Washington State Department of Transportation

BRIDGE 17.7 – I-405 UNDER RENTON AVE S

I-405; RENTON TO BELLEVUE WIDENING AND EXPRESS TOLL LANES PROJECT

Final Submittal
Design Calculations

December 21, 2021

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COWI

wood.

FINAL DESIGN
NOT FOR CONSTRUCTION

COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

PROJECT CALCULATION REGISTER BOOK I0

LATEST DATE: **2021 Dec 21**

Book No.	Page #'s	to	Page #'s	Title	Issued To	Issued Date	Completed Date	Comments
I0	n/a	-	n/a	**Book I0 Comprises this Register Sheet Only.**	PM / PQM			
I1	I1-0	-	I1-58	Loading / Analysis	LDLI	2021 Jan 04	2021 Oct 22	
I2	I2-0	-	I2-131	Girders	LDLI	2021 Mar 05	2021 Nov 21	
I3	I3-0	-	I3-59	Diaphragms	LDLI	2021 Nov 21	2021 Nov 30	
I4	I4-0	-	I4-52	Transverse Box Girder Design	LDLI	2021 Mar 05	2021 Oct 27	
I5	I5-0	-	I5-57	Existing and Proposed Conditon Comparison	LDLI	2021 Nov 25	2021 Nov 25	
I6	I6-0	-	I6-56	Seismic Analysis + Pier 1 (West Abutment) Design	RSGR	2022 Nov 25	2021 Dec 05	
I7	I7-0	-	I7-49	Existing Pier 2 - Design Checks	TOTG	2023 Nov 25	2021 Dec 05	
I8	I8-0	-	I8-55	Existing Pier 3 (East Abutment) - Design Checks	RSGR	2024 Nov 25	2021 Dec 05	
I9	I9-0	-	I9-1	Miscellaneous	FCCA	2025 Nov 25	2021 Dec 05	
I10	I10-0	-	I10-35	Retaining Wall 03.66L	CEKH	2021 Nov 24	2021 Nov 28	
I11	I11-0	-	I11-1	Tendon Geometry	MWBM	2021 Nov 17	2021 Nov 30	
I12	I12-0	-	I12-30	Shear Key Checks for Non-Seismic Loading	LDLI	2021 Nov 10	2021 Nov 26	

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Book: I1 CALC BOOK COMPLETION DATE: **2021 Oct 22**

Title: Loading / Analysis

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I1-0	-	I1-0	Calc Register (this sheet)	N/A	N/A	
2	I1-1	-	I1-1	1 General	LDLI	CEKH	
3	I1-1	-	I1-1	1.1 Purpose	LDLI	CEKH	
4	I1-1	-	I1-1	1.2 Bridge Description	LDLI	CEKH	
5	I1-1	-	I1-1	1.3 Design Specificant Adopted	LDLI	CEKH	
6	I1-2	-	I1-2	2 Model Geometry	LDLI	CEKH	
7	I1-2	-	I1-2	2.1 Assumption	LDLI	CEKH	
8	I1-2	-	I1-2	2.2 Bridge Stations	LDLI	CEKH	
9	I1-2	-	I1-3	3 Material Properties	LDLI	CEKH	
10	I1-3	-	I1-8	4 Structural Components	LDLI	CEKH	
11	I1-3	-	I1-5	4.1 Box Girder	LDLI	CEKH	
12	I1-3	-	I1-6	4.2 East Abutment	LDLI	CEKH	
13	I1-3	-	I1-6	4.3 Intermediate Pier	LDLI	CEKH	
14	I1-3	-	I1-6	4.4 New West Abutment	LDLI	CEKH	
15	I1-6	-	I1-7	4.5 Tendon	LDLI	CEKH	
16	I1-7	-	I1-8	4.6 Diaphragms	LDLI	CEKH	
17	I1-8	-	I1-11	5 Boundary Conditions	LDLI	CEKH	
18	I1-8	-	I1-8	5.1 Bearing Fixity	LDLI	CEKH	
19	I1-9	-	I1-11	5.2 Footing Fixity	LDLI	CEKH	
20	I1-11	-	I1-11	5.3 Pile Point of Fixity at East Abutment (Pier 3)	LDLI	CEKH	
21	I1-11	-	I1-11	5.4 Foundation Wall Point of Fixity at Existing West Abutment (Existing Pier 1)	LDLI	CEKH	
22	I1-11	-	I1-11	6 Model Loading	LDLI	CEKH	
23	I1-11	-	I1-11	6.1 Self-weight of Non-structural Components	LDLI	CEKH	
24	I1-11	-	I1-19	6.2 Live Load, Pedestrian Load, Impact Load, Centrifugal Force, and Braking Force	LDLI	CEKH	
25	I1-19	-	I1-20	6.3 Tendon Stress	LDLI	CEKH	
26	I1-20	-	I1-22	6.4 Temperature Effects	LDLI	CEKH	
27	I1-22	-	I1-24	6.5 Creep and Shrinkage	LDLI	CEKH	
28	I1-24	-	I1-26	6.6 Wind Load	LDLI	CEKH	
29	I1-26	-	I1-27	6.7 Earth Pressure	LDLI	CEKH	
30	I1-27	-	I1-28	6.8 Settlement	LDLI	CEKH	

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Book: I1 CALC BOOK COMPLETION DATE: **2021 Oct 22**

Title: Loading / Analysis

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
31	I1-28	-	I1-28	6.9 Collison Forces	LDLI	CEKH	
32	I1-28	-	I1-29	6.10 Staged Construction	LDLI	CEKH	
33	I1-29	-	I1-30	7 Finalized Model Screenshots	LDLI	CEKH	
34	I1-30	-	I1-43	8 Finalized Model Screenshots	LDLI	CEKH	
35	I1-31	-	I1-34	8.1 Strength Limit States Envelopes	LDLI	CEKH	
36	I1-34	-	I1-35	8.2 Strength Limit States Envelopes	LDLI	CEKH	
37	I1-36	-	I1-43	8.3 Forces and Displacements at Piers	LDLI	MWBM	
38	I1-44	-	I1-58	9 Appendix	LDLI	CEKH	

1. General

1.1 Purpose

The purpose of this calculation book is to present the structural analysis carried out for the BR 17.7 in WSDOT I-405 R2B. Items included in this calculation package:

- model geometry
- material properties
- structural components
- boundary conditions
- model loading
- finalized model screenshots
- analysis results

1.2 Bridge Description

Bridge 405/17.7 carries Renton Ave S/S 3rd St over I-405. It is a two-span reinforced concrete box girder bridge built in two phases, one phase for each span. East span was constructed first. It is flared on the south side at the beginning (east abutment). West span was constructed approximately 2 years after the east span was built. It has a curvature at the end (existing west abutment). This project will lengthen the existing superstructure in the west span for 12.5 ft using post-tension tendons to increase the capacity.

1.3 Design Specifications Adopted

- I-405 17.7 BRIDGE DESIGN CRITERIA_2020
- AASHTO LRFD Bridge Design Specifications, 8th Edition, with 2018 Errata
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd edition with 2015 interim revisions
- AASHTO Manual for Bridge Evaluation, 2nd Edition with interim revisions
- WSDOT Bridge Design Manual LRFD M 23-50 (BDM), June 2018 issued in RFP Appendix D

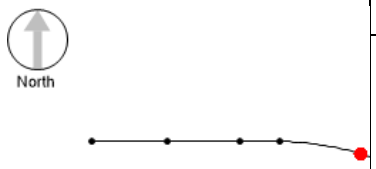
2. Model Geometry

2.1 Assumptions

- The bridge geometry is calculated assuming the existing west span is lengthening for 12.5 ft
- The superelevations in the bridge cross-sections and the slopes along the bridge roadway are neglected in modelling.
- The existing east abutment and intermediate pier are assumed to be perpendicular to the centerline of the bridge deck. The existing west abutment and new west abutment are assumed to be parallel to the existing east abutment and intermediate pier.

2.2 Bridge Stations

- Stations for bridge horizontal layout are calculated based on Figures A-1 and A-2 shown in Appendix A.
- Parameters used for the model layout are shown in the table below:

Layout Line Plan View	Layout Line Segment Type	Station Description	Station (ft)	Radius (ft)	Bearing PI to EC
	Initial Station and Bearing	CL of East Abut	0	-	N900000E
		End Point of East Span Flare	39.13	-	N900000E
		CL of Pier	76.63	-	N900000E
	Straight at Previous Bearing to Station	End Point of the Straight Centerline	97.69	-	N900000E
	Curve Right to New Bearing to Station	CL of Existing West Abutment	140.425	138	
		CL of New West Abutment	152.925	138	S670600E

3. Material Properties

- Based on the general notes on the as-built drawings and design criteria. The material properties are:
    - Cast-in-place concrete:

Location	Strength (psi)	Young's Modulus (ksi)	Density (lb/cf)
Superstructure	5000	4903.484	166
Others	4000	4555.378	155

    - The superstructure concrete density is increased from 155 lb/cf to 166 lb/cf to account for the weight of the tendons, ducts, grout, and utilities. As per design criteria, the weight of concrete is increased to 165 lb/cf to account for the weight

of tendons, ducts, and grout. Then the weight of the utilities is approximately 61lb/ft, which is approximately 0.5% of the self-weight of the superstructure, so it's adding 0.775 lb/cf to the concrete density of the superstructure. This is further scaled by 1/1.25*1.5 to account for the difference of load factor between the self-weight of the components and utilities. By adding the weight of the utilities of 0.93 lb/cf to 165 lb/cf, the equivalent concrete density including the weight of concrete, tendons, ducts, grouts, and utilities is 166 lb/cf.

- Concrete Young's modulus are calculated based on Eq. 5.4.2.4-1 in AASHTO. The superstructure concrete density is taken as 155 lb/cf to calculate the modulus.

$$E_c = 120{,}000 K_1 w_c^{2.0} f'^{0.33}_c \qquad (5.4.2.4\text{-}1)$$

where:

K_1 = correction factor for source of aggregate to be taken as 1.0 unless determined by physical test, and as approved by the owner

w_c = unit weight of concrete (kcf); refer to Table 3.5.1-1 or Article C5.4.2.4

f'_c = compressive strength of concrete for use in design (ksi)

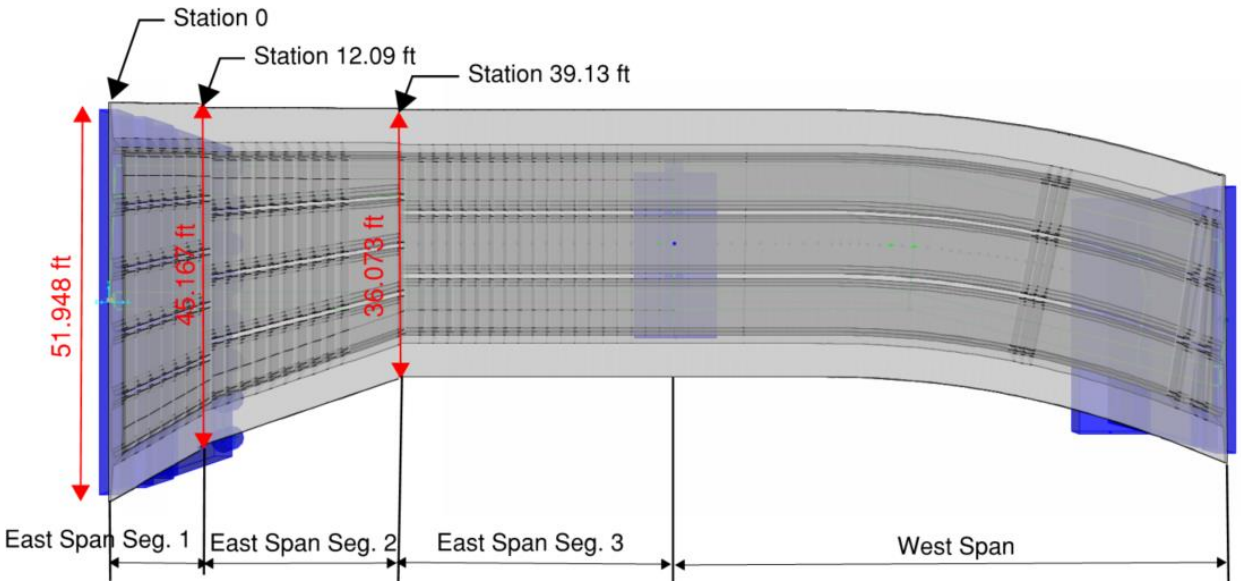
For this bridge, K_1=1.0

- Prestressing strand: 0.6” (A = 0.217 in^2), AASHTO M 203, Low relaxation, 7-wire strand, grade 270 ksi

