1. **Overview**

The purpose of Design Data Sheet ICD-2-18 is to provide information to designers regarding standard design and detailing practices for integral abutments for prestressed concrete I-beam bridges. This drawing shows geometric requirements for integral abutments including width, height, and length of the diaphragm, as well as dimensional requirements for the pile cap. Treatments of wingwalls, approach slabs, and railings are also shown on the drawings. Minimum reinforcing for the diaphragm and pile cap are shown, and this supplement includes design methodology and example calculations for piles and reinforcing. Bearing sizes and details are presented, and limitations on the use of integral abutments for prestressed concrete I-beam bridges are stated.

The following sections of this document will discuss the ICD-2-18 drawing in greater detail.

2. **Plan Preparation Requirements**

Design Data Sheets are not intended to be used as contract drawings. Project plans shall include all details, notes, and pay items needed for construction.

3. **Detail Information**

3.1 **Sheets 1/9 through 3/9**

These sheets show details for square and skewed structures with concrete parapets. The concrete parapet ends at the back face of the diaphragm. For skewed structures, a triangular-shaped (in plan view) diaphragm protrusion supports the squared-off end of the parapet. This detailing method provides adequate clearance between the diaphragm and the first post of the bridge terminal assembly, while providing support for the full-length of the parapet. The diaphragm protrusion extends down only to the level of the approach slab seat, thus allowing for installation of the neoprene sheeting on a flat surface.

Wingwalls shall be parallel to the centerline of abutment bearings. Turned-back or flared wingwalls shall not be used due to the increased rigidity of the abutment/wingwall pile group for these wingwall configurations. A 2” PEJF expansion joint shall be provided between the diaphragm and the wingwalls, and shall be located immediately outside of the edge of deck.

Curbs shall be supported on the approach slabs. The approach slab seat shall extend only to the face of curb. Therefore, the approach slab corners will be notched-out. PEJF shall be provided at the approach slab corners as shown.

3.2 **Sheets 4/9 and 5/9**

These sheets show details for square and skewed structures with twin steel tube (TST) bridge railing. Dimensions are shown to establish acceptable locations for the top-mounted TST post, the first side-mounted TST post on the bridge, and the first post of the bridge terminal assembly. Refer to Bridge Standard Drawing TST-1-99 and Roadway Standard Drawing MGS-3.1 for additional railing details.
Wingwalls shall be parallel to the centerline of abutment bearings. Turned-back or flared wingwalls shall not be used due to the increased rigidity of the abutment/wingwall pile group for these wingwall configurations. A 2” PEJF expansion joint shall be provided between the diaphragm and the wingwalls, and shall be located 1’-6” outside of the edge of deck to provide room for the top-mounted TST post on the diaphragm.

Curbs are not required when connecting a bridge terminal assembly to a TST bridge railing. The approach slab edges shall be aligned with the bridge deck edges. PEJF shall be provided at the approach slab corners as shown.

3.3 Sheet 6/9

This sheet shows representative elevation and section views of an integral abutment with concrete parapets and no skew. Dimensions are shown in Section D-D for no skew, with a reference to Sheet 7/9 for dimensions with skew. Construction joint locations are also shown, and are dependent on the bridge skew. An optional horizontal construction joint shall be shown in the diaphragm when the skew is less than 10°, since the applicable BDM Section 700 note allows the diaphragm to be poured either with or before the deck concrete. A mandatory vertical construction joint located at 1’-0” from the front face of the diaphragm shall be shown when the skew is greater than or equal to 10°. For the higher skews, per the applicable BDM Section 700 note, the contractor may elect to submit an alternate procedure that places the diaphragm and deck concrete in the same pour; however, this requires approval of the Engineer.

The following design methodologies may be used. See Section 4 for example calculations. The reinforcing steel provided shall not be less than the minimums shown on Sheet 6/9 of the Design Data Sheet.

Determine the pile spacing based on consideration of axial loads only, unless noted otherwise in the general notes on Sheet 9/9 of the Design Data Sheet. Assume that 1/3 of the approach slab dead load is supported by the abutment. Distribute superstructure dead and live loads uniformly over a length equal to the length of the diaphragm along the skew plus two times the height of the pile cap. Apply the maximum number of lanes that will fit on the superstructure and apply the multiple presence factor. Do not apply a dynamic load allowance.

For design of the horizontal reinforcing at the top and bottom of the pile cap, model the pile cap as a continuous beam with supports at each pile location. A depth equal to the minimum pile cap depth may be used for the entire length of the pile cap model. Model the dead load beam reactions as concentrated loads. Use the live load reaction per wheel and distribute the wheel loads to the beams assuming the deck to act as simple spans between beams. Place wheel loads at locations that cause highest shears and moments. Apply the multiple presence factor and dynamic load allowance.

For design of the horizontal reinforcing at the front and back faces of the diaphragm, model the diaphragm as a continuous beam with supports at each prestressed concrete I-beam location. Use a beam width of 1’, corresponding to the bottom 1’-height of the diaphragm. Calculate the earth pressure acting on the back face of the diaphragm when the bridge expands into the backfill. To calculate the bridge expansion movement, use an expansion length equal to 2/3 of the total bridge length, a temperature rise of 35°, and a factor of 1.2 corresponding to the load factor for temperature (TU)
effects for deformations. The movement required to mobilize full passive pressure may be assumed to be equal to 5% of the diaphragm height, measured from the top of the pile cap to the bottom of the approach slab. The pressure on the back face of the diaphragm may be assumed to be a linear interpolation between at-rest pressure (for zero movement) and full passive pressure (for the movement specified above). Use a load factor of 1.5 for the earth pressure. For typical backfill behind abutments (Type B granular material per CMS 503.08), use a unit weight of 0.120 kcf, an angle of internal friction (φ) of 38°, and a friction angle between fill and wall (δ) of 19°. Apply a live load surcharge equal to 50% of the surcharge specified in AASHTO LRFD 3.11.6.4.

For design of the horizontal reinforcing at the front and back faces of the pile cap, model the pile cap as a continuous beam with supports at each pile location. Use a beam width of 1’, corresponding to the bottom 1’-height of the pile cap. Calculate the earth pressure acting on the back face of the pile cap when the bridge expands into the backfill. Calculate the bridge expansion movement as described above for design of the diaphragm. The movement required to mobilize full passive pressure may be assumed to be equal to 5% of the total abutment height, measured from the bottom of the pile cap to the bottom of the approach slab. The pressure on the back face of the pile cap may be assumed to be a linear interpolation between at-rest pressure (for zero movement) and full passive pressure (for the movement specified above). Use a load factor of 1.5 for the earth pressure. Use φ and δ angles as described above for design of the diaphragm. Apply a live load surcharge equal to 50% of the surcharge specified in AASHTO LRFD 3.11.6.4.

The horizontal reinforcing at the front and back faces of the pile cap shall also meet the requirements for skin reinforcing in AASHTO LRFD 5.6.7.

For the design of the “X” bars connecting the diaphragm to the pile cap, calculate the seismic horizontal connection force per the Bridge Design Manual and the AASHTO LRFD Bridge Design Specifications. Ensure that the area of steel provided results in adequate shear friction to resist the seismic horizontal connection force at the Extreme Event Limit State.

For crack control checks, assume a Class 1 exposure condition (Y_e = 1).

3.4 Sheet 7/9

This sheet shows the information necessary to determine the diaphragm and pile cap width for skewed structures. Note that the top flange may be clipped only for WF beams and Modified AASHTO Type 4 beams. The maximum clip dimension, normal to centerline beam, shall be 6”. Do not clip the top flange of AASHTO Type 2, 3, and 4 beams. Do not clip the bottom flange for any beam type.

3.5 Sheet 8/9

This sheet shows standard bearing sizes and details to be used for integral prestressed I-beam bridges. If the actual dead load reaction exceeds the 200-kip limit stated in note (a), then provide a special design for the elastomeric bearings. The calculated vertical dead load should consider the placement sequence of the abutment diaphragm and deck. The bearing does not need to be designed for vertical loads that are placed after the abutment diaphragm has cured.

The number and thickness of elastomer layers depends on the bridge length and on the timing of abutment and pier diaphragm pours. Refer to note (b) for more specific directions.
3.6  Sheet 9/9

This sheet contains reinforcing steel details and general notes. The length limitations for integral prestressed I-beam bridges are defined. For 0° skew, the limit of 500’ for concrete bridges corresponds to the same superstructure contraction movement demand for steel bridges with a maximum length of 400’, considering a temperature fall of 90° F for steel bridges and 45° F for concrete bridges, and a shrinkage coefficient of 0.0002 in/in for concrete bridges.

