

Distance between estimated scour line and roadway elevation

Pile deflection at roadway elevation

Moment at estimated scour line

Shear at estimated scour line

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

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)**
\[ d_1 := \frac{w_3 h^4}{8 E I} \quad d_1 = 0.017 \text{ in} \quad \text{Pile deflection at roadway elevation} \]

\[ M := \frac{w_3 h^2}{2} \quad M = 3.64 \text{ ft·kip} \quad \text{Moment at estimated scour line} \]

\[ V := w_3 h \quad V = 0.91 \text{ kip} \quad \text{Shear at estimated scour line} \]

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

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

\[ d_{r2} := d_1 + d_2 + \theta \cdot h \quad d_{r2} = 0.040 \text{ in} \quad \text{Total pile deflection at roadway elevation from Part a) of live load surcharge} \]

**Part b)**

\[ L := L_f + ES + BW + 1 \cdot \text{ft} \quad L = 10.993 \text{ ft} \quad \text{Distance between point of fixity and 1 ft above roadway elevation} \]
\[ x := L_f + ES + BW \]
\[ x = 9.993 \text{ ft} \quad \text{Distance between roadway elevation and point of fixity} \]
\[ d_{r3} := \frac{w_2 \cdot 1 \cdot 6 \cdot x^2}{2 \cdot E \cdot I} \left[ \left( -\frac{1}{3} \right) \cdot x - \frac{1}{2} \cdot (1 \text{ ft}) + L \right] \]
\[ d_{r3} = 0.080 \text{ in} \quad \text{Total pile deflection from Part b) of live load surcharge} \]

**Part c)**

\[ d_1 := \frac{w_4 \cdot (6\text{ ft})^2}{120 \cdot (6\text{ ft}) \cdot E \cdot I} \cdot 11 \cdot (6\text{ ft})^3 \]
\[ d_1 = 0.023 \text{ in} \quad \text{Pile deflection at roadway elevation} \]
\[ V := \frac{1}{2} \cdot w_4 \cdot (6\text{ ft}) \]
\[ V = 2.03 \text{ kip} \quad \text{Shear at stream elevation} \]
\[ M := V \cdot \left( \frac{2}{3} \right) \cdot 6\text{ ft} \]
\[ M = 8.14 \text{ ft-kip} \quad \text{Moment at stream elevation} \]
\[ x := L_f + ES \]
\[ x = 3.993 \text{ ft} \quad \text{Distance between point of pile fixity and roadway elevation} \]
\[ d_2 := \frac{1}{E \cdot I} \left( \frac{M \cdot x^2}{2} + \frac{V \cdot x^3}{3} \right) \]
\[ d_2 = 0.031 \text{ in} \quad \text{Pile deflection at stream elevation} \]
\[ \theta := \frac{1}{E \cdot I} \left[ M \cdot (x) + \frac{V \cdot x^2}{2} \right] \]
\[ \theta = 1.152 \times 10^{-3} \text{ rad} \quad \text{Pile slope at stream elevation} \]
\[ d_{r4} := d_1 + d_2 + \theta \cdot (6\text{ ft}) \]
\[ d_{r4} = 0.136 \text{ in} \quad \text{Total pile deflection from Part c) of live load surcharge} \]
BRAKING FORCE

\[ d_{r5} := \frac{BFP \cdot x_1^2}{6E \cdot I} \left( 3 \cdot x_2 - x_1 \right) \]

\[ d_{r5} = 0.013 \text{ in} \]

\[ x_1 := L_f + ES + Z_b \]

\[ x_1 = 5.576 \text{ ft} \]

\[ x_2 := h + L_f \]

\[ x_2 = 9.993 \text{ ft} \]

\[ BFP = 0.353 \text{ kip} \]

\[ L_f = 1.993 \text{ ft} \]

\[ ES = 2.0 \text{ ft} \]

\[ BW = 6.0 \text{ ft} \]

\[ Z_b = 1.583 \text{ ft} \]

\[ h = 8.0 \text{ ft} \]

Distance between point of pile fixity and bearing elevation

Distance between point of pile fixity and roadway elevation

Total pile deflection from braking force

PASSIVE EARTH PRESSURE
\[ w_5 := 9 \cdot C_u \cdot B \quad \text{and} \quad w_5 = 10.576 \text{ klf} \]

\[ d_{r6} := \frac{-w_5 \cdot f^3}{24 \cdot E \cdot l} \cdot \left[ 4 \cdot (Lf + h) - (Lf + h - f) \right] \quad d_{r6} = -0.001 \text{ in} \]

\[ d_{rT} := d_{r1} + d_{r2} + d_{r3} + d_{r4} + d_{r5} + d_{r6} \quad d_{rT} = 0.371 \text{ in} \]

Total pile deflection from passive soil reaction

Total pile deflection at roadway elevation

\[ 0.371 \text{ in} \leq 1.50 \text{ in} \]