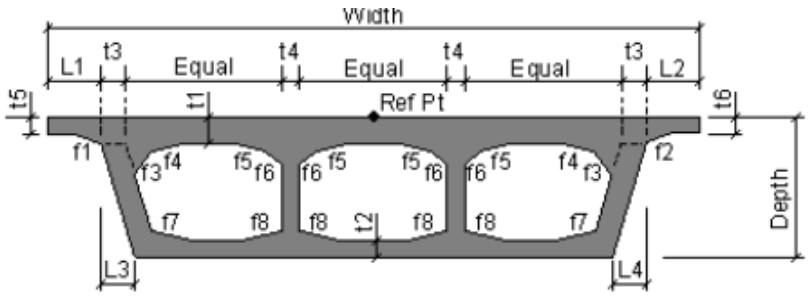
4. Structural Components

4.1 Box Girder

- The bridge is divided into four segments to define the cross-section dimensions for the box girder:
  - The flare region nearby the east abutment is simplified by breaking the flare region into East Span Segment 1 and East Span Segment 2. Segment 1 is between Stations 0 and 12.09 ft and Segment 2 is between Stations 12.09 and 39.13 ft. Within each segment, the total deck width varies linearly.
  - East Span Segment 3 and West Span have consistent deck width



- The dimensions of each segment were obtained from Figures A-2, A-3, A-4, and A-5 as summarized in the table below:



	East Span (Seg. 1)	East Span (Seg. 2)	East Span (Seg. 3)	West Span
Width (ft)	51.948	45.167	36.073	36.073
Depth (ft)	4.250			4.250
L/R exterior girder bottom offset (L3&4) (ft)	1.500			1.500
Top slab thickness, t1 (ft)	0.708			0.708
Bot slab thickness, t2 (ft)	0.542			0.5
Ext. girder thickness, t3 (ft)	0.833			0.833
Int girder thickness, t4 (ft)	0.750			0.750
fillet_horizontal (f1&2) (ft)	4.5			4.5
fillet_horizontal (f3-6) (ft)	0.5			0.5
fillet_horizontal (f7) (ft)	2.5			2.5
fillet_vertical (f1-6) (ft)	0.5			0.5

fillet_vertical (f7) (ft)	0.083	0.083
overhang length (ft)	4.5	4.5
overhang thickness (ft)	0.583	0.583

4.2 East Abutment (Pier 3)

- Cap beam is modeled as a frame element with dimensions summarized below (obtained from Figure A-6):

Thickness (ft)	4.75
Length (ft)	8.5
Width (ft)	51.948

- The secant pile group is lumped into one vertical frame element. The width and length of the element match with those for the secant pile group and the thickness and the modifiers of the moment of inertia were calculated to match the volume and moment of inertia. The dimensions and moment of inertia for the vertical frame element are summarized in the table below:

Width (ft)	51.95
Length (ft)	22.9
Thickness (ft)	3.087
I22 (ft^4)	36061.430
I33 (ft^4)	127.335
I22 Modifier	0.948
I33 Modifier	1.172

- The individual pile below the secant pile group were idealized as piles fixed at the point of fixity (POF) to account for the effect of soil-pile interaction. POF is taken as 57 ft below the ground elevation. The elevation of ground is shown in Figure A-9, which is approximately 1 ft below the top of the superstructure. The dimensions and locations of the piles are summarized in the table below:

Pile#	Diameter (ft)	Distance from Cap Beam Edge (ft)	Length below Ground (ft)	Length below Bot of Secant Pile Wall (ft)
1	4	2.635	57	22.35
2	4	9.302	57	22.35
3	4	15.969	57	22.35
4	4	22.635	57	22.35
5	4	29.302	57	22.35
6	4	35.969	57	22.35
7	4	42.635	57	22.35
8	4	49.302	57	22.35

Note: The length of the individual pile below the bottom of the secant pile wall is calculated as Depth of DOF+ Vertical distance from the top of superstructure to grade line-height of superstructure-height of beam cap-height of secant wall=57ft+1ft-4.25ft-8.5ft-22.9ft=22.35ft.

- The moment of inertia about the bridge longitudinal axis is reduced by 50% for the cap beam and secant wall to account for the cross-section cracking. The moment of inertia about the bridge longitudinal axis and the torsional constant of piles are reduced by 50% to account for cracking.

4.3 Intermediate Pier (Pier 2)

- The dimensions of the intermediate pier were obtained from Figure A-7 as summarized in the table below:

	Pier Wall	Footing
Thickness (ft)	2.5	3.5
Length (ft)	32.1	28
Width (ft)	24	14

- The moment of inertia about the bridge longitudinal axis is reduced by 50% for the pier wall to account for the cross-section cracking.

4.4 New West Abutment (Pier 1)

- The new west abutment is assumed to consist of a foundation wall and a footing.
- The thickness of the wall is assumed to be consistent with that of the pier wall. The width of the wall is assumed to be consistent with the width of the deck at the west end. The bottom of the wall is assumed to be at the same elevation as the bottom of the pier wall. The thickness of the footing is assumed to be consistent with the pier footing. The length and width of the footing were proposed by Mehdi Dastfan. The dimensions of the new west abutment are summarized as:

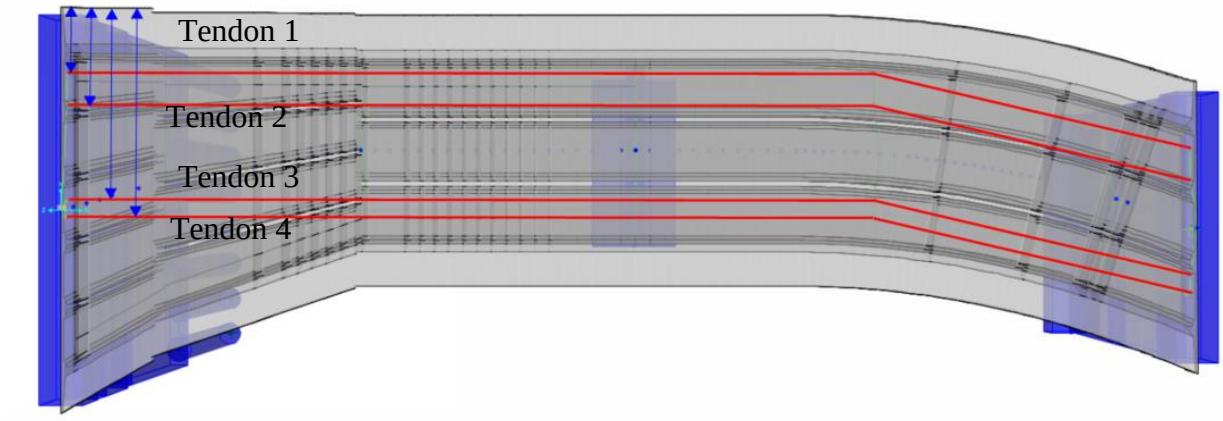
	Pier Wall	Footing
Thickness (ft)	2.5	3.5
Length (ft)	32.1	37
Width (ft)	36.073	8

- The moment of inertia about the bridge longitudinal axis is reduced by 50% for the foundation wall to account for the cross-section cracking.

4.5 Tendon

- 4 tendons with 7-0.6'' AASHTO M203, Low relaxation, 7-wire strand. Grade 270ksi. Each tendon is divided into 5 segments.

4.5.1 Tendon Vertical Layout



- The tendon horizontal layout was defined in the model based on Figure A-8. Tendons are assumed to be 2 ft away from the adjacent box girder surface. The offsets for each tendon from the north edge of the deck are:

Stations (ft)	Tendon 1 (ft)	Tendon 2 (ft)	Tendon 3 (ft)	Tendon 4 (ft)
0	7.33	11.51	24.57	27.27
41.63	7.33	11.51	24.57	27.27
76.63	7.33	11.51	24.57	27.27
105.63	7.33	11.51	24.57	27.27
140.425	14.56	18.74	31.80	34.50
152.925	17.60	21.78	34.84	37.54

4.5.2 Tendon Vertical Layout

- The tendon vertical layout was defined in the model based on Figure A-8.
- The vertical offset distances from the top of the superstructure to the centroid of the tendons are:

Distance from tendon east end (ft)	Offset from deck top (ft)
0	1.63
41.63	3.35
76.63	1.06
105.63	3.31
138.93	3.31
152.925	1.63

4.6 Diaphragms

- The locations of diaphragms were defined based on Figure A-8a and the thickness of the diaphragms were defined based on the corresponding detailed drawings:

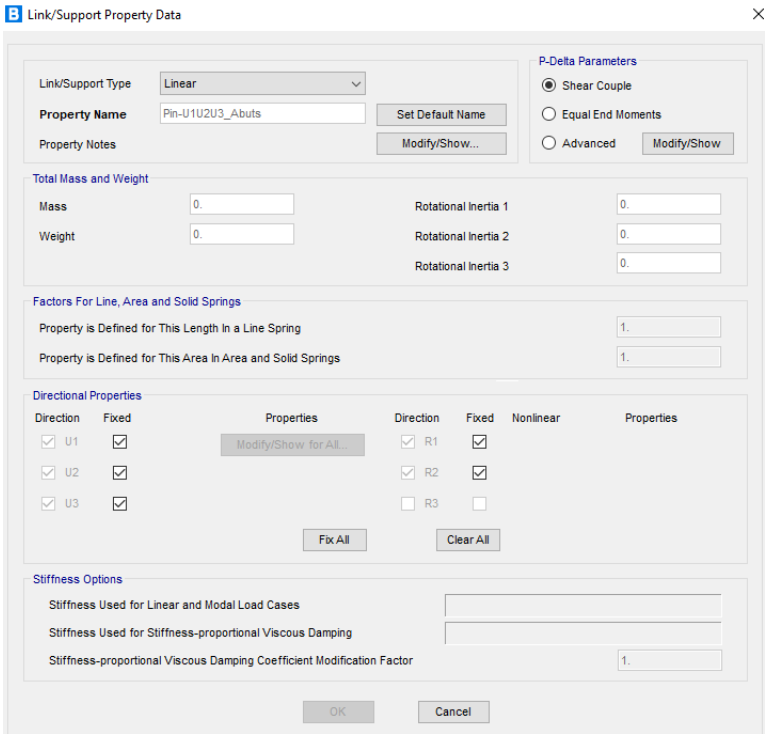
Diaphragms	Location/Span	Thickness (in)	Distance from Pier 3 (ft)
1	East Abutment	32.5	0
2	East span	24	38.1
3	Pier	30	76.6
4	West span	24	105.6
5	Existing West Abutment	56.5	140.4
6	New West Abutment	32.5	152.9

Note that the weight of Diaphragms 2 and 4 are overestimated in CSiBridge as the model automatically generates the diaphragms as a link with weight and mass same as solid concrete with no hollow cells.