Minimum pile lengths are defined, and are based on the pile type and size as well as the predominant soil type. For cases where the minimum pile lengths are satisfied, the determination of pile spacing may be based on axial loads only. For cases where the minimum pile lengths cannot be satisfied (e.g. relatively shallow depths to rock or difficult driving conditions that may prevent achieving the minimum lengths), calculations which consider the lateral loading on the piles must be provided as stated in the notes. The minimum pile lengths are based on the information provided in Publication FHWA/IN/JTRP-2004/24, Jointless and Smoother Bridges: Behavior and Design of Piles, Frosch et al, 2006.
Design Example

ICD-2-18 Integral Construction Details for Prestressed Concrete I-Beam Bridges on Flexible Abutments

The purpose of this design example is to further describe the application of the design methodologies outlined in the "Detail Information" section of this Designer Supplement.

Design Information:
- Design Loading: HL-93 Live Load and future wearing surface of 60 psf.
- 3-span bridge, end span length = 90' c/c bearings, total bridge length = 276'
- Skew = 10°
- Deck thickness = 8.5" (including 1" monolithic wearing surface)
- Average haunch thickness over length of each span = 3"
- Abutment pile cap height varies from 5' minimum at edge of deck to 5.5' maximum at crown
- Beam clip = 1", measured normal to the CL of the beam

\[
\begin{align*}
W_{SS} &= 45ft + 4in = 45.33\text{ ft} \\
W_b &= 20in \\
\theta_{skew} &= 10\text{deg} \\
W &= 49in \\
s_{beam} &= 9ft + 10in = 9.83\text{ ft} \\
d_{clip} &= 1.0in \\
W_1 &= \frac{W}{2} - d_{clip} = 23.5\text{-in} \\
N_I &= 3 \quad \text{Number of interior beams} \\
N_E &= 2 \quad \text{Number of exterior beams} \\
\left(\frac{W_{SS} - W_b}{2}\right) &= \frac{12ft}{3.5} = 3.5 \quad N_L = 3 \quad \text{Number of lanes on bridge}
\end{align*}
\]

Deck width normal to roadway alignment
Barrier width
Bridge skew
Larger beam flange width
Beam spacing
Clip distance
Distance from beam CL to flange edge
Number of interior beams
Number of exterior beams
Number of lanes on bridge
m = \begin{cases} 
1.20 & \text{if } N_L = 1 \\
1.00 & \text{if } N_L = 2 \\
0.85 & \text{if } N_L = 3 \\
0.65 & \text{otherwise} 
\end{cases}

Multiple presence factor [LRFD 3.6.1.1.2]

**Step 1: Select pile size and spacing**

*Factored Loads for Pile Spacing*

\[ L_{\text{diaph}} = \frac{W_{SS}}{\cos(\theta_{\text{skew}})} = 46.03 \text{ ft} \]

Diaphragm length measured along CL of bearing

\[ H_{\text{abut}} = \frac{5.5 \text{ ft} + 5.0 \text{ ft}}{2} = 5.25 \text{ ft} \]

Average abutment height

Width of end diaphragm and abutment per sheet 7/9 on the ICD-2-18 design data sheet:

\[ N = \max \left[ 20 \text{ in} + \left( \frac{W}{2} \cdot \tan(\theta_{\text{skew}}) \right) \cdot \cos(\theta_{\text{skew}}), 8 \text{ in} \cdot \cos(\theta_{\text{skew}}) + W_1 \cdot \sin(\theta_{\text{skew}}) + 12 \text{ in} \right] \cdot 2 = 47.92 \text{ in} \]

End diaphragm width

Deck thickness

Haunch height

Beam height

Bearing height

Approach slab thickness

End diaphragm height

**Service I Reactions at abutment from LEAP Bridge Concrete**

- \( R_{\text{beam}} = 45.7 \text{ kip} \)
- \( R_{\text{precast}_I} = 5.5 \text{ kip} \)
- \( R_{\text{precast}_E} = 6.1 \text{ kip} \)
- \( R_{\text{deck}_I} = 48.4 \text{ kip} \)
- \( R_{\text{deck}_E} = 40.3 \text{ kip} \)
- \( R_{\text{diaph}_I} = 5.1 \text{ kip} \)
- \( R_{\text{diaph}_E} = 2.6 \text{ kip} \)
- \( R_{\text{parapet}} = 11.0 \text{ kip} \)
- \( R_{\text{fws}} = 22.7 \text{ kip} \)
- \( R_{\text{lane LL}} = 93.3 \text{ kip} \)

\( \gamma_{\text{DC}} = 1.25 \quad \gamma_{\text{DW}} = 1.5 \quad \gamma_{\text{LL}} = 1.75 \)

LRFD load factors [LRFD 3.4.1]

**Component and Attachment Loads**

Superstructure:

\[ R_{\text{DC1}} = \gamma_{\text{DC}} \left[ (R_{\text{beam}} + R_{\text{precast}_I} + R_{\text{deck}_I} + R_{\text{diaph}_I} + R_{\text{parapet}}) \cdot N_I \right. \]

\[ \left. + (R_{\text{beam}} + R_{\text{precast}_E} + R_{\text{deck}_E} + R_{\text{diaph}_E} + R_{\text{parapet}}) \cdot N_E \right] = 698.12 \text{ kip} \]

\[ w_{\text{DC1}} = \frac{R_{\text{DC1}}}{L_{\text{diaph}}} = 15.17 \frac{\text{ kip}}{\text{ ft}} \]
End Diaphragm:

\[ w_{DC2} = \gamma_{DC} N_{E} H_{150pcf} = 4.34 \text{kip/ft} \]

Abutment Pile Cap:

\[ w_{DC3} = \gamma_{DC} N_{abut} H_{150pcf} = 3.94 \text{kip/ft} \]

Approach Slab:

\[ L_{AS} = 25 \text{ft} \quad \text{Approach slab length} \]
\[ t_{AS} = 1.25 \text{ft} \quad \text{Approach slab thickness} \]

\[ w_{DC4} = \gamma_{DC} \frac{L_{AS}}{3} t_{AS} H_{150pcf} = 1.95 \text{kip/ft} \]

\[ w_{DC} = w_{DC1} + w_{DC2} + w_{DC3} + w_{DC4} = 25.4 \text{kip/ft} \]

Wearing Surface and Utility Loads

Future Wearing Surface:

\[ R_{DW} = \gamma_{DW} R_{fws} (N_{L} + N_{E}) = 170.25 \text{-kip} \]

\[ w_{DW} = \frac{R_{DW}}{L_{diaph}} = 3.7 \text{-kip/ft} \]

Live Loads

\[ R_{LL} = \gamma_{LL} R_{lane_LL} N_{L} N_{m} = 416.35 \text{-kip} \]

\[ w_{LL} = \frac{R_{LL}}{L_{diaph}} = 9.04 \text{-kip/ft} \]

Total Factored Line Load

\[ w_{T} = w_{DC} + w_{DW} + w_{LL} = 38.14 \text{-kip/ft} \]

Pile Spacing

\[ R_{R_{\max}} = 310 \text{kip} \]

*if piles are not driven to bedrock, substitute \( R_{R_{\max}} \) with \( R_{endr} \) [BDM 202.2.3.2a]

\[ s_{\text{pyle}} = \frac{R_{R_{\max}}}{w_{T}} = 8.13 \text{ft} \]

\[ s_{\text{pyle}_{\text{max}}} = 8 \text{ft} \]

\[ s_{\text{pyle}} = \min(s_{\text{pyle}}, s_{\text{pyle}_{\text{max}}}) = 8 \text{ft} \]

Ultimate pile load, HP10x42 driven to bedrock [BDM 202.2.3.2a]

Maximum pile spacing [BDM 303.4.2.2]
Step 2: Design horizontal reinforcing at top and bottom of pile cap

The pile cap is designed with uniform dead load from the approach slab, diaphragm, and abutment. The superstructure loads applied to the substructure through the beam bearings are positioned to produce the maximum load effects (shear and moment) in the pile cap. The live load positions considered for the example structure are shown below. The designer shall consider unique live load positioning for each individual structure, in order to maximize the load effects in the pile cap.
Pile Cap Loading

**Abutment, Diaphragm, and Approach Slab Loads**

Pile Cap:

\[ W_{1\text{service}} = N \cdot H_{abut} \cdot 150 \text{pcf} = 3.15 \frac{\text{kip}}{\text{ft}} \quad W_{1\text{strength}} = \gamma_{DC} \cdot W_{1\text{service}} = 3.94 \frac{\text{kip}}{\text{ft}} \]