\( \text{OK} \)
**THIS WORKSHEET IS ONLY FOR STEEL PILES IN A COHESIVE SOIL.**

<table>
<thead>
<tr>
<th><strong>General Bridge Input</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Span length</td>
<td>60.00 ft</td>
</tr>
<tr>
<td>2. Roadway width</td>
<td>24.00 ft</td>
</tr>
<tr>
<td>3. Location of exterior pile relative to the edge of the roadway</td>
<td>0.92 ft</td>
</tr>
<tr>
<td></td>
<td>Maximum number of piles</td>
</tr>
<tr>
<td></td>
<td>Minimum number of piles</td>
</tr>
<tr>
<td>4. Number of piles</td>
<td>8</td>
</tr>
<tr>
<td>5. Backwall height</td>
<td>6.00 ft</td>
</tr>
<tr>
<td>6. Estimated scour depth</td>
<td>2.00 ft</td>
</tr>
<tr>
<td>7. Superstructure system</td>
<td>prestressed girder</td>
</tr>
<tr>
<td>Estimated dead load abutment reaction</td>
<td>210.7 kip per abutment (default value)</td>
</tr>
<tr>
<td>Dead load abutment reaction for this analysis</td>
<td>210.7 kip per abutment</td>
</tr>
<tr>
<td>Estimated live load abutment reaction</td>
<td>121.5 kip per abutment (default value)</td>
</tr>
<tr>
<td>Live load abutment reaction for this analysis</td>
<td>121.5 kip per abutment</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Foundation Material Input</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>10. Soil SPT blow count (N)</td>
<td>11</td>
</tr>
<tr>
<td>11. Correlated soil un-drained shear strength (C_u)</td>
<td>1,397 psf</td>
</tr>
<tr>
<td>12. Soil undrained shear strength for this analysis</td>
<td>1,397 psf</td>
</tr>
<tr>
<td>13. Estimated friction bearing value for depths &lt; 30 ft</td>
<td>0.7 tons per ft</td>
</tr>
<tr>
<td>14. Estimated friction bearing value for depths &gt; 30 ft</td>
<td>0.8 tons per ft</td>
</tr>
<tr>
<td>15. Depth to adequate end bearing foundation material</td>
<td>40 ft</td>
</tr>
<tr>
<td>16. SPT blow count for end bearing foundation material</td>
<td>100 &lt; N &lt; 200</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Pile Input</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>17. Pile steel yield stress</td>
<td>36 ksi</td>
</tr>
<tr>
<td>18. Select pile type</td>
<td>HP10x42</td>
</tr>
<tr>
<td>19. Pile cross sectional area</td>
<td>12.4 in^2</td>
</tr>
<tr>
<td>20. Pile depth</td>
<td>9.70 in.</td>
</tr>
<tr>
<td>21. Pile web thickness</td>
<td>0.415 in.</td>
</tr>
<tr>
<td>22. Pile flange width</td>
<td>10.1 in.</td>
</tr>
<tr>
<td>23. Pile flange thickness</td>
<td>0.420 in.</td>
</tr>
<tr>
<td>24. Pile moment of inertia (strong axis)</td>
<td>210 in^4</td>
</tr>
<tr>
<td>25. Pile section modulus (strong axis)</td>
<td>43.4 in^3</td>
</tr>
<tr>
<td>26. Pile section modulus (weak axis)</td>
<td>14.2 in^3</td>
</tr>
<tr>
<td>27. Pile radius of gyration (strong axis)</td>
<td>4.13 in.</td>
</tr>
<tr>
<td>28. Pile radius of gyration (weak axis)</td>
<td>2.41 in.</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th><strong>Lateral Restraint Input</strong></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>29. Superstructure bearing elevation</td>
<td>1.58 ft</td>
</tr>
<tr>
<td>30. Type of lateral restraint system</td>
<td>no lateral restraint system</td>
</tr>
<tr>
<td>31.</td>
<td></td>
</tr>
<tr>
<td>32.</td>
<td></td>
</tr>
<tr>
<td>33.</td>
<td></td>
</tr>
<tr>
<td>34.</td>
<td></td>
</tr>
<tr>
<td>35.</td>
<td></td>
</tr>
</tbody>
</table>
### Geotechnical, Structural and Serviceability Requirements