5. Boundary Conditions

5.1 Bearing Fixity

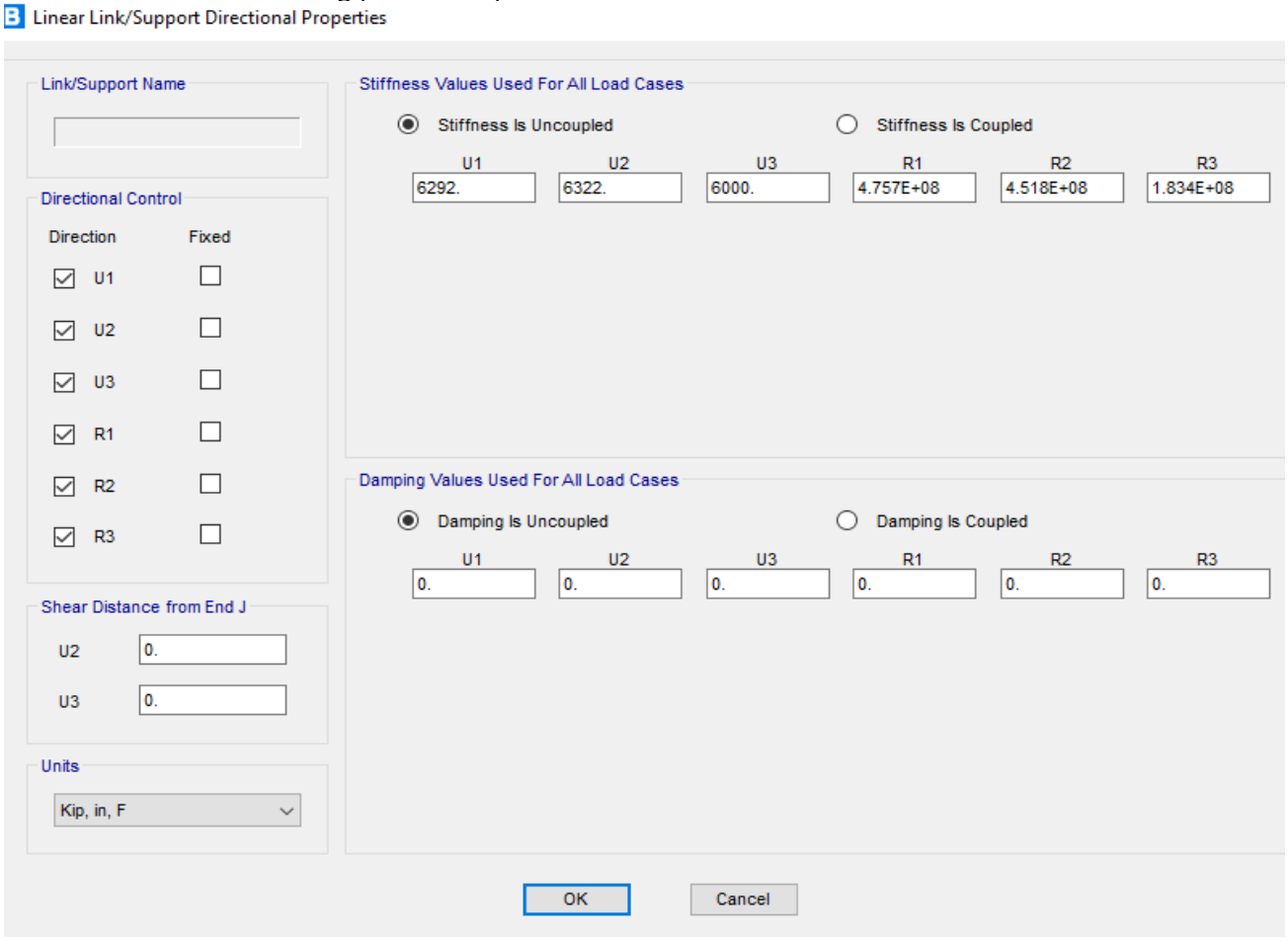
As shown in Figure A-6, the bridge superstructure is pinned at the supports. The bearing is continuous in the transverse direction of the bridge deck. These bearings are modeled a single line of bearing that only allow rotation about the transverse axis. The definition of the bearing fixity is shown in the figure below:



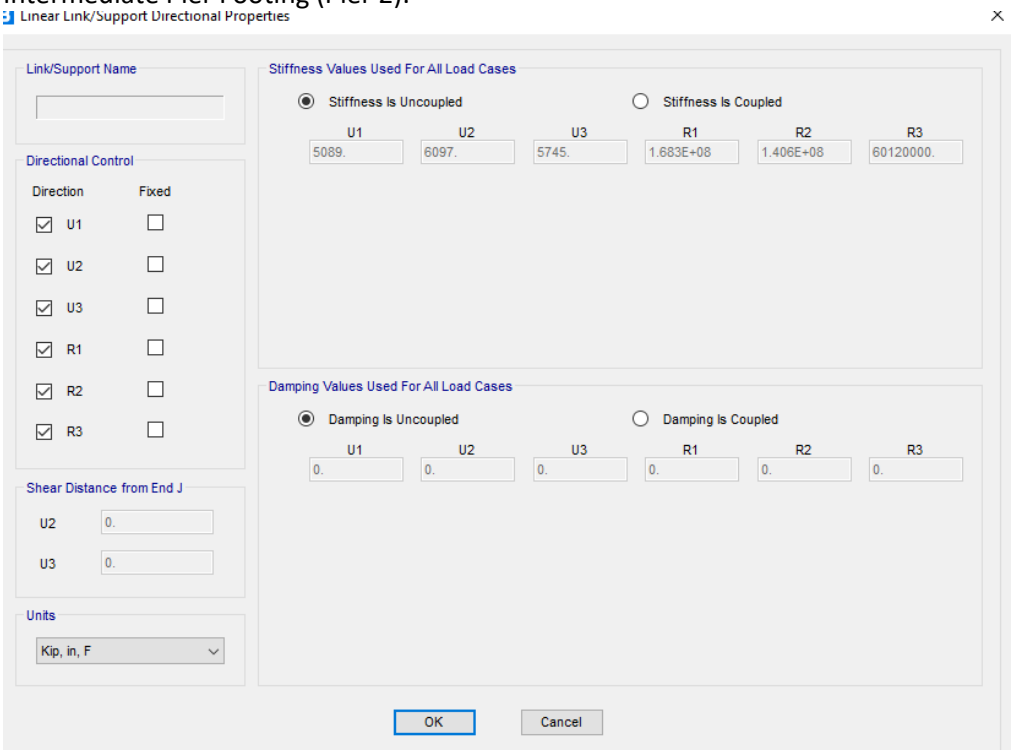
5.2 Footing Fixity

The bottom of the footings at the west abutment and intermediate pier is attached to a one-node link with stiffnesses provided by GeoEngineers.

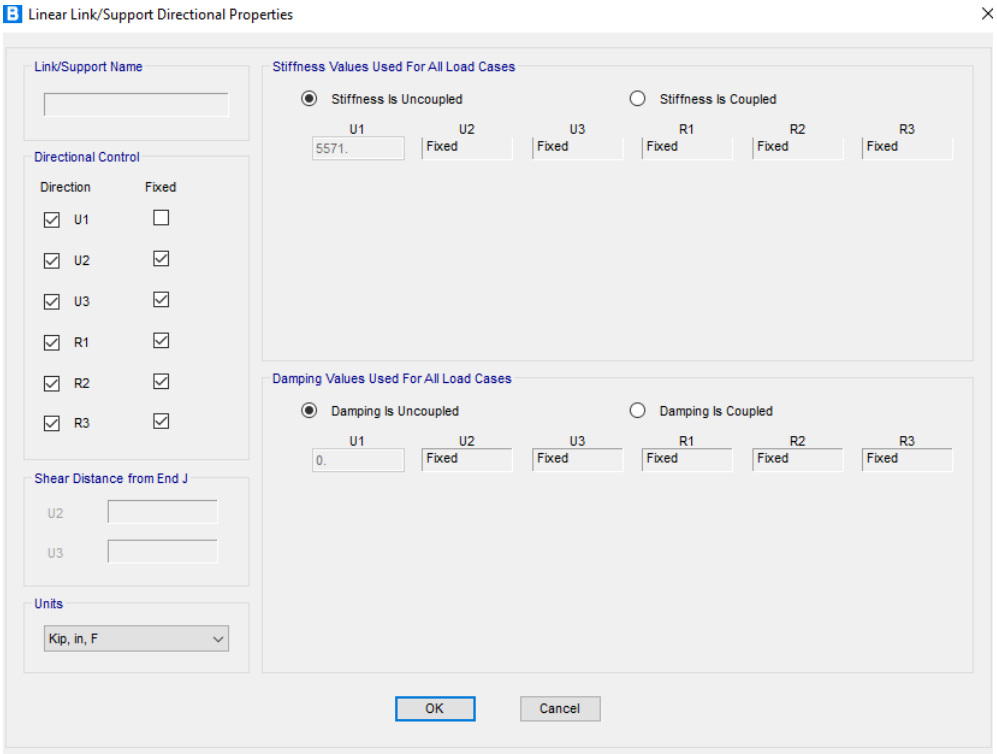
New West Abutment Footing (New Pier 1)"



Intermediate Pier Footing (Pier 2):



East Abutment Individual Pile (Pier 3):



where axis 1 is the vertical axis, axis 2 is the longitudinal axis, and axis 3 is the transverse axis.
The reports provided by GeoEngineers are attached in Appendices C, D, and E.

5.3 Pile Point of Fixity at East Abutment (Pier 3)

The point of fixity is provided by RSGR as 51.25 ft below grade. The calculations are included in Calcbook I6

5.4 Foundation Wall Point of Fixity at Existing West Abutment (Existing Pier 1)

The point of fixity at existing west abutment is assumed to be at 12.5 m (5 times of thickness of foundation wall) below grade.

6. Model Loading

6.1 Self-weight of Non-structural Components

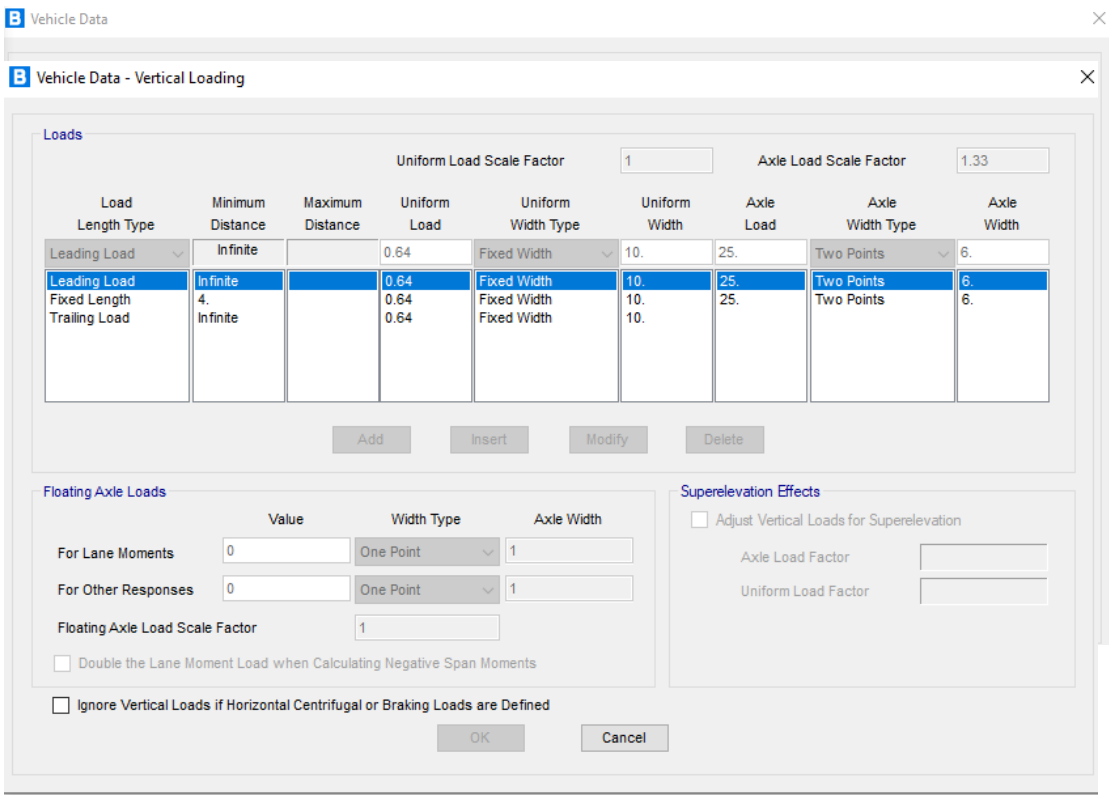
- SW of Non-structural components include:
  - SW of barriers = 0.73 kip/ft
  - SW of fascia wall = (h*t*w)*γ_c=(10 ft)(2.33 ft)(51.948 ft)(0.155 kip/cf)=188 kip
  - SW of sidewalk = t* γ_c = (0.5 ft)(0.155 kip/cf)=0.0775 kip/sf