Diaphragm and Approach Slab:

\[ W_{2\text{service}} = N \cdot (h_{\text{deck}} + h_{\text{haunch}} + h_{\text{beam}} + h_{\text{bear}}) + \frac{L_{AS}}{3} \cdot I_{AS} \cdot 150 \text{pcf} = 5.04 \frac{\text{kip}}{\text{ft}} \]

\[ W_{2\text{strength}} = \gamma_{DC} \cdot W_{2\text{service}} = 6.3 \frac{\text{kip}}{\text{ft}} \]

**Component and Attachment Loads**

Interior Beams (P2, P3, P4)

\[ P_{I\text{DCservice}} = R_{\text{beam}} + R_{\text{precast}_I} + R_{\text{deck}_I} + R_{\text{diaph}_I} + R_{\text{parapet}} = 115.7 \text{kip} \]

\[ P_{I\text{DCstrength}} = \gamma_{DC} \cdot P_{I\text{DCservice}} = 144.62 \text{kip} \]

Exterior Beams (P1, P5)

\[ P_{E\text{DCservice}} = R_{\text{beam}} + R_{\text{precast}_E} + R_{\text{deck}_E} + R_{\text{diaph}_E} + R_{\text{parapet}} = 105.7 \text{kip} \]

\[ P_{E\text{DCstrength}} = \gamma_{DC} \cdot P_{E\text{DCservice}} = 132.13 \text{kip} \]

**Wearing Surface and Utility Loads**

\[ P_{\text{DWservice}} = R_{\text{fws}} = 22.7 \text{kip} \]

\[ P_{\text{DWstrength}} = \gamma_{DW} \cdot P_{\text{DWservice}} = 34.05 \text{kip} \]

**Live Loads**

Truck and lane reactions are from LEAP Bridge Concrete analysis.

\[ R_{\text{truck}} = 64.5 \text{kip} \]

\[ \text{IM} = 1.33 \]

\[ R_{\text{lane}} = 28.8 \text{kip} \]

\[ w_{\text{lane}} = \frac{R_{\text{lane}}}{10\text{ft}} = 2.88 \frac{\text{kip}}{\text{ft}} \]

\[ x = 0.5 \]
Case 1:

\[ m_1 = 1.2 \]

Beam 1:

\[ P_{11\_truck} = \frac{(9.167\,\text{ft} + 3.167\,\text{ft})}{s_{beam}} R_{\text{truck}} \cdot IM \cdot x \cdot m_1 = 64.56\,\text{kip} \]

\[ P_{11\_lane} = w_{\text{lane}} \cdot 1.333\,\text{ft} + w_{\text{lane}} \cdot 8.667\,\text{ft} \cdot \frac{5.5\,\text{ft}}{s_{beam}} = 17.8\,\text{kip} \]

\[ P_{11\_LL\text{service}} = P_{11\_truck} + P_{11\_lane} = 82.36\,\text{kip} \]

\[ P_{11\_LL\text{strength}} = \gamma_{\text{LL}} \cdot P_{11\_LL\text{service}} = 144.13\,\text{kip} \]

Beam 2:

\[ P_{21\_truck} = \frac{(0.667\,\text{ft} + 6.667\,\text{ft})}{s_{beam}} R_{\text{truck}} \cdot IM \cdot x \cdot m_1 = 38.39\,\text{kip} \]

\[ P_{21\_lane} = w_{\text{lane}} \cdot 8.667\,\text{ft} \cdot \frac{4.333\,\text{ft}}{s_{beam}} = 11\,\text{kip} \]

\[ P_{21\_LL\text{service}} = P_{21\_truck} + P_{21\_lane} = 49.39\,\text{kip} \]

\[ P_{21\_LL\text{strength}} = \gamma_{\text{LL}} \cdot P_{21\_LL\text{service}} = 86.43\,\text{kip} \]
Case 2:

Beam 1:
\[ P_{12\_truck} = \frac{(9.167\text{ft} + 3.167\text{ft})}{s_{\text{beam}}} \cdot R_{\text{truck}} \cdot IM \cdot m_2 = 53.8 \text{kip} \]
\[ P_{12\_lane} = w_{\text{lane}} \cdot 1.333\text{ft} + w_{\text{lane}} \cdot 8.667\text{ft} \left( \frac{5.5\text{ft}}{s_{\text{beam}}} \right) = 17.8 \text{kip} \]
\[ P_{12\_LL\text{service}} = P_{12\_truck} + P_{12\_lane} = 71.6 \text{kip} \]
\[ P_{12\_LL\text{strength}} = \gamma_{LL} \cdot P_{12\_LL\text{service}} = 125.3 \text{kip} \]

Beam 2:
\[ P_{22\_truck} = \frac{(0.667\text{ft} + 6.667\text{ft} + 7\text{ft} + 1\text{ft})}{s_{\text{beam}}} \cdot R_{\text{truck}} \cdot IM \cdot m_2 = 66.89 \text{kip} \]
\[ P_{22\_lane} = w_{\text{lane}} \left[ 8.667\text{ft} \left( \frac{4.333\text{ft}}{s_{\text{beam}}} \right) + 9\text{ft} \left( \frac{4.5\text{ft}}{s_{\text{beam}}} \right) \right] = 22.86 \text{kip} \]
\[ P_{22\_LL\text{service}} = P_{22\_truck} + P_{22\_lane} = 89.75 \text{kip} \]
\[ P_{22\_LL\text{strength}} = \gamma_{LL} \cdot P_{22\_LL\text{service}} = 157.06 \text{kip} \]

Beam 3:
\[ P_{32\_truck} = \frac{(2.833\text{ft} + 8.833\text{ft})}{s_{\text{beam}}} \cdot R_{\text{truck}} \cdot IM \cdot m_2 = 50.89 \text{kip} \]
\[ P_{32\_lane} = w_{\text{lane}} \cdot 1\text{ft} \left( \frac{9.333\text{ft}}{s_{\text{beam}}} \right) = 2.73 \text{kip} \]
\[ P_{32\_LL\text{service}} = P_{32\_truck} + P_{32\_lane} = 53.62 \text{kip} \]
\[ P_{32\_LL\text{strength}} = \gamma_{LL} \cdot P_{32\_LL\text{service}} = 93.83 \text{kip} \]

Beam 4:
\[ P_{42\_lane} = w_{\text{lane}} \cdot 1\text{ft} \left( \frac{0.5\text{ft}}{s_{\text{beam}}} \right) = 0.15 \text{kip} \]
\[ P_{42\_LL\text{service}} = P_{42\_lane} = 0.15 \text{kip} \]
\[ P_{42\_LL\text{strength}} = \gamma_{LL} \cdot P_{42\_LL\text{service}} = 0.26 \text{kip} \]
Case 3:

\[ m_1 = 1.2 \]

**Beam 1:**

\[ P_{13\_lane} = w_{lane} \cdot 2ft \cdot \left( \frac{1ft}{s_{beam}} \right) = 0.59\text{-kip} \]

\[ P_{13\_LL\_service} = P_{13\_lane} = 0.59\text{-kip} \]

\[ P_{13\_LL\_strength} = \gamma_{LL} \cdot P_{13\_LL\_service} = 1.03\text{-kip} \]

**Beam 2:**

\[ P_{23\_truck} = \left( \frac{s_{beam} + 3.833ft}{s_{beam}} \right) \cdot R_{truck} \cdot \text{IM} \cdot x \cdot m_1 = 71.53\text{-kip} \]

\[ P_{23\_lane} = w_{lane} \cdot \left[ 2ft \left( \frac{8.833ft}{s_{beam}} \right) + 8ft \left( \frac{5.833ft}{s_{beam}} \right) \right] = 18.84\text{-kip} \]

\[ P_{23\_LL\_service} = P_{23\_truck} + P_{23\_lane} = 90.38\text{-kip} \]

\[ P_{23\_LL\_strength} = \gamma_{LL} \cdot P_{23\_LL\_service} = 158.16\text{-kip} \]

**Beam 3:**

\[ P_{33\_truck} = \left( \frac{6ft}{s_{beam}} \right) \cdot R_{truck} \cdot \text{IM} \cdot x \cdot m_1 = 31.41\text{-kip} \]

\[ P_{33\_lane} = w_{lane} \cdot 8ft \left( \frac{4ft}{s_{beam}} \right) = 9.37\text{-kip} \]

\[ P_{33\_LL\_service} = P_{33\_truck} + P_{33\_lane} = 40.78\text{-kip} \]

\[ P_{33\_LL\_strength} = \gamma_{LL} \cdot P_{33\_LL\_service} = 71.36\text{-kip} \]
Case 4:

**Beam 1:**

\[ P_{14\_truck} = \frac{(7.167ft + 1.167ft)}{s_{beam}} \cdot R_{truck} \cdot I \cdot M \cdot m_2 = 36.35 \text{-kip} \]

\[ P_{14\_lane} = w_{lane} \cdot 9.167ft \cdot \left( \frac{4.583ft}{s_{beam}} \right) = 12.3 \text{-kip} \]

\[ P_{14\_LLservice} = P_{14\_truck} + P_{14\_lane} = 48.66 \text{-kip} \]

\[ P_{14\_LLstrength} = \gamma_{LL} \cdot P_{14\_LLservice} = 85.15 \text{-kip} \]

**Beam 2:**

\[ P_{24\_truck} = \frac{(2.667ft + 8.667ft + 7ft + 1ft)}{s_{beam}} \cdot R_{truck} \cdot I \cdot M \cdot m_2 = 84.33 \text{-kip} \]

\[ P_{24\_lane} = w_{lane} \left[ 9.167ft \cdot \left( \frac{5.25ft}{s_{beam}} \right) + 9.833ft \cdot \left( \frac{4.917ft}{s_{beam}} \right) \right] = 28.26 \text{-kip} \]

\[ P_{24\_LLservice} = P_{24\_truck} + P_{24\_lane} = 112.59 \text{-kip} \]

\[ P_{24\_LLstrength} = \gamma_{LL} \cdot P_{24\_LLservice} = 197.03 \text{-kip} \]

**Beam 3:**

\[ P_{34\_truck} = \frac{(2.833ft + 8.833ft)}{s_{beam}} \cdot R_{truck} \cdot I \cdot M \cdot m_2 = 50.89 \text{-kip} \]

\[ P_{34\_lane} = w_{lane} \left[ s_{beam} \left( \frac{4.917ft}{s_{beam}} \right) + 1ft \cdot \left( \frac{9.333ft}{s_{beam}} \right) \right] = 16.89 \text{-kip} \]

\[ P_{34\_LLservice} = P_{34\_truck} + P_{34\_lane} = 67.78 \text{-kip} \]

\[ P_{34\_LLstrength} = \gamma_{LL} \cdot P_{34\_LLservice} = 118.62 \text{-kip} \]

**Beam 4:**

\[ P_{44\_lane} = w_{lane} \cdot 1ft \cdot \left( \frac{0.5ft}{s_{beam}} \right) = 0.15 \text{-kip} \]

\[ P_{44\_LLservice} = P_{44\_lane} = 0.15 \text{-kip} \]

\[ P_{44\_LLstrength} = \gamma_{LL} \cdot P_{44\_LLservice} = 0.26 \text{-kip} \]
Beam Reactions:

Service:

Case 1:
\[
\begin{align*}
P_{11\_service} &= P_{E\_DCservice} + P_{D Wservice} + P_{11\_LLservice} = 210.76 - \text{kip} \\
P_{21\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{21\_LLservice} = 187.79 - \text{kip}
\end{align*}
\]

Case 2:
\[
\begin{align*}
P_{12\_service} &= P_{E\_DCservice} + P_{D Wservice} + P_{12\_LLservice} = 200 \text{-kip} \\
P_{22\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{22\_LLservice} = 228.15 \text{-kip} \\
P_{32\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{32\_LLservice} = 192.02 \text{-kip} \\
P_{42\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{42\_LLservice} = 138.55 \text{-kip}
\end{align*}
\]

Case 3:
\[
\begin{align*}
P_{13\_service} &= P_{E\_DCservice} + P_{D Wservice} + P_{13\_LLservice} = 128.99 \text{-kip} \\
P_{23\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{23\_LLservice} = 228.78 \text{-kip} \\
P_{33\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{33\_LLservice} = 179.18 \text{-kip}
\end{align*}
\]

Case 4:
\[
\begin{align*}
P_{14\_service} &= P_{E\_DCservice} + P_{D Wservice} + P_{14\_LLservice} = 177.06 \text{-kip} \\
P_{24\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{24\_LLservice} = 250.99 \text{-kip} \\
P_{34\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{34\_LLservice} = 206.18 \text{-kip} \\
P_{44\_service} &= P_{1\_DCservice} + P_{D Wservice} + P_{44\_LLservice} = 138.55 \text{-kip}
\end{align*}
\]

Strength:

Case 1:
\[
\begin{align*}
P_{11\_strength} &= P_{E\_DCstrength} + P_{D Wstrength} + P_{11\_LLstrength} = 310.31 \text{-kip} \\
P_{21\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{21\_LLstrength} = 265.1 \text{-kip}
\end{align*}
\]

Case 2:
\[
\begin{align*}
P_{12\_strength} &= P_{E\_DCstrength} + P_{D Wstrength} + P_{12\_LLstrength} = 291.48 \text{-kip} \\
P_{22\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{22\_LLstrength} = 335.73 \text{-kip} \\
P_{32\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{32\_LLstrength} = 272.51 \text{-kip} \\
P_{42\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{42\_LLstrength} = 178.93 \text{-kip}
\end{align*}
\]

Case 3:
\[
\begin{align*}
P_{13\_strength} &= P_{E\_DCstrength} + P_{D Wstrength} + P_{13\_LLstrength} = 167.2 \text{-kip} \\
P_{23\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{23\_LLstrength} = 336.83 \text{-kip} \\
P_{33\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{33\_LLstrength} = 250.04 \text{-kip}
\end{align*}
\]

Case 4:
\[
\begin{align*}
P_{14\_strength} &= P_{E\_DCstrength} + P_{D Wstrength} + P_{14\_LLstrength} = 251.32 \text{-kip} \\
P_{24\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{24\_LLstrength} = 375.71 \text{-kip} \\
P_{34\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{34\_LLstrength} = 297.29 \text{-kip} \\
P_{44\_strength} &= P_{1\_DCstrength} + P_{D Wstrength} + P_{44\_LLstrength} = 178.93 \text{-kip}
\end{align*}
\]
Beam loads when beam does not carry live loads:

\[
\begin{align*}
    P_{I\text{ service}} &= P_{I\text{ DC} \text{ service}} + P_{D\text{ service}} = 138.4 \text{-kip} \\
    P_{E\text{ service}} &= P_{E\text{ DC} \text{ service}} + P_{D\text{ service}} = 128.4 \text{-kip} \\
    P_{I\text{ strength}} &= P_{I\text{ DC} \text{ strength}} + P_{D\text{ strength}} = 178.67 \text{-kip} \\
    P_{E\text{ strength}} &= P_{E\text{ DC} \text{ strength}} + P_{D\text{ strength}} = 166.17 \text{-kip}
\end{align*}
\]

Results from an external continuous beam analysis are as follows:

**Service:**

Case 1:
\[
\begin{align*}
    M_{1\text{ pos service}} &= 342.57 \text{kip-ft} \\
    M_{1\text{ neg service}} &= -282.71 \text{kip-ft}
\end{align*}
\]

Case 2:
\[
\begin{align*}
    M_{2\text{ pos service}} &= 319.26 \text{kip-ft} \\
    M_{2\text{ neg service}} &= -286.61 \text{kip-ft}
\end{align*}
\]

Case 3:
\[
\begin{align*}
    M_{3\text{ pos service}} &= 220.43 \text{kip-ft} \\
    M_{3\text{ neg service}} &= -229.59 \text{kip-ft}
\end{align*}
\]

Case 4:
\[
\begin{align*}
    M_{4\text{ pos service}} &= 279.40 \text{kip-ft} \\
    M_{4\text{ neg service}} &= -275.23 \text{kip-ft}
\end{align*}
\]

**Strength:**

Case 1:
\[
\begin{align*}
    M_{1\text{ pos strength}} &= 497.66 \text{kip-ft} \\
    M_{1\text{ neg strength}} &= -400.58 \text{kip-ft} \\
    V_{1\text{ strength}} &= 247.34 \text{kip}
\end{align*}
\]

Case 2:
\[
\begin{align*}
    M_{2\text{ pos strength}} &= 456.87 \text{kip-ft} \\
    M_{2\text{ neg strength}} &= -407.48 \text{kip-ft} \\
    V_{2\text{ strength}} &= 267.48 \text{kip}
\end{align*}
\]