<table>
<thead>
<tr>
<th>Design Checks</th>
<th>1 Axial pile stress</th>
<th>$\frac{P}{A} \leq \sigma_{ALL}$</th>
<th>4.49 ksi</th>
<th>OK</th>
</tr>
</thead>
<tbody>
<tr>
<td>2 Pile bearing capacity</td>
<td>Axial Pile Load ≤ Capacity</td>
<td>111.6 kip</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>3 Interaction equation validation</td>
<td>$\frac{1}{(1 - f_a/F_y)} &gt; 1.0$</td>
<td>1.13</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>4 Combined loading interaction requirement # 1</td>
<td>$\frac{f_a}{F_y} + \frac{C_{ex} f_{ex} + C_{ey} f_{ey}}{(1 - f_a/F_y) F_b} \leq 1.0$</td>
<td>0.91</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>5 Combined loading interaction requirement # 2</td>
<td>$\frac{f_a + f_{bx} + f_{by}}{0.472 F_y} \leq 1.0$</td>
<td>0.91</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>6 Buried anchor block location</td>
<td>Anchor rod length ≥ minimum</td>
<td>OK</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7 Anchor rod stress</td>
<td>$\sigma \leq 0.55 F_y$</td>
<td>N/A</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>8 Anchor block capacity</td>
<td>Total Anchor Force ≤ Capacity</td>
<td>N/A</td>
<td>OK</td>
<td></td>
</tr>
<tr>
<td>9 Maximum displacement</td>
<td>$d_{MAX} \leq 1.5$ in.</td>
<td>0.371 in.</td>
<td>OK</td>
<td></td>
</tr>
</tbody>
</table>

#### Anchor Design Worksheet
Not applicable, buried concrete anchor option not selected

### Foundation Summary

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

<table>
<thead>
<tr>
<th>Roadway width (ft)</th>
<th>Soil type (N-value)</th>
<th>Backwall plus scour height (ft)</th>
<th>Pile type</th>
<th>Number of piles</th>
<th>Pile spacing (ft)</th>
<th>*Pile size</th>
<th>Axial pile load (ton)</th>
<th>Minimum embedded length (ft)</th>
<th>Anchor rod detail</th>
<th>Anchor block detail</th>
<th>Anchor elevation above stream (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>24</td>
<td>Cohesionless</td>
<td>8</td>
<td>Steel</td>
<td>7</td>
<td>3' - 8&quot;</td>
<td>HP 10x57</td>
<td>24.1</td>
<td>43</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>Timber</td>
<td>10</td>
<td>Timber</td>
<td>7</td>
<td>3' - 10&quot;</td>
<td>13&quot;, 10&quot;</td>
<td>24.0</td>
<td>39</td>
<td>1</td>
<td>a</td>
<td>1' - 4&quot;</td>
</tr>
<tr>
<td>24</td>
<td>Cohesive</td>
<td>N=20</td>
<td>Steel</td>
<td>8</td>
<td>3' - 2&quot;</td>
<td>HP 10x53</td>
<td>21.1</td>
<td>38</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>Timber</td>
<td>10</td>
<td>Timber</td>
<td>7</td>
<td>3' - 10&quot;</td>
<td>13&quot;, 10&quot;</td>
<td>24.0</td>
<td>39</td>
<td>1</td>
<td>b</td>
<td>3' - 1&quot;</td>
</tr>
<tr>
<td>30</td>
<td>Cohesionless</td>
<td>N=20</td>
<td>Steel</td>
<td>9</td>
<td>3' - 8&quot;</td>
<td>HP 10x57</td>
<td>24.4</td>
<td>44</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>Timber</td>
<td>10</td>
<td>Steel</td>
<td>9</td>
<td>3' - 10&quot;</td>
<td>13&quot;, 10&quot;</td>
<td>24.3</td>
<td>40</td>
<td>2</td>
<td>a</td>
<td>1' - 4&quot;</td>
</tr>
<tr>
<td>30</td>
<td>Cohesive</td>
<td>N=10</td>
<td>Steel</td>
<td>10</td>
<td>3' - 2&quot;</td>
<td>HP 10x57</td>
<td>22.0</td>
<td>40</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>8</td>
<td>Timber</td>
<td>10</td>
<td>Timber</td>
<td>9</td>
<td>3' - 10&quot;</td>
<td>13&quot;, 10&quot;</td>
<td>24.3</td>
<td>40</td>
<td>2</td>
<td>b</td>
<td>3' - 1&quot;</td>
</tr>
</tbody>
</table>

* For timber piles, the two values provided refer to the pile butt and tip diameter, respectively.
Table 2. Anchor rod details.

<table>
<thead>
<tr>
<th>Detail</th>
<th>Number of Rods</th>
<th>Rod Diameter (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5</td>
<td>0.75</td>
</tr>
<tr>
<td>2</td>
<td>7</td>
<td>0.75</td>
</tr>
</tbody>
</table>

Table 3. Anchor block details.

<table>
<thead>
<tr>
<th>Detail</th>
<th>Height (ft)</th>
<th>Width (in.)</th>
<th>Flexural Steel (for each anchor block face)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Quantity</td>
</tr>
<tr>
<td>a</td>
<td>2.50</td>
<td>12</td>
<td>3</td>
</tr>
<tr>
<td>b</td>
<td>3.00</td>
<td>12</td>
<td>3</td>
</tr>
</tbody>
</table>