6.2 Live Load, Pedestrian Load, Impact Load, Centrifugal Force, and Braking Force

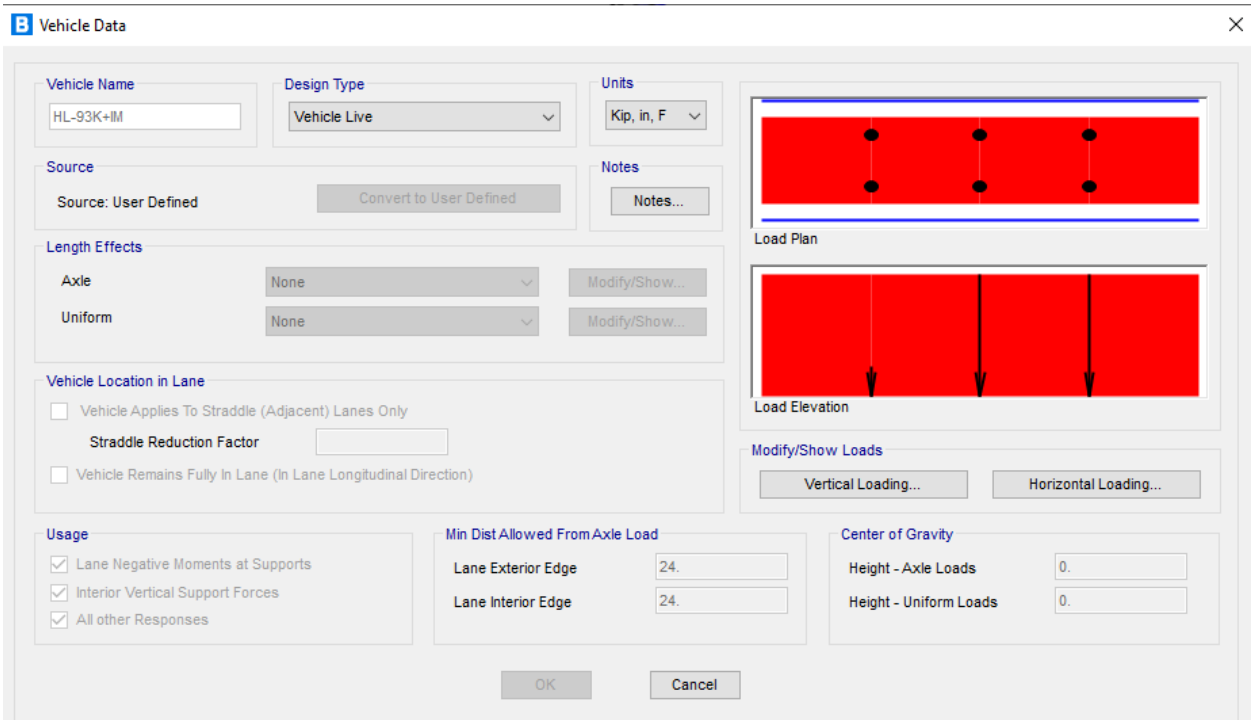
6.2.1 Design Loads

Vehicle Live Load

- As per AASHTO 3.6.1.2, four vehicles were defined for live load analysis for strength check:
  - Tandem+ Lane



- Truck+ Lane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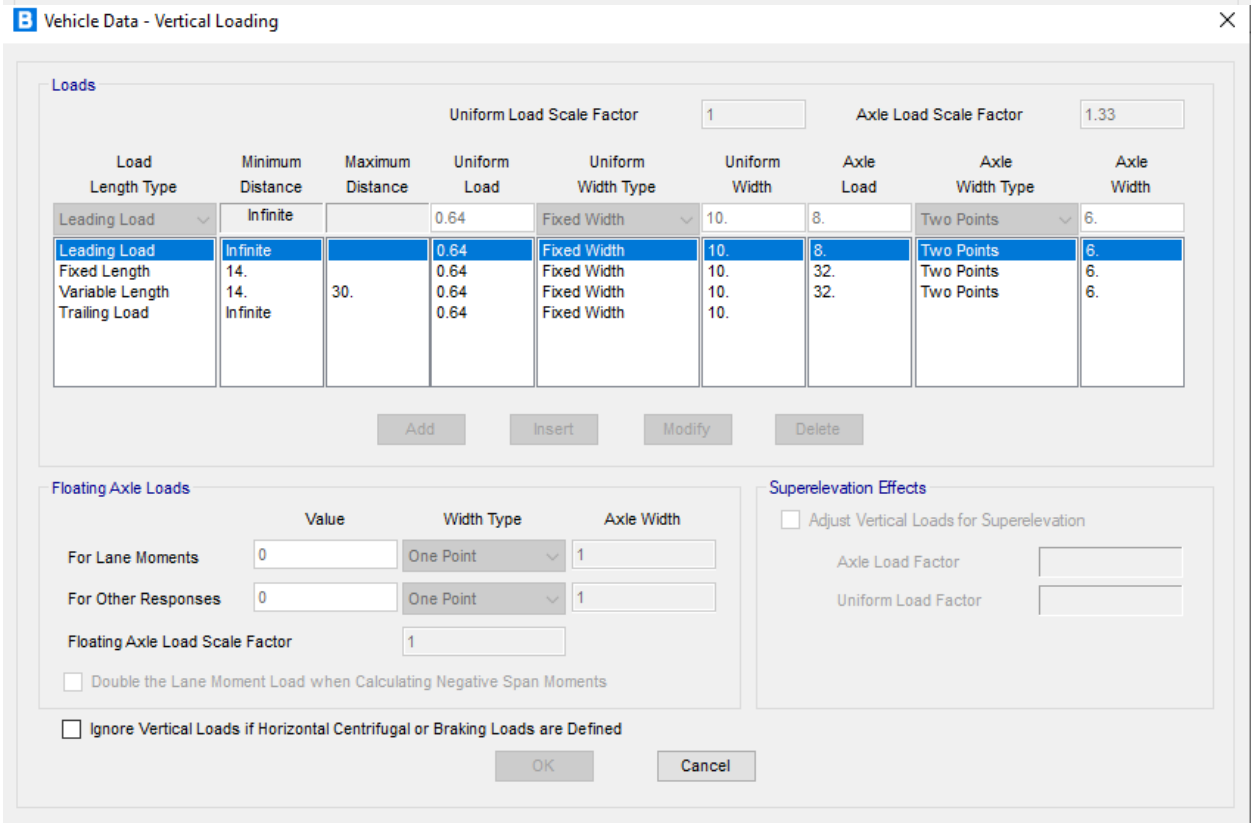
Vehicle Name
HL-93K+IM
Design Type
Vehicle Live
Units
Kip, in, F
Source
Source: User Defined
Convert to User Defined
Notes
Notes...
Load Plan
Load Elevation
Length Effects
Axle
None
Modify/Show...
Uniform
None
Modify/Show...
Vehicle Location in Lane
Vehicle Applies To Straddle (Adjacent) Lanes Only
Straddle Reduction Factor
Vehicle Remains Fully In Lane (In Lane Longitudinal Direction)
Modify/Show Loads
Vertical Loading...
Horizontal Loading...
Usage
Lane Negative Moments at Supports
Interior Vertical Support Forces
All other Responses
Min Dist Allowed From Axle Load
Lane Exterior Edge
24.
Lane Interior Edge
24.
Center of Gravity
Height - Axle Loads
0.
Height - Uniform Loads
0.
OK
Cancel


Vehicle Data - Vertical Loading
Loads
Uniform Load Scale Factor
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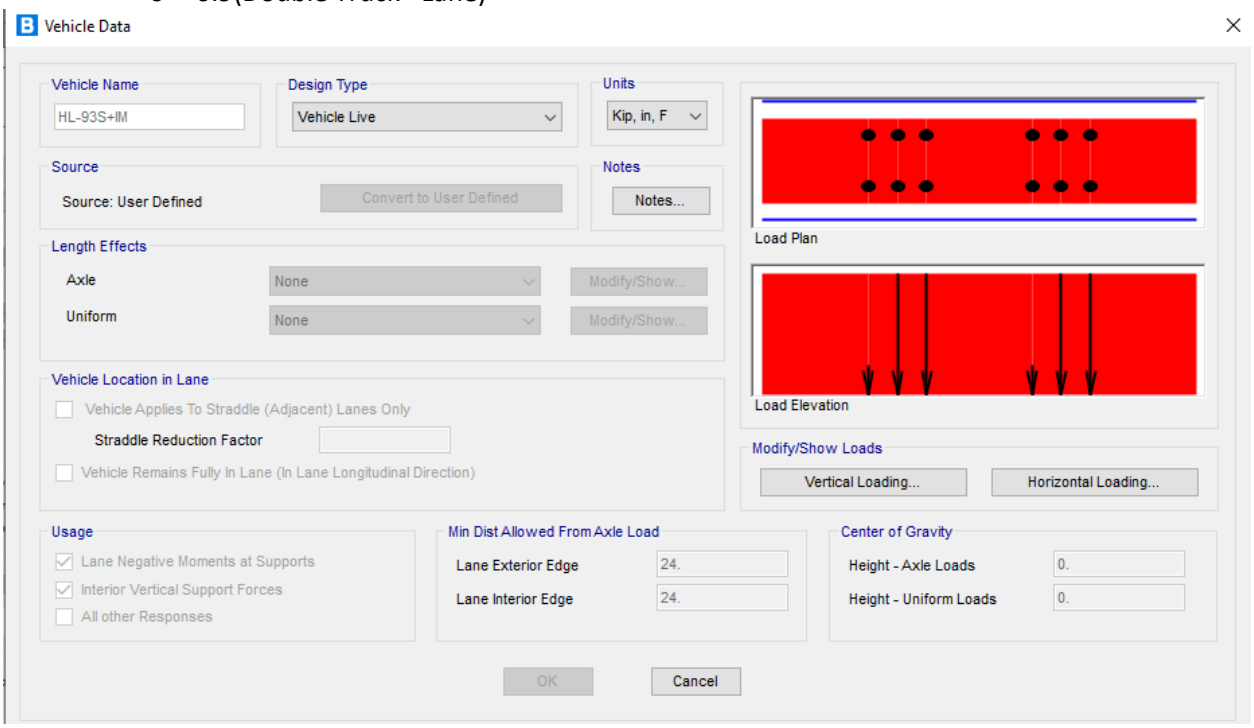
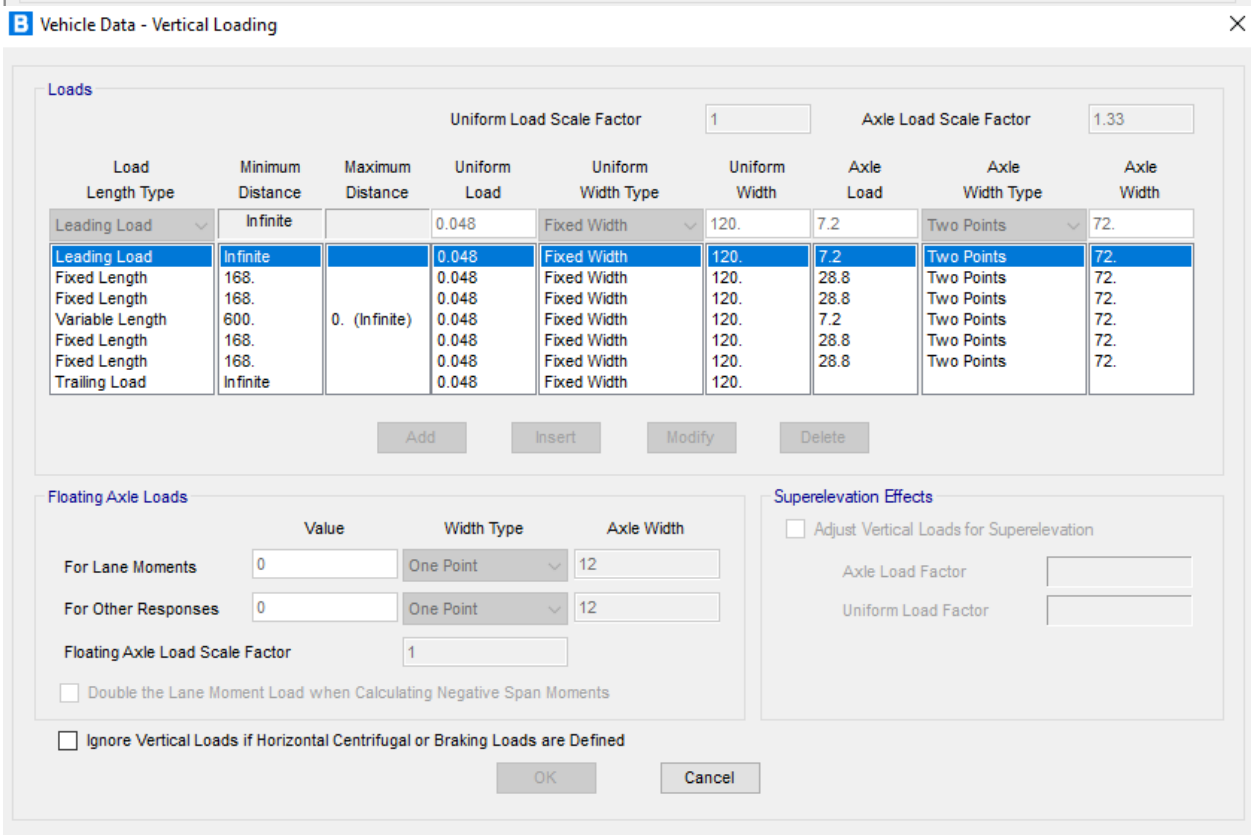
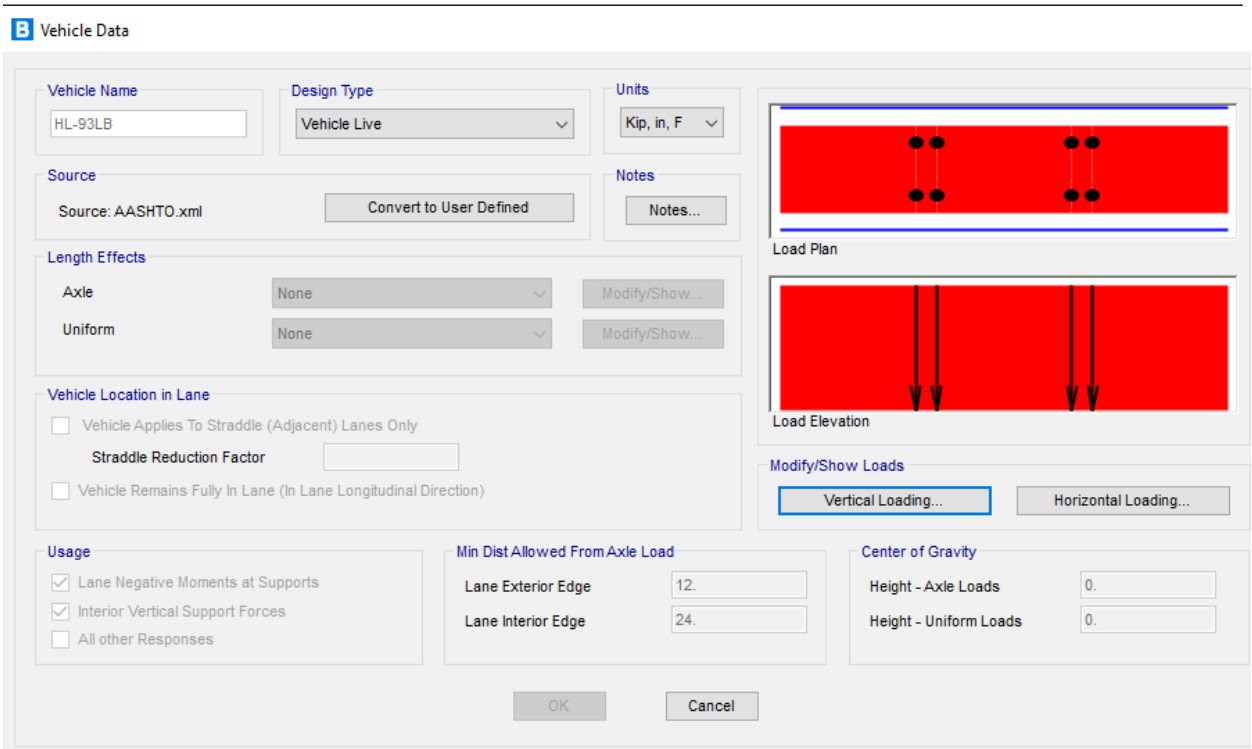
Axle Load Scale Factor
1.33
Load Length Type
Minimum Distance
Maximum Distance
Uniform Load
Uniform Width Type
Uniform Width
Axle Load
Axle Width Type
Axle Width
Leading Load
Infinite
0.64
Fixed Width
10.
8.
Two Points
6.
Leading Load
Infinite
0.64
Fixed Width
10.
8.
Two Points
6.
Fixed Length
14.
0.64
Fixed Width
10.
32.
Two Points
6.
Variable Length
14.
30.
0.64
Fixed Width
10.
32.
Two Points
6.
Trailing Load
Infinite
0.64
Fixed Width
10.
Add
Insert
Modify
Delete
Floating Axle Loads
Value
Width Type
Axle Width
For Lane Moments
0
One Point
1
For Other Responses
0
One Point
1
Floating Axle Load Scale Factor
1
Double the Lane Moment Load when Calculating Negative Span Moments
Superelevation Effects
Adjust Vertical Loads for Superelevation
Axle Load Factor
Uniform Load Factor
Ignore Vertical Loads if Horizontal Centrifugal or Braking Loads are Defined
OK
Cancel