Case 3:
\[
\begin{align*}
    M_{3\text{ pos strength}} &= 283.91 \text{kip-ft} \\
    M_{3\text{ neg strength}} &= -307.6 \text{kip-ft} \\
    V_{3\text{ strength}} &= 284.21 \text{kip}
\end{align*}
\]

Case 4:
\[
\begin{align*}
    M_{4\text{ pos strength}} &= 387.11 \text{kip-ft} \\
    M_{4\text{ neg strength}} &= -387.49 \text{kip-ft} \\
    V_{4\text{ strength}} &= 304.19 \text{kip}
\end{align*}
\]

**Design Forces:**

\[
\begin{align*}
    M_{\text{pos service}} &= \max(M_{1\text{ pos service}}, M_{2\text{ pos service}}, M_{3\text{ pos service}}, M_{4\text{ pos service}}) = 342.57 \text{kip-ft} \\
    M_{\text{neg service}} &= \min(M_{1\text{ neg service}}, M_{2\text{ neg service}}, M_{3\text{ neg service}}, M_{4\text{ neg service}}) = -286.61 \text{kip-ft} \\
    M_{\text{pos strength}} &= \max(M_{1\text{ pos strength}}, M_{2\text{ pos strength}}, M_{3\text{ pos strength}}, M_{4\text{ pos strength}}) = 497.66 \text{kip-ft} \\
    M_{\text{neg strength}} &= \max(M_{1\text{ neg strength}}, M_{2\text{ neg strength}}, M_{3\text{ neg strength}}, M_{4\text{ neg strength}}) = -307.6 \text{kip-ft} \\
    V_{\text{strength}} &= \max(V_{1\text{ strength}}, V_{2\text{ strength}}, V_{3\text{ strength}}, V_{4\text{ strength}}) = 304.19 \text{kip}
\end{align*}
\]
Calculate the flexural resistance provided using the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet.

Minimum Reinforcement [LRFD 5.6.3.3]

\[ f'_{cA} = 4 \text{ksi} \quad f_y = 60 \text{ksi} \]

\[ \gamma_3 = 0.67 \quad \gamma_1 = 1.6 \]

\[ N = 48 \text{-in} \quad H_{\text{min}} = 5 \text{ft} \]

Pile cap dimensions

\[ S_c = \frac{N \cdot H_{\text{min}}}{6} = 28800 \text{-in}^3 \]

Pile cap section modulus

\[ f_r = 0.24 \cdot \sqrt[2]{f'_{cA} \text{ksi}} = 480 \text{ psi} \]

Modulus of rupture [LRFD 5.4.2.6]

\[ M_{cr} = \gamma_3 \gamma_1 \cdot f_r \cdot S_c = 1234.94 \text{-kip-ft} \]

[LRFD 5.6.3.3]

\[ M_{\text{min,steel}} = \min(M_{cr}, 1.33 \cdot M_{\text{pos,strength}}) = 661.89 \text{-kip-ft} \]

(4 - #8 bars)

\[ A_s = 0.79 \text{in}^2 \cdot 4 = 3.16 \text{in}^2 \]

\[ b = N = 48 \text{-in} \]

\[ a = \frac{A_s \cdot f_y}{0.85f'_{cA} \cdot b} = 1.16 \text{-in} \]

\[ d_5 = 0.625 \text{in} \quad d_8 = 1 \text{in} \]

\[ d_s = H_{\text{min}} - 3 \text{in} - d_5 - \frac{d_8}{2} = 55.875 \text{-in} \]

\[ M_n = A_s f_y \left( d_s - \frac{a}{2} \right) = 873.65 \text{-kip-ft} \]

[LRFD 5.6.3.2.3]

\[ \phi = 0.9 \]

\[ M_r = \phi \cdot M_n = 786.28 \text{-kip-ft} \]

[LRFD Eqn. 5.6.3.2.1-1]

Since \( M_r > M_u \) and \( M_r > M_{\text{min,steel}} \), the minimum reinforcing steel (4 - #8 bars) is adequate for the strength limit state.

Check spacing of reinforcement for crack control at the service limit state.

\[ d_c = 3 \text{in} + d_5 + \frac{d_8}{2} = 4.125 \text{-in} \]

\[ \beta_s = 1 + d_c \left[ 0.7 \left( H_{\text{min}} - d_c \right) \right] = 1.11 \]

[LRFD Eqn. 5.6.7-2]

\[ \rho = A_s \div (b \cdot d_s) = 0.00118 \]
\[ E_s = 29000 \text{ksi} \]

\[ n = \frac{E_s}{\text{ksi}} = 6.8 \]

\[ 120000 \left( \frac{\text{ft}^3}{\text{kip}} \right)^2 \cdot \left( 0.15 \frac{\text{kip}}{\text{ft}^3} \right)^2 \cdot \left( f'cA \right)^{0.33} \]

\[ k = \left[ 2 \cdot \rho \cdot n + (\rho \cdot n)^2 \right]^{0.5} - \rho \cdot n = 0.12 \]

\[ j = 1 - \frac{k}{3} = 0.96 \]

\[ M_s = M_{\text{pos service}} = 342.57 \text{kip-ft} \]

\[ f_{ss} = M_s \div (A_s \cdot j \cdot d_s) = 24.24 \text{ksi} < 0.6 \cdot f_y = 36 \text{ksi} \]

\[ \gamma_e = 1.0 \frac{\text{kip}}{\text{in}} \]

\[ s_{\text{max}} = \frac{700 \gamma_e}{\beta_s f_{ss}} - 2 \cdot d_c = 17.87 \text{in} \]

[LRFD Eqn. 5.6.7-1]

\[ s = \frac{N - 2 \cdot (2 \cdot \text{in} - d_s - d_8 + 2)}{3} = 15.42 \text{in} \]

Spacing provided

Since the actual spacing provided, \( s_{\text{max}} \), the minimum reinforcing steel (4 - #8 bars) is adequate for the service limit state.
**Step 3 - Design stirrups in pile cap for shear due to vertical loading**

As previously shown, the maximum applied shear at the strength limit state, \( V_u = V_{\text{strength}} = 304.19 \text{ kip} \)

Calculate the shear resistance provided using the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet. For this example, the simplified procedure for nonprestressed sections [LRFD 5.7.3.4.1] will be used to calculate the shear resistance. Therefore, the section must contain at least the minimum amount of transverse reinforcement specified in LRFD 5.7.2.5.

\[
A_v = 0.31 \text{ in}^2 \cdot 2 = 0.62 \text{ in}^2 \quad 2 - \#5 \text{ bars at } 12'' \text{ c/c}
\]

\[ s_v = 12\text{ in} \]

\[ d_v = d_s - \frac{a}{2} = 55.29\text{ in} \]

\[ \beta = 2 \quad \lambda = 1.0 \quad b_v = N = 48\text{ in} \]

\[
V_c = 0.0316 \cdot \beta \cdot \lambda \cdot \sqrt{\frac{f'cA}{\text{ksi}}} \cdot b_v \cdot d_v = 335.48 \text{ kip} \\
\text{[LRFD 5.7.3.3-3]}
\]

\[ \theta_1 = 45\text{ deg} \]

\[
V_s = A_v \cdot f_y \cdot d_v \left( \frac{1}{\tan(\theta_1)} \right) + s_v = 171.41 \text{ kip} \\
\text{[LRFD C5.7.3.3-1]}
\]

\[ V_e + V_s = 506.89 \text{ kip} \]

\[ 0.25 f'_c A \cdot b_v \cdot d_v = 2654.12 \text{ kip} \]

\[ V_n = \min(V_e + V_s, 0.25 f'_c A \cdot b_v \cdot d_v) = 506.89 \text{ kip} \]

\[ \phi_v = 0.9 \]

\[ V_r = \phi_v \cdot V_n = 456.2 \text{ kip} \]  

\text{[LRFD Eqn. 5.4.2.1-1]}

Check minimum transverse reinforcement per LRFD 5.7.2.5.

\[
A_{v\text{min}} = 0.0316 \cdot \sqrt{\frac{f'cA}{\text{ksi}}} \cdot b_v \cdot \frac{s_v}{f_y} = 0.61 \text{ in}^2 \text{ per foot} \\
\text{[LRFD 5.7.2.5-1]}
\]

\[ A_v = 0.62 \text{ in}^2 \]

Since \( V_r > V_u \) and the minimum transverse reinforcement requirements of LRFD 5.7.2.5 are satisfied, the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet (#5 bars @ 12" c/c) is adequate for the strength limit state.
**Step 4 - Design horizontal reinforcing at front and back faces of diaphragm**