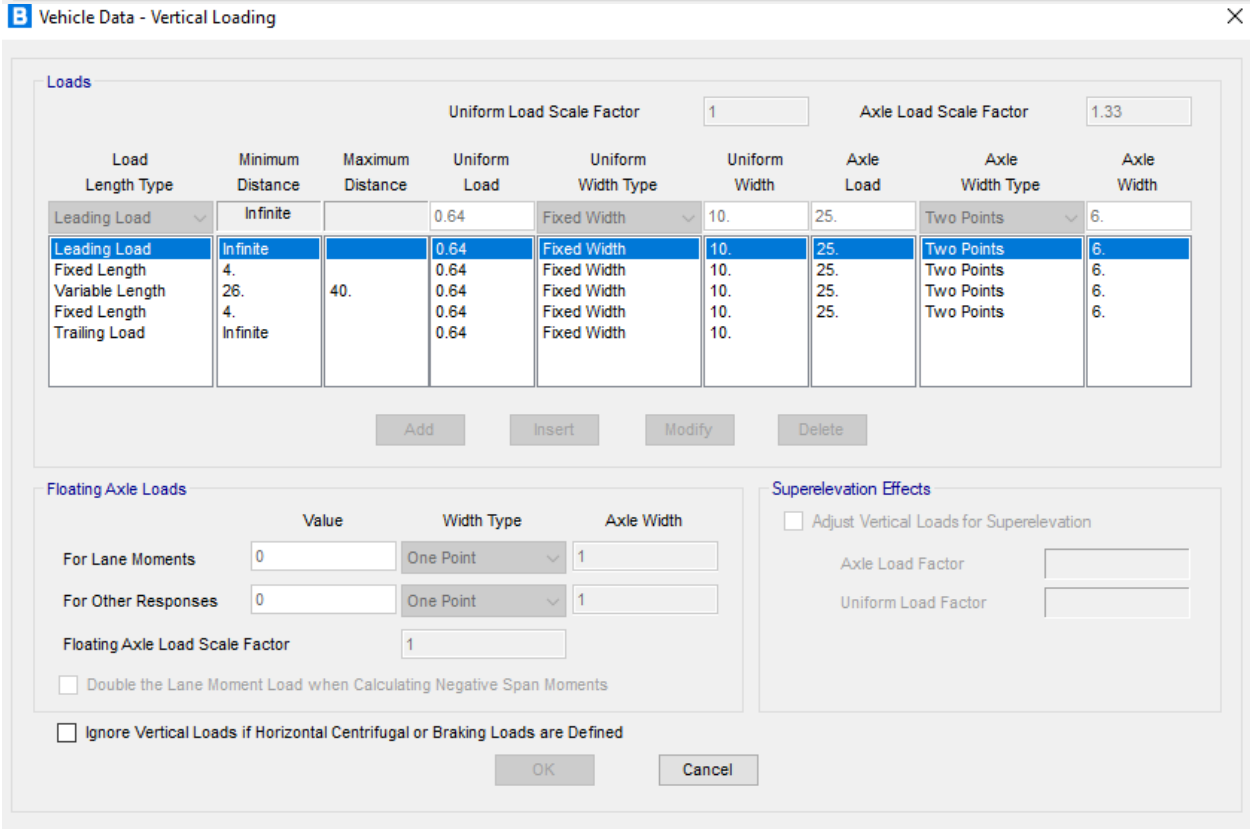
- 0.9(Double Truck+ Lane)





- 0.9(Double Tandem+ Lane)

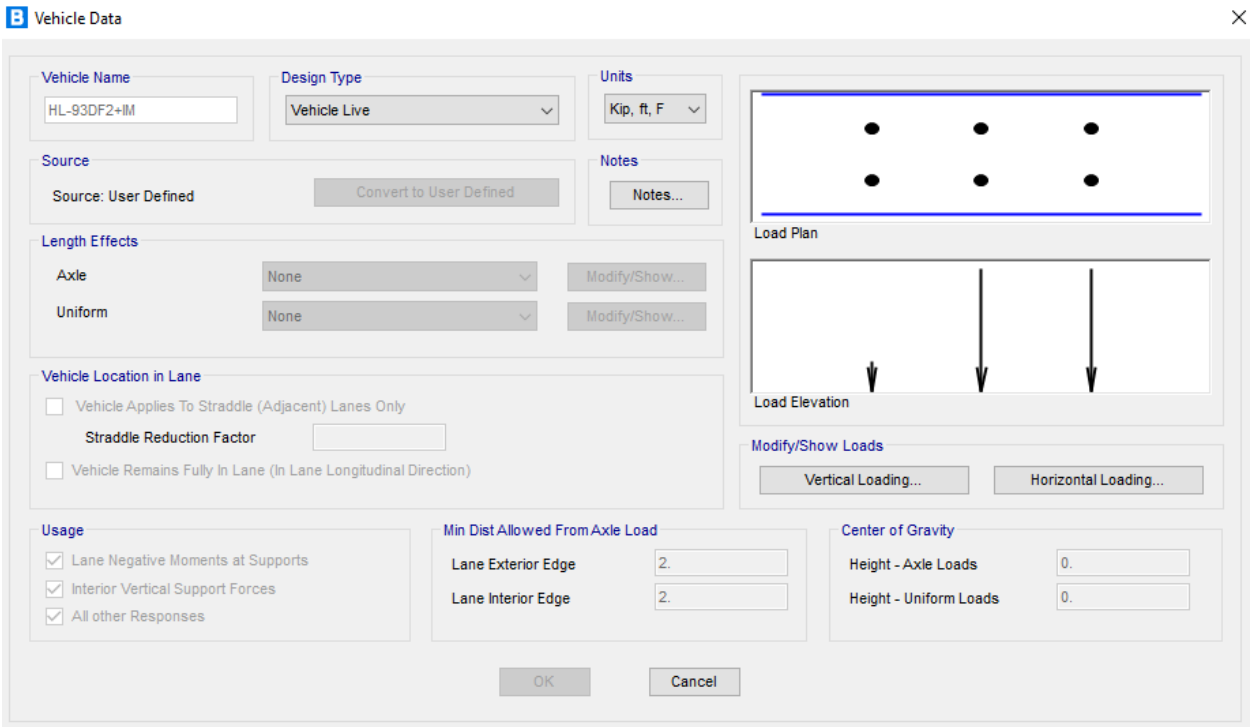


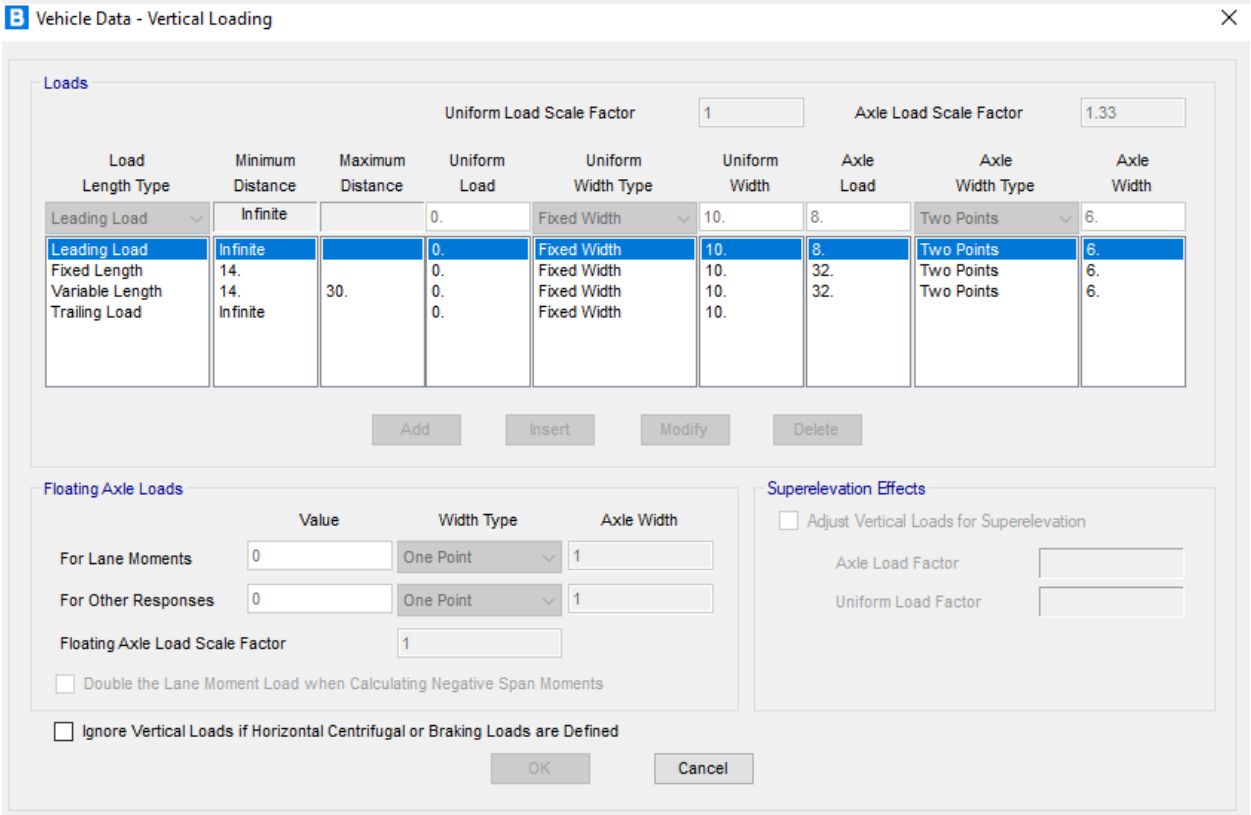


These four vehicles were combined into an envelope for the strength live load analysis.

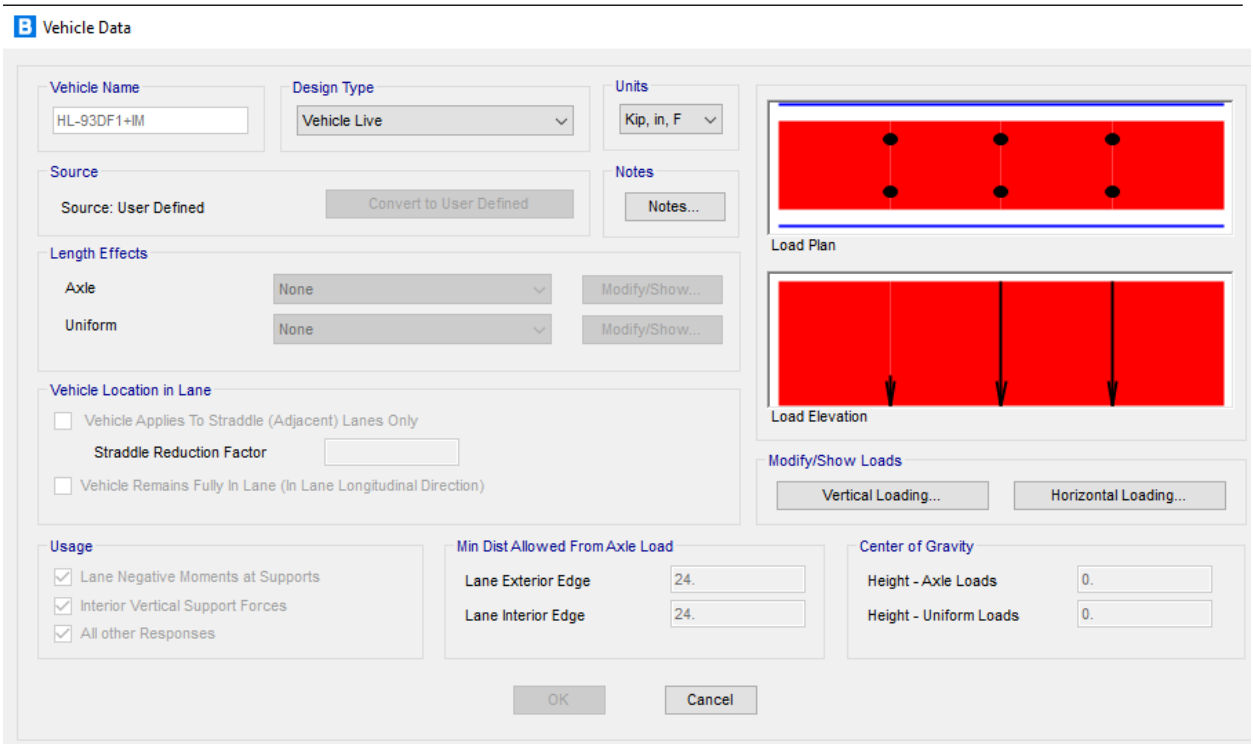
- As per AASHTO 2.5.2.6 and 3.6.1.3.2, two vehicles were defined for live load analysis for deflection check:

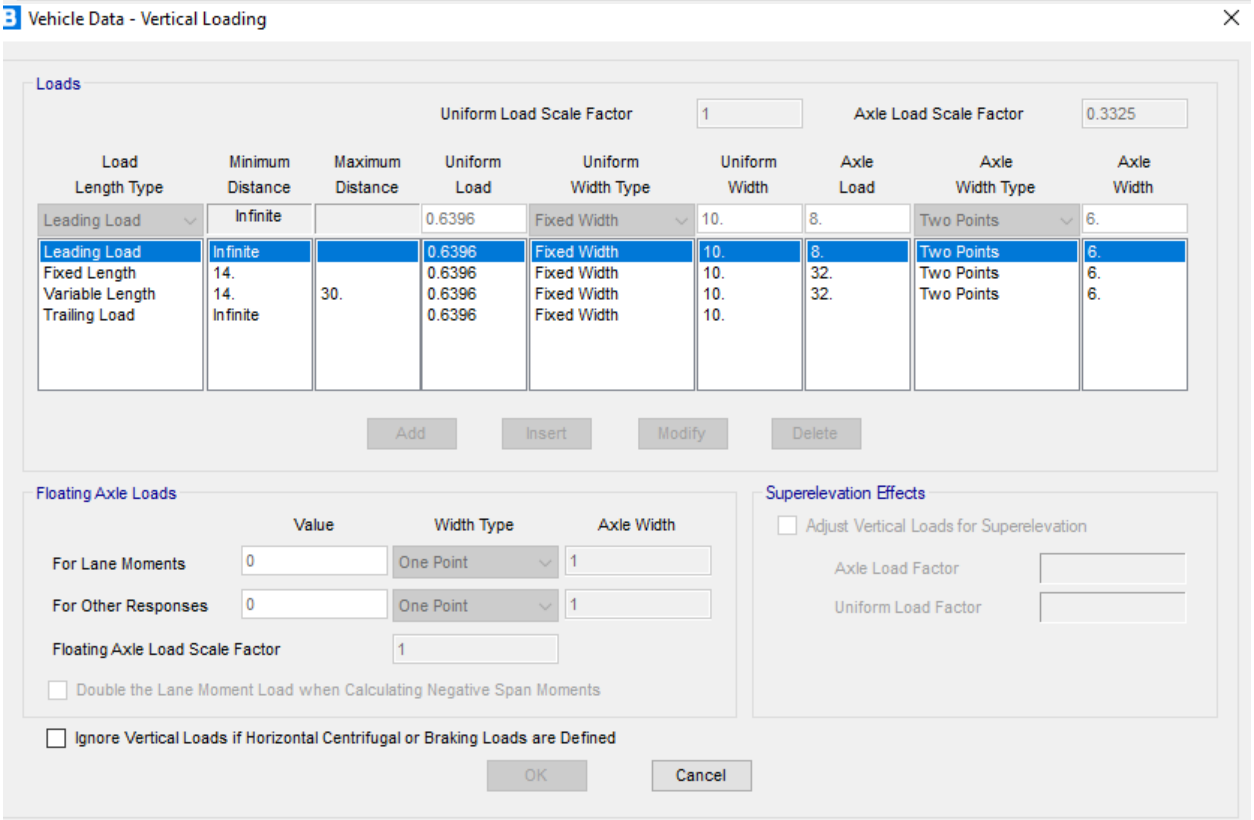
- Truck





- 0.25 Design Truck+ Lane

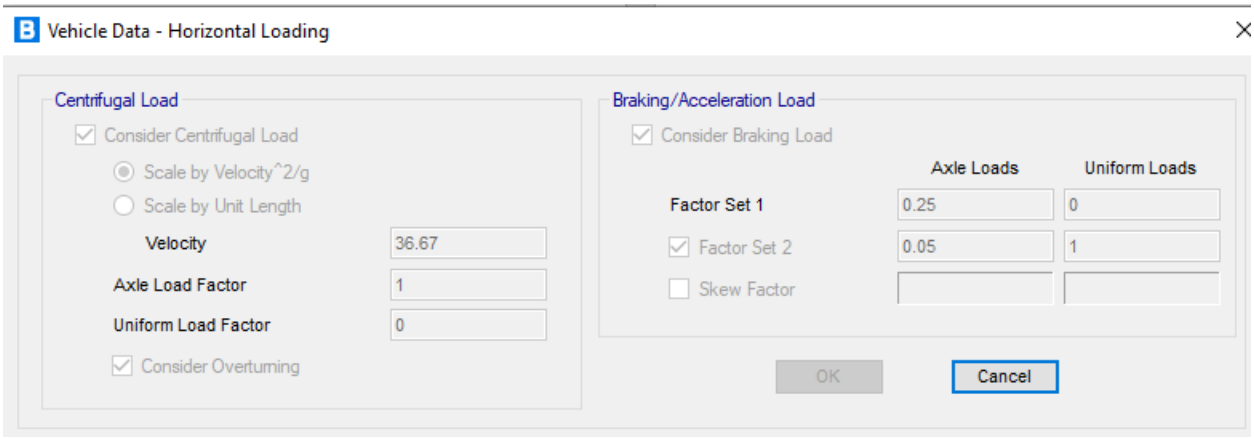




These two vehicles were combined into an envelope for the deformation live load analysis.