Anticipated movement of superstructure:

\[
L = 276	ext{ft} \quad \alpha = 0.000006 \text{ (in + in)} \quad \Delta_T = 35\text{F}
\]

\[
\Delta = \frac{2}{3} \cdot L \cdot \alpha \cdot \Delta_T \cdot 1.0 \cdot \cos(\theta_{\text{skew}}) = 0.46 \text{ in}
\]

Height of diaphragm from top of pile cap to bottom of approach slab

\[
H_D = H - t_{AS} = 54.5 \text{ in}
\]

For typical backfill behind abutments: (Type B granular material per C&MS 503.08)

\[
\phi_f' = 38\text{deg} \quad \delta = 19\text{deg} \quad \gamma_{\text{soil}} = 120\text{pcf} \quad \theta = 90\text{deg}
\]

Movement required to mobilize full passive pressure

\[
\delta_{\text{pass}} = 0.05 \cdot H_D = 2.73 \text{ in}
\]

At rest lateral earth pressure coefficient

\[
k_o = 1 - \sin(\phi_f') = 0.38 \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad [\text{LRFD 3.11.5.2-1}]
\]

Passive lateral earth pressure coefficient, \(k_p = 9.10\)

\[
k = k_o + (k_p - k_o) \cdot \frac{\Delta}{\delta_{\text{pass}}} = 1.84
\]

Design lateral earth pressure coefficient

\[
\rho_{\text{EP}} = k \cdot \gamma_{\text{soil}} \cdot H_D = 1.01 \text{ ksf}
\]

Height of surcharge

\[
H_s = 4\text{ ft} \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad \quad [\text{LRFD Table 3.11.6.4-1}]
\]

Surcharge pressure

\[
\rho_{\text{surcharge}} = 0.5 \cdot k \cdot \gamma_{\text{soil}} \cdot H_s = 0.51 \text{ ksf}
\]

The following moments are calculated based on uniformly distributed loads applied to a continuous beam with supports at each prestressed concrete I-beam location. The moments are calculated a 1 foot high strip located at the top of the pile cap.

\[
M_{\text{EP}} = 0.11 \cdot \rho_{\text{EP}} \cdot 1\text{ft} \left( \frac{s_{\text{beam}}}{\cos(\theta_{\text{skew}})} \right)^2 = 11.03 \text{ kip-ft}
\]

Moment due to earth pressure

\[
M_{\text{surcharge}} = 0.11 \cdot \rho_{\text{surcharge}} \cdot 1\text{ft} \left( \frac{s_{\text{beam}}}{\cos(\theta_{\text{skew}})} \right)^2 = 5.62 \text{ kip-ft}
\]

Moment due to surcharge

\[
M_s = M_{\text{EP}} + M_{\text{surcharge}} = 16.7 \text{ kip-ft}
\]

Total service Moment

\[
M_u = 1.5 \cdot M_{\text{EP}} + 1.75 \cdot M_{\text{surcharge}} = 26.4 \text{ kip ft}
\]

Total factored moment

\[
S_c = \frac{1\text{ft} \cdot N^2}{6} = 4608 \text{ in}^3
\]

Pile cap section modulus

\[
f_{cD} = 4.5\text{ksi}
\]

\[
f_r = 0.24 \cdot \sqrt[3]{\frac{f_{cD}}{\text{ksi}}} = 509.12 \text{ psi}
\]

Modulus of rupture [LRFD 5.4.2.6]
\[ M_{cr} = \gamma_3 \cdot \gamma_1 \cdot f_y \cdot S_c = 209.58 \text{kip-ft} \]  
[LRFD 5.6.3.3]

\[ 1.33 \cdot M_u = 35.09 \text{kip-ft} \]

\[ M_{\text{min_steel}} = \min(M_{cr}, 1.33 \cdot M_{\text{pos_strength}}) = 209.58 \text{kip-ft} \]

Calculate the flexural resistance provided using the minimum reinforcing steel specified on sheet 6/9 of the Design Data Sheet.

\[ A_s = 0.79 \text{in}^2 \quad (\#8 \text{bars @ 12"}) \quad b = 12\text{in} \]

\[ a = \frac{A_s \cdot f_y}{0.85 f_{cD} b} = 1.03 \text{in} \]  
[LRFD 5.6.2.2 & Eqn. 5.6.3.1.2-4]

\[ d_s = N - 2b - \frac{d_8}{2} = 44.875 \text{in} \]

\[ M_n = A_s \cdot f_y \left( \frac{d_s - a}{2} \right) = 175.22 \text{kip-ft} \]  
[LRFD 5.6.3.2.3]

\[ \phi = 0.9 \]

\[ M_r = \phi \cdot M_n = 157.7 \text{kip-ft} \]  
[LRFD Eqn. 5.6.3.2.1-1]

Since \( M_r > M_u \) and \( M_r > M_{\text{min_steel}} \), the minimum reinforcing steel (4 - #8 bars) is adequate for the strength limit state.

Check spacing of reinforcement for crack control at the service limit state.

\[ d_c = 2b + d_5 + \frac{d_8}{2} = 3.125 \text{in} \]

\[ \beta_s = 1 + d_c + \left[ 0.7 \left( N - d_c \right) \right] = 1.1 \]  
[LRFD Eqn. 5.6.7-2]

\[ \rho = A_s + (b \cdot d_s) = 0.00147 \]

\[ E_s = 29000\text{ksi} \]

\[ n = \frac{E_s}{k_s \cdot f_{cD}^2} = 6.54 \]

\[ 120000 \left( \frac{f_c^3}{\text{kip}} \right)^2 \left( \frac{0.15 \text{kip}}{\text{ft}} \right)^2 \left( \frac{f_cD}{\text{ksi}} \right)^{0.33} \]

\[ k = \left[ 2 \cdot \rho \cdot n + (\rho \cdot n)^2 \right]^{0.5} = 0.13 \]

\[ j = 1 - \frac{k}{3} = 0.96 \]

\[ f_{ss} = M_s + (A_s \cdot j \cdot d_s) = 6.51 \text{ksi} < 0.6 \cdot f_y = 36 \text{ksi} \]

\[ \gamma_{e} = 1.0 \text{kip/in} \]

\[ s_{\text{max}} = \frac{700 \cdot \gamma_{e}}{\beta_s f_{ss}} - 2 \cdot d_c = 91.58 \text{in} \]  
[LRFD Eqn. 5.6.7-1]

\[ s_v = 12 \text{in} \]

Spacing provided

Since the actual spacing provided, \( s_{\text{max}} \), the minimum reinforcing steel (#8 bars @ 12") is adequate for the service limit state.
Step 5 - Design stirrups in diaphragm for shear due to horizontal loading

\[
V_{EP} = 0.6 \cdot \rho_{EP} \cdot \text{ft} \cdot \frac{s_{\text{beam}}}{\cos(\theta_{\text{skew}})} = 6.98 \cdot \text{kip}
\]
Shear due to earth pressure

\[
V_{\text{surcharge}} = 0.6 \cdot \rho_{\text{surcharge}} \cdot \text{ft} \cdot \frac{s_{\text{beam}}}{\cos(\theta_{\text{skew}})} = 3.07 \cdot \text{kip}
\]
Shear due to surcharge

\[
V_u = 1.5 \cdot V_{EP} + 1.75 \cdot V_{\text{surcharge}} = 15.84 \cdot \text{kip}
\]
Total factored shear

Calculate the shear resistance provided using the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet. For this example, the simplified procedure for nonprestressed sections [LRFD 5.7.3.4.1] will be used to calculate the shear resistance. Therefore, the section must contain at least the minimum amount of transverse reinforcement specified in LRFD 5.7.2.5.

\[
A_v = 0.62 \cdot \text{in}^2 \quad s_v = 12 \cdot \text{in} \quad 2 \cdot \#5 \text{ legs @ 12'' c/c}
\]

\[
d_v = d_s - \frac{a}{2} = 44.36 \cdot \text{in}
\]

\[
\beta = 2 \quad \lambda = 1.0 \quad b_v = 12 \text{in}
\]

\[
\theta_1 = 45 \text{deg}
\]

\[
V_c = 0.0316 \cdot \beta \cdot \lambda \cdot \frac{f_{cD}}{\text{ksi}} \cdot b_v \cdot d_v = 71.36 \cdot \text{kip}
\]  
[LRFD 5.7.3.3-3]

\[
V_s = A_v \cdot f_{y} \cdot d_v \left( \frac{1}{\tan(\theta_1)} \right) + s_v = 137.51 \cdot \text{kip}
\]  
[LRFD C5.7.3.3-1]

\[
V_c + V_s = 208.88 \cdot \text{kip}
\]

\[
0.25f_{cD} b_v d_v = 598.84 \cdot \text{kip}
\]

\[
V_n = \min(V_c + V_s, 0.25f_{cD} b_v d_v) = 208.88 \cdot \text{kip}
\]

\[
\phi_v = 0.9 \quad V_r = \phi_v \cdot V_n = 187.99 \cdot \text{kip}
\]  
[LRFD Eqn. 5.4.2.1-1]