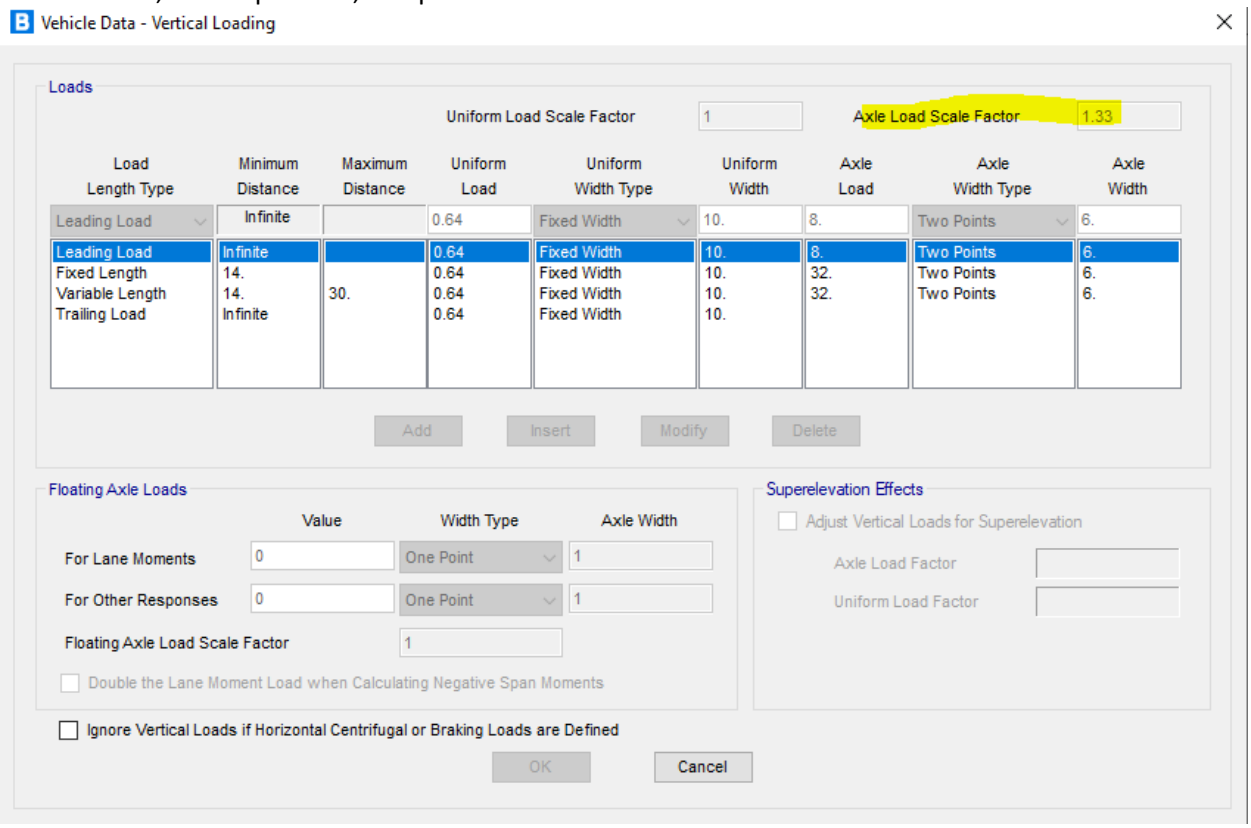
Centrifugal Force and Braking Force

These vehicles also include centrifugal and braking forces. The design speed used to evaluate the centrifugal load was taken as 25 MPH as per design criteria. Braking force is defined as per AASHTO 3.6.4.



Impact Load

As per AASHTO 3.6.2, dynamic load allowance was accounted for the designs for the superstructure, pier wall, and cap beam in the east abutment, but not the footings, foundation walls, secant pile wall, and piles.



6.2.2 Distribution to Superstructure

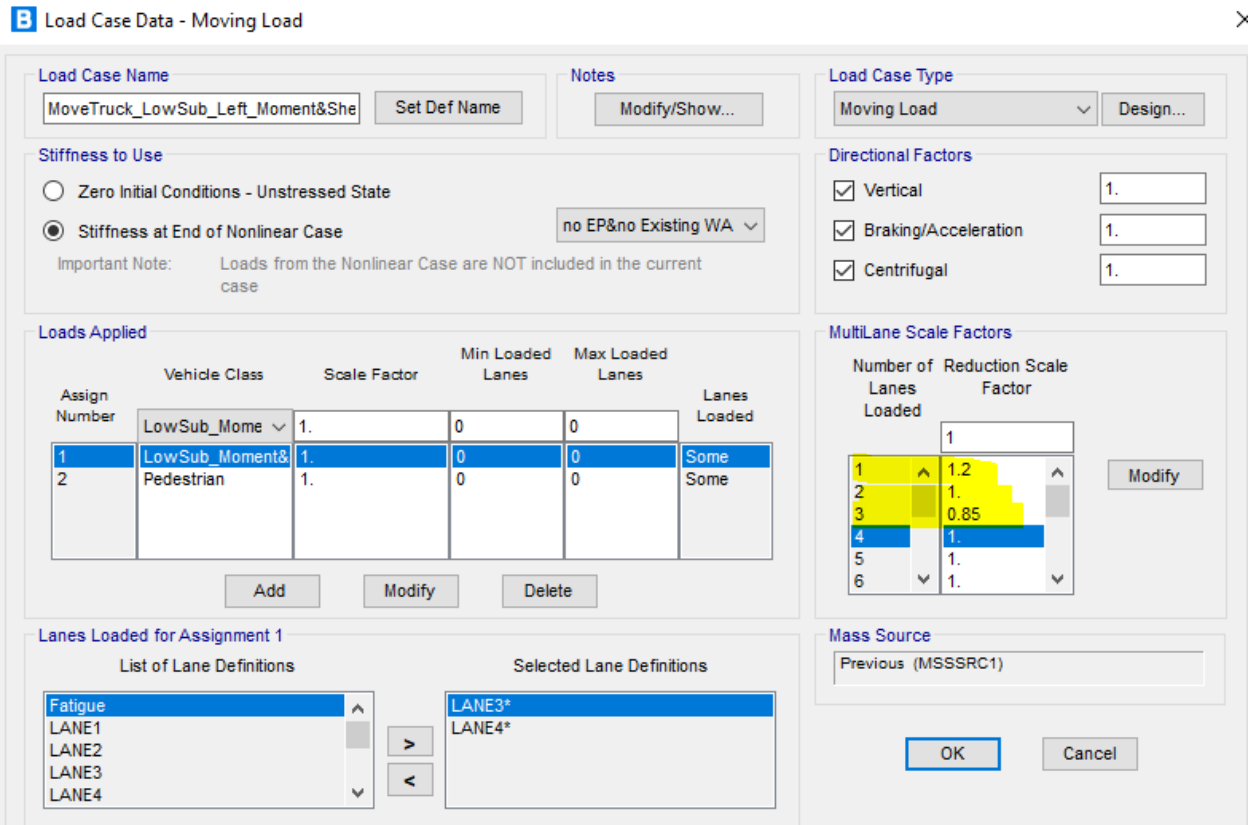
- The load distribution factors were determined as per AASHTO LRFD for interior girders from Table 4.6.2.2.2b-1 for bending moment, and Table 4.6.2.2.3a-1 for shear. As per WSDOT BDM 3.9.1, the live load distribution factors for interior girders were multiplied by the number of webs to obtain

the design live load for the entire superstructure. Correction factors for live load distribution for skewed support were calculated based on AASHTO Tables 4.6.2.2.2e-1 for bending moment and 4.6.2.2.3c-1 for shear and applied to the calculated live load distribution factors. Since the adjusted live load distribution factors are greater than the total number of design lanes, as per WSDOT BDM 3.9.1, the total number of design lanes (2 lanes) was taken as the distribution factors.

- The multiple presence factor (MPF) was applied as per AASHTO 3.6.1.1.2. In this project, the bridge has a sidewalk and two lanes, so it was investigated for:
  - 1 lane of vehicular live load alone with MPF=1.20;
  - the greater of the more significant lanes of vehicular live load and the pedestrian load or two lanes of vehicular live with MPF=1.0;
  - 2 lanes of vehicular live load and for pedestrian loads with MPF=0.85.

Table 3.6.1.1.2-1—Multiple Presence Factors, *m*

Number of Loaded Lanes	Multiple Presence Factors, *m*
1	1.20
2	1.00
3	0.85
>3	0.65



6.2.3 Distribution to Substructure

- As per WSDOT BDM 3.9.1, for substructure design, no distribution factors were used. A detailed live load analysis was conducted. The number of traffic lanes used in the substructure design was determined by dividing the entire roadway slab width by 12, so the number of design lanes is 2. Three situations were considered to generate the maximum stress in the substructure:
  - two lanes placed along the centerline of the bridge
  - two lanes placed next to the sidewalk (right edge of the roadway)
  - two lanes placed next to the left edge of the bridge
- Same as the live load analysis for superstructure design, the multiple presence factor was applied as per AASHTO 3.6.1.1.2.

6.3 Tendon Stress

- Detail calculations of tendon stress are in Calc Book I2- Tendon Stresses
- The tendons are assumed to be jacked at the west end of the bridge. The jacking stress is assumed to be the minimum between 0.7 f_{pu} +anchorage loss or 0.81f_{pu} at the stressing end as per BDM 5.8.3D.
- Each tendon is divided into **5 segments** at the location of slope change (i.e. location of deviator diaphragm). The forces after short-term losses were applied at the ends of each segment manually.
- Short-term tendon losses include:
  - Anchorage slip:
    - as per AASHTO C5.9.3.2.1, the anchorage slip is assumed to equal 0.375 in.
  - Friction loss; and
  - Elastic Shortening.
- To simplify the application of tendon stress, the stress is assumed to be constant between the faces of diaphragms and will experience a sudden change at the face of the diaphragm due to friction.

Tendon stresses from west to east after short-term losses are:

DEVIATOR LOCATION	DISTANCE FROM BACK OF PIER 1 (ft)	SHORT TERM
1	0	178
1	12	179
2	12	179
2	47	191
3	47	191
3	76	197
4	76	192
4	115	192
5	115	184
5	153	184

CEKH: Negligible effect on model and superstructure demands. Model has not been changed. Acceptable.

Note: The tendon stresses were recalculated when the tendon location was changed. The latest stresses are shown in this report, but are not updated in the model because the change is not significant and the model now has lower stresses which is conservative for superstructure box girder global shear and flexure design. For the deviator design, the deviation forces are calculated based on the updated stresses.

Tendon Loads - Tension Force Or Stress

File View Edit Format-Filter-Sort Select Options

Units: As Noted

Filter:

Tendon Loads - Tension Force Or Stress

Tendon Text	LoadPat Text	LoadType Text	Stress Kip/in2	JackFrom Text	Curvature Unitless	Wobble 1/in	LossAnchor	LossEShort Kip/in2	LossCreep Kip/in2	LossShrink Kip/in2	Los
169	PT4	Stress	176	J-End	0	0	0	0	0	0	
174	PT4	Stress	183	J-End	0	0	0	0	0	0	
177	PT4	Stress	194	J-End	0	0	0	0	0	0	
180	PT4	Stress	192	J-End	0	0	0	0	0	0	
184	PT4	Stress	180	J-End	0	0	0	0	0	0	

- Long-term tendon losses include:
  - concrete creep and shrinkage:
    - parameters of creep and shrinkage were defined in the material properties of the superstructure in csibridge. These are described in Section 6.5 of this calc book
  - tendon relaxation:
    - parameters of tendon relaxation were defined in the material properties of tendons in csibridge. The tendon relaxation is estimated based on CEB-FIP 90 model. The tendons are CEB-FIP Class 2 (for low-relaxation tendons).

6.4 Temperature Effects

6.4.1 Temperature Gradient

- The positive and negative temperature gradients were applied in the model as per AASHTO 3.12.3
- The bridge is located in Zone 3 as shown in Figure 3.12.3-1

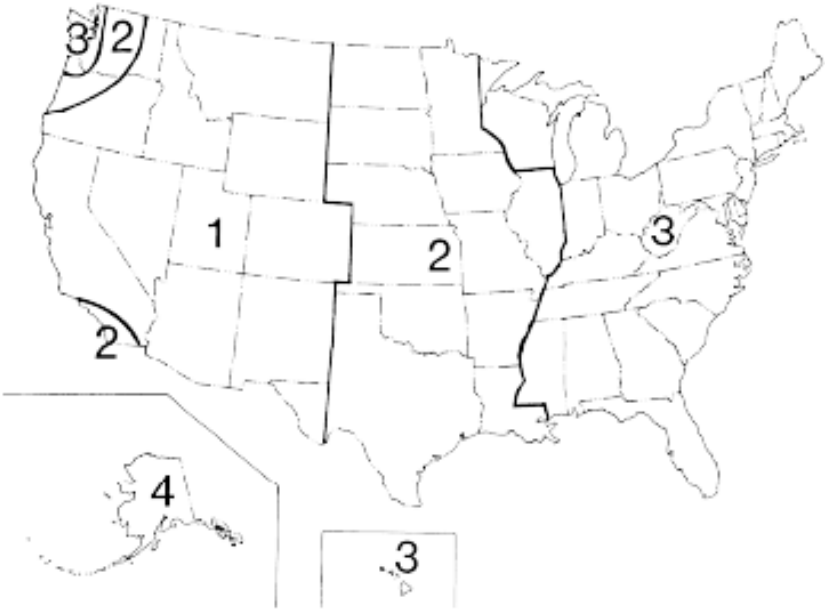


Figure 3.12.3-1—Solar Radiation Zones for the United States

- Therefore, the temperature gradients are:

Table 3.12.3-1—Basis for Temperature Gradients

Zone	T_1 (°F)	T_2 (°F)
1	54	14
2	46	12
3	41	11
4	38	9

For the purpose of this Article, the country shall be subdivided into zones as indicated in Figure 3.12.3-1. Positive temperature values for the zones shall be taken as specified for various deck surface conditions in Table 3.12.3-1. Negative temperature values shall be obtained by multiplying the values specified in Table 3.12.3-1 by −0.30 for plain concrete decks and −0.20 for decks with an asphalt overlay.

The vertical temperature gradient in concrete and steel superstructures with concrete decks may be taken as shown in Figure 3.12.3-2.

Dimension A in Figure 3.12.3-2 shall be taken as:

- For concrete superstructures that are 16.0 in. or more in depth—12.0 in.
- For concrete sections shallower than 16.0 in.—4.0 in. less than the actual depth

- For steel superstructures—12.0 in. and the distance t shall be taken as the depth of the concrete deck

Temperature value T_3 shall be taken as 0.0°F, unless a site-specific study is made to determine an appropriate value, but it shall not exceed 5°F.

Where temperature gradient is considered, internal stresses and structure deformations due to both positive and negative temperature gradients may be determined in accordance with the provisions of Article 4.6.6.

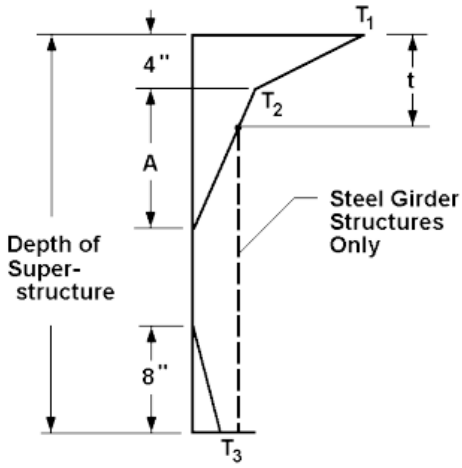
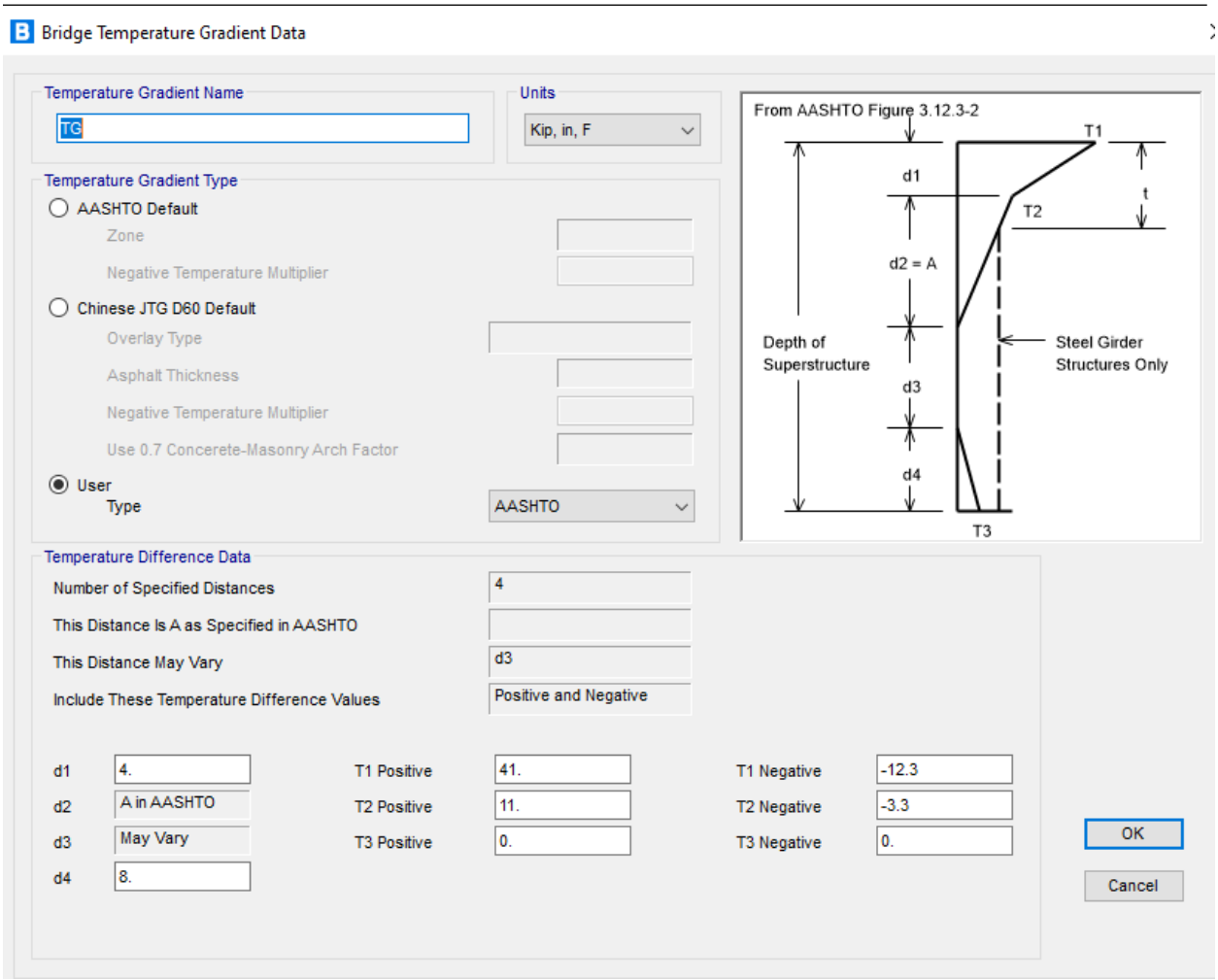


Figure 3.12.3-2—Positive Vertical Temperature Gradient in Concrete and Steel Superstructures



6.4.2 Uniform Temperature change

- As per design criteria, the maximum temperature rise is 36 °F and the maximum temperature drop is -64 °F.

6.5 Creep and Shrinkage

- Creep and shrinkage effects were evaluated based on CEB-FIP 90 model in CSiBridge with parameters:
  - Relative Humidity=75% (as per AASHTO Figure 5.4.2.3.3-1 and WSDOT BDM 5.1.1E)

E. Shrinkage and Creep

Shrinkage and creep shall be calculated in accordance with AASHTO LRFD Section 5.4.2.3. The relative humidity, *H*, may be taken as 75 percent for standard conditions. The maturity of concrete, *t*, may be taken as 2,000 days for standard conditions. The volume-to-surface ratio, *V/S*, is given in Table 5.6.1-1 for standard WSDOT prestressed concrete girders.

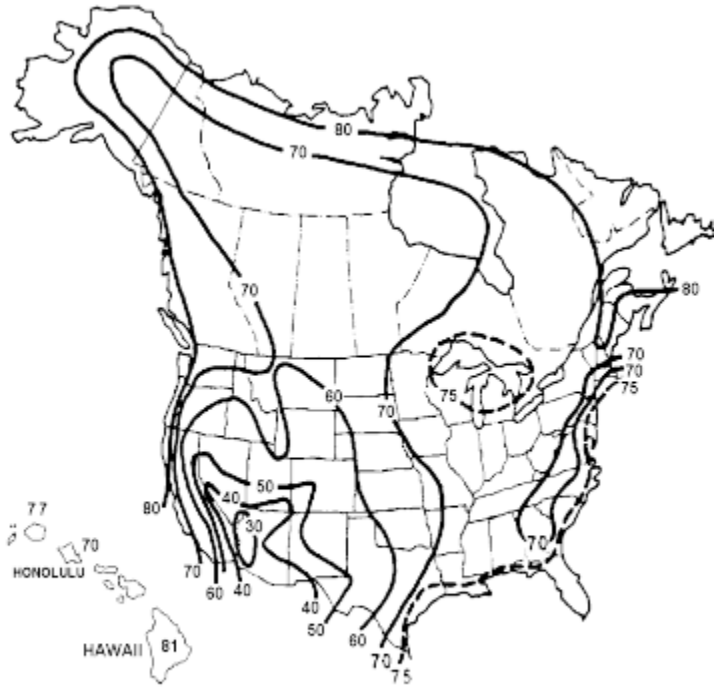


Figure 5.4.2.3.3-1—Average Annual Ambient Relative Humidity, in Percent

  - Shrinkage Coefficient =0.0002 (as per CEB-FIP 90)
  - Shrinkage Start Age = 28 days
- The notional size (i.e. volume-to-surface ratio) was calculated as per AASHTO 5.4.2.3.1, such that for poorly ventilated enclosed cells, only 50% of the interior perimeter was used in calculating the surface area.

> The surface area used in determining the volume-to-surface ratio should include only the area that is exposed to atmospheric drying. For poorly ventilated enclosed cells, only 50 percent of the interior perimeter should be used in calculating the surface area. For precast members with cast-in-place topping, the total precast surface should be used. For pretensioned stemmed members (I-beams, T-beams, and box beams), with an average web thickness of 6.0 to 8.0 in., the value of k_s may be taken as 1.00.

- Therefore, the notional sizes for all three superstructure cross-sections are:

	East Span Seg. 1	East Span Seg. 2	East Span Seg 3 and West Span
Total Area of Superstructure (sf)	79.03	67.31	51.80
Out Perimeter	109.46	96.04	77.21
Cell Perimeter (ft)	97.63	101.83	61.71
50% of perimeter	48.81	50.92	30.85
Total Perimeter	158.27	146.96	108.06
Notional Size	**1.00**	**0.92**	**0.96**

6.6 Wind Load

- Wind Exposure Category: B (as per AASHTO 3.8.1.2)

> The exposure category of the structure shall be determined as follows:
>
> - Wind Exposure Category B: Wind Exposure Category B shall apply where the Ground Surface Roughness Category B, as defined in Article 3.8.1.1.4, prevails in the upwind direction for a distance greater than 1,500 ft for structures with a mean height of less than or equal to 33 ft, and for a distance greater than 2,600 ft or 20 times the height of the structure, whichever is greater, for structures with a mean height greater than 33 ft.

6.6.1 Wind Load to Superstructure

- As per WSDOT BDM 3.11.1, wind pressure to superstructure are:

Limit State	Transverse (ksf)	Longitudinal (ksf)
Strength III	0.029	0.007
Strength V	0.021	0.005
Service I	0.016	0.004

  - Transverse Load:
    - Transverse wind load is applied as a line load located at the top surface of the superstructure
    - The projection height of the transverse wind load is:

      total height = $H_{superstructure}$ +$H_{barrier}$=4.25+2.75=7 ft
    - At the top surface of the superstructure, the eccentricity of the wind load is:

      e=total height/2- $H_{barrier}$=0.75ft
  - Longitudinal Load:

- Longitudinal wind load is applied as a point load located at the midpoint of the deck cross-section at the top of the deck.
- The width of deck is assumed to be consistent along the bridge. Therefore, the projection area of the longitudinal wind load is approximated by multiplying the deck width and height of the superstructure:

  projection area= $H_{superstructure} \times B = 4.25 \times 36.073 = 153.310$ sf
- At the top surface of the superstructure, the eccentricity of the wind load is approximately 2.06 ft

- Therefore, the wind loads on superstructure are:

	Longitudinal (x)		Transverse (y)	
Limit State	Point Load (kip)	Moment (kip.ft)	Line Load (kip/ft)	Moment (kip.ft)
Strength III	1.073	2.213	0.203	0.152
Strength V	0.767	1.581	0.147	0.110
Service I	0.613	1.265	0.112	0.084

6.6.2 Wind Load to Substructure

- Gust Wind Speed (AASHTO TABLE 3.8.1.1.2-1)

Limit State	3-second Gust Wind Speed (mph)
Strength III	110
Strength V	80
Service I	70

- Parameters and wind coefficients assumed for the wind pressure calculation:

Structure Height, z (ft)	29.300	Vertical clearance as labeled on Page 1 of dwg + height of superstructure
Pressure exposure coefficient for Strength III and Service IV, K_{zb}	0.682	AASHTO C3.8.1.2.1
Pressure exposure coefficient for Strength V and Service I, K_z	1.0	AASHTO C3.8.1.2.1
Gust Effect Factor, G_e	1	AASHTO Table 3.8.1.2.1-1
Drag Coefficient, C_{d_sub}	1.6	AASHTO Table 3.8.1.2.1-2
Drag Coefficient, C_{d_sup}	1.3	AASHTO Table 3.8.1.2.1-2

- Wind Pressure

Limit State	Wind Pressure (ksf)
Strength III	0.0338
Strength V	0.0262
Service I	0.0201

- Skew Coefficients