Check minimum transverse reinforcement requirements of LRFD 5.7.2.5. The minimum reinforcement check will be based on the full diaphragm height, rather than a 1 foot high strip.

\[
b_v = H_D = 54.5 \cdot \text{in}
\]

\[
A_{v_{\text{min}}} = 0.0316 \cdot \frac{f_{cD}}{\text{ksi}} \cdot b_v \cdot s_v = 0.73 \cdot \text{in}^2 \text{ per foot}
\]  
[LRFD 5.7.2.5-1]

\[
A_{v_{\text{min}}} \text{ exceeds the minimum reinforcing steel shown on Sheet 6/9 of the Design Data Sheet.}
\]

Try \( s_v = 10 \text{in} \)

\[
A_{v_{\text{min}}} = 0.0316 \cdot \frac{f_{cD}}{\text{ksi}} \cdot b_v \cdot s_v = 0.61 \cdot \text{in}^2
\]

Since \( V_r > V_u \) and the minimum transverse reinforcement requirements of LRFD 5.7.2.5 are satisfied, \#5 bars at 10'' is adequate for the strength limit state. As an alternative to providing the minimum reinforcement per LRFD 5.7.2.5, the designer may elect to calculate the shear resistance based on the general procedure outlined in LRFD 5.7.3.4.2, using Eqn. 5.7.3.4.2-2 for \( \beta \), which is for sections that do not contain the minimum amount of shear reinforcement as required under LRFD 5.7.2.5. In all cases, at a minimum, the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet shall be provided.
Step 6 - Design horizontal reinforcing at front and back faces of pile cap

\[ \Delta = 0.46 \text{ in} \]

Height of abutment from bottom of pile cap to bottom of approach slab

\[ H_A = H_D + H_{\text{min}} = 9.54 \text{ ft} \]

For typical backfill behind abutments: (Type B granular material per C&MS 503.08)

\[ \phi'_f = 38 \text{deg} \quad \delta = 19 \text{deg} \quad \gamma_{\text{soil}} = 120 \text{pcf} \]

Movement required to mobilize full passive pressure

\[ \delta_{\text{pass}} = 0.05 \cdot H_A = 5.72 \text{ in} \]

At rest lateral earth pressure coefficient

\[ k_o = 1 - \sin(\phi'_f) = 0.38 \]

Passive lateral earth pressure coefficient, \( k_p = 9.10 \) \[ \text{LRFD Fig 3.11.5.4-1} \]

\[ k = k_o + (k_p - k_o) \cdot \frac{\Delta}{\delta_{\text{pass}}} = 1.08 \]

Design lateral earth pressure coefficient

\[ \rho_{\text{EP}} = k \cdot \gamma_{\text{soil}} \cdot H_A = 1.24 \text{ ksf} \]

Height of surcharge (Interpolate)

\[ H_s = 4 \text{ ft} + (3 \text{ ft} - 4 \text{ ft}) \cdot \left[ (H_A - H_{\text{min}}) + (10 \text{ ft} - H_{\text{min}}) \right] = 3.09 \text{ ft} \] \[ \text{LRFD Table 3.11.6.4-1} \]

\[ \rho_{\text{surcharge}} = 0.5 \cdot k \cdot \gamma_{\text{soil}} \cdot H_s = 0.23 \text{ ksf} \]

Surcharge pressure

The following moments are calculated based on uniformly distributed loads applied to a continuous beam with supports at each prestressed concrete I-beam location. The moments are calculated a 1 foot high strip located at the top of the pile cap.

\[ M_{\text{EP}} = 0.11 \cdot \rho_{\text{EP}} \cdot 1 \text{ ft} \cdot (\text{s pile})^2 = 8.70 \text{ kip-ft} \]

Moment due to earth pressure

\[ M_{\text{surcharge}} = 0.11 \cdot \rho_{\text{surcharge}} \cdot 1 \text{ ft} \cdot (\text{s pile})^2 = 1.59 \text{ kip-ft} \]

Moment due to surcharge

\[ M_s = M_{\text{EP}} + M_{\text{surcharge}} = 10.29 \text{ kip-ft} \]

Total service Moment

\[ M_u = 1.5 \cdot M_{\text{EP}} + 1.75 \cdot M_{\text{surcharge}} = 15.83 \text{ kip-ft} \]

Total factored moment

\[ S_c = \frac{1 \text{ ft} \cdot N^2}{6} = 4608 \text{ in}^3 \]

Pile cap section modulus

\[ f_{cD} = 4.5 \text{ksi} \]

\[ f_r = 0.24 \cdot \frac{f_{cA}}{\text{ksi}} = 480 \text{ psi} \]

Modulus of rupture \[ \text{LRFD 5.4.2.6} \]

\[ M_{\text{cr}} = \gamma_3 \cdot \gamma_f \cdot f_r \cdot S_c = 197.59 \text{ kip-ft} \]

\[ 1.33 \cdot M_u = 21.1 \text{ kip-ft} \]

\[ M_{\text{min_steel}} = \min \left( M_{\text{cr}}, 1.33 \cdot M_{\pos_strength} \right) = 197.59 \text{ kip-ft} \]
Calculate the flexural resistance provided using the minimum reinforcing steel specified on sheet 6/9 of the Design Data Sheet.

\[ A_s = 0.44\text{in}^2 \left( \frac{12\text{in}}{9\text{in}} \right) = 0.59\text{-in}^2 \text{per foot} \quad (#6\text{ bars @ 9"}) \]

\[ b = 12\text{in} \quad d_{6} = 0.75\text{in} \]

\[ a = \frac{A_s f_y}{0.85 f'c A} = 0.86\text{-in} \quad \text{[LRFD 5.6.2.2 & Eqn. 5.6.3.1.2-4]} \]

\[ d_s = N - 2\text{in} - d_5 - \frac{d_6}{2} = 45\text{-in} \]

\[ M_n = A_s f_y \left( d_s - \frac{a}{2} \right) = 130.73\text{-kip-ft} \quad \text{[LRFD 5.6.3.2.3]} \]

\[ \phi = 0.9 \]

\[ M_r = \phi M_n = 117.66\text{-kip-ft} \quad \text{[LRFD Eqn. 5.6.3.2.1-1]} \]

Since \( M_r > M_u \) and \( M_r > M_{\text{min,steel}} \), the minimum reinforcing steel (#6 bars at 9") is adequate for the strength limit state.

Check spacing of reinforcement for crack control at the service limit state.

\[ d_c = 2\text{in} + d_5 + \frac{d_6}{2} = 3\text{-in} \]

\[ \beta_s = 1 + d_c + \left[ 0.7 \left( h_{\text{beam}} - d_c \right) \right] = 1.1 \quad \text{[LRFD 5.6.7-2]} \]

\[ \rho = A_s + \left( b \cdot d_s \right) = 0.00109 \]

\[ E_s = 29000\text{ksi} \]

\[ n = \frac{\rho}{\frac{E_s}{\text{ksi}}} = 6.8 \]

\[ k = \left[ 2\cdot \rho \cdot n + (\rho \cdot n)^{2} \right]^{0.5} - \rho \cdot n = 0.11 \]

\[ j = 1 - \frac{k}{3} = 0.96 \]

\[ f_{ss} = M_s + \left( A_s j \cdot d_s \right) = 5.39\text{-ksi} \quad < 0.6 f_y = 36\text{-ksi} \]

\[ \gamma_e = 1.0 \frac{\text{kip}}{\text{in}} \]

\[ s_{\text{max}} = \frac{700 \gamma_e}{\beta_s f_{ss}} - 2 \cdot d_c = 112.5\text{-in} \quad \text{[LRFD Eqn. 5.6.7-1]} \]

\[ s_{\text{prov}} = 9\text{in} \quad \text{Spacing provided} \]

Since the actual spacing provided, \( s_{\text{max}} \), the minimum reinforcing steel (#6 bars at 9") is adequate for the service limit state.
**Step 7 - Design stirrups in pile cap for shear due to horizontal loading**

Shear due to earth pressure

\[ V_{EP} = 0.6 \cdot \rho_{EP} \cdot 1\text{ft} \cdot s_{pile} = 6.7\text{-kip} \]

Shear due to surcharge

\[ V_{surcharge} = 0.6 \cdot \rho_{surcharge} \cdot 1\text{ft} \cdot s_{pile} = 1.08\text{-kip} \]

Total factored shear

\[ V_u = 1.5 \cdot V_{EP} + 1.75 \cdot V_{surcharge} = 11.94\text{-kip} \]

Calculate the shear resistance provided using the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet. For this example, the simplified procedure for nonprestressed sections [LRFD 5.7.3.4.1] will be used to calculate the shear resistance. Therefore, the section must contain at least the minimum amount of transverse reinforcement specified in LRFD 5.7.2.5.

\[ A_v = 0.62\text{-in}^2 \quad s_v = 12\text{in} \quad 2 \cdot #5 \text{ legs @ } 12\text{"} \text{ c/c} \]

\[ d_v = d_s - \frac{a}{2} = 44.57\text{-in} \]

\[ \beta = 2 \quad \lambda = 1.0 \quad b_v = 12\text{in} \]

\[ V_c = 0.0316 \cdot \beta \cdot \lambda \cdot \frac{f'_c A}{\text{ksi}} \cdot b_v \cdot d_v = 67.6\text{-kip} \quad \text{[LRFD 5.7.3.3-3]} \]

\[ \theta_1 = 45\text{deg} \]

\[ V_s = A_v \cdot f_y \cdot d_v \left( \frac{1}{\tan(\theta_1)} \right) + s_v = 138.16\text{-kip} \quad \text{[LRFD C5.7.3.3-1]} \]

\[ V_c + V_s = 205.76\text{-kip} \]

\[ 0.25 f'_c A \cdot b_v \cdot d_v = 534.82\text{-kip} \]

\[ V_n = \min(V_c + V_s, 0.25 f'_c A \cdot b_v \cdot d_v) = 205.76\text{-kip} \]

\[ \phi_v = 0.9 \quad V_r = \phi_v \cdot V_n = 185.19\text{-kip} \quad \text{[LRFD Eqn. 5.4.2.1-1]} \]

Check minimum transverse reinforcement requirements of LRFD 5.7.2.5. The minimum reinforcement check will be based on the full abutment height, rather than a 1 foot high strip.

\[ b_v = H_{min} = 60\text{-in} \]

\[ A_{v\min} = 0.0316 \cdot \frac{f'_c A}{\text{ksi}} \cdot b_v \cdot s_v = 0.76\text{-in}^2 \text{ per foot} \quad \text{[LRFD 5.7.2.5-1]} \]

\[ A_{v\min} \text{ exceeds the minimum reinforcing steel shown on Sheet 6/9 of the Design Data Sheet.} \]

Try \[ s_v = 9\text{in} \]

\[ A_{v\min} = 0.0316 \cdot \frac{f'_c A}{\text{ksi}} \cdot b_v \cdot s_v = 0.57\text{-in}^2 \]

Since \[ V_r > V_u \] and the minimum transverse reinforcement requirements of LRFD 5.7.2.5 are satisfied, #5 bars at 9" is adequate for the strength limit state. As an alternative to providing the minimum reinforcement per LRFD 5.7.2.5, the designer may elect to calculate the shear resistance based on the general procedure outlined in LRFD 5.7.3.4.2, using Eqn. 5.7.3.4.2-2 for \( \beta \), which is for sections that do not contain the minimum amount of shear reinforcement as required under LRFD 5.7.2.5. In all cases, at a minimum, the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet shall be provided.
**Step 8 - Design "X" bars connecting diaphragm to pile cap**

For this example, the magnitude of the seismic horizontal connection force will be equal to 0.25 times the tributary permanent load. For an integral bridge with expansion bearings at all piers, the tributary permanent load at each abutment is equal to one-half of the total dead load of the superstructure, including a future wearing surface allowance. It will be assumed that the dead load reactions at each of the pier bearing points are the same as the dead load reactions at the abutment. From Step 2, the unfactored dead load reactions at the abutment are as follows:

\[
P_{L\,DC\text{service}} = 115.7\text{kip} \quad \text{Interior beam, DC}
\]
\[
P_{E\,DC\text{service}} = 105.7\text{kip} \quad \text{Exterior beam, DC}
\]
\[
P_{D\text{W service}} = 22.7\text{kip} \quad \text{Beam, DW}
\]
\[
W_{2\text{ service}} = 5.04\text{kip/ft} \quad \text{Dist. load for diaphragm}
\]

Therefore, for the 3-span bridge, the tributary permanent load at each abutment is:

\[
N_{bp} = 3 \cdot 2 = 6 \quad \text{Number of bearing points, number of bearing points in a beam line}
\]

\[
P_{tpl} = \left[ P_{L\,DC\text{service}}N_{I} + P_{E\,DC\text{service}}N_{E} + P_{D\text{W service}}(N_{I} + N_{E}) \right] \frac{N_{bp}}{2} + W_{2\,\text{service}}L_{\text{diaph}} = 2247.9\text{kip}
\]

Seismic horizontal connection force

\[
V_{ui} = 0.25\cdot P_{tpl} = 561.97\text{kip}
\]

From LRFD 5.7.4.4, for normal weight concrete placed against a clean surface, free of laitance, but not intentionally roughened:

\[
c = 0.075\text{ksi} \quad \mu = 0.6 \quad K_{1} = 0.2 \quad K_{2} = 0.8\text{ksi}
\]

The interface area

\[
A_{cv} = (N - 12\text{in} - 12\text{in})L_{\text{diaph}} = 13257\text{in}^{2}
\]

The minimum number of "X" bars to be provided in each bay, according to Sheet 6/9 of the Design Data Sheet, is:

\[
N_{\text{bars_B}} = \left( \frac{\text{size beam} - W - 2\cdot2\text{in}}{\cos(\theta_{\text{skew}})\cdot\text{ft}} \right) + 1 = 6.5 \quad \text{say} \quad N_{\text{bars_B}} = 7
\]

The minimum number of "X" bars to be provided at each fascia, according to Sheet 6/9 of the Design Data Sheet, is:

\[
L_{\text{OH}} = 3\text{ft} \quad \text{Deck Overhang}
\]

\[
N_{\text{bars_OH}} = \left( \frac{L_{\text{OH}} - W}{2} - 2\cdot2\text{in} \right) \cos(\theta_{\text{skew}})\cdot\text{ft} + 1 = 1.63 \quad \text{say} \quad N_{\text{bars_OH}} = 2
\]

The total number of "X" bars provided in each diaphragm is:

\[
N_{\text{bars}} = N_{\text{bars_B}}(N_{I} + N_{E} - 1) + 2\cdot N_{\text{bars_OH}} = 32
\]

Area of interface shear reinforcement

\[
A_{vf} = N_{\text{bars}}\cdot2\cdot0.44\text{in}^{2}\cdot\cos(45\text{deg}) = 19.91\text{in}^{2}
\]

\[
P_{c} = 0\text{kip}
\]
Nominal interface shear resistance

\[ V_{n_i 1} = c \cdot A_{cv} + \frac{\mu (A_{vf} f_y + P_c)}{} = 1711 \text{-kip} \]  
LRFD Eqn. 5.7.4.3-3

\[ V_{n_i 2} = K_1 f'c A_{cv} = 10606 \text{-kip} \]  
LRFD Eqn. 5.7.4.3-4

\[ V_{n_i 3} = K_2 A_{cv} = 10606 \text{-kip} \]  
LRFD Eqn. 5.7.4.3-5

\[ V_{n_i} = \min(V_{n_i 1}, V_{n_i 2}, V_{n_i 3}) = 1711 \text{-kip} \]

For the Extreme Event limit state, the resistance factor, \( \phi \), may be taken as 1.

\[ \phi_{n_i} = 1.0 \]

\[ V_{ri} = \phi_{n_i} V_{n_i} = 1711 \text{-kip} \]  
LRFD Eqn. 5.7.4.3-1

The AASHTO LRFD requirements for minimum area of interface shear reinforcement also need to be met.

\[ A_{vf \text{min}} = 0.05 \text{ksi} \frac{A_{cv}}{f_y} = 11.05 \text{-in}^2 \]  
LRFD Eqn. 5.7.4.2-1

Since \( V_{ri} > V_{ui} \) and the area reinforcing steel provided meets the AASHTO minimum, the minimum reinforcing steel specified on Sheet 6/9 of the Design Data Sheet (#6 bars @ 12") is adequate for the Extreme Event limit state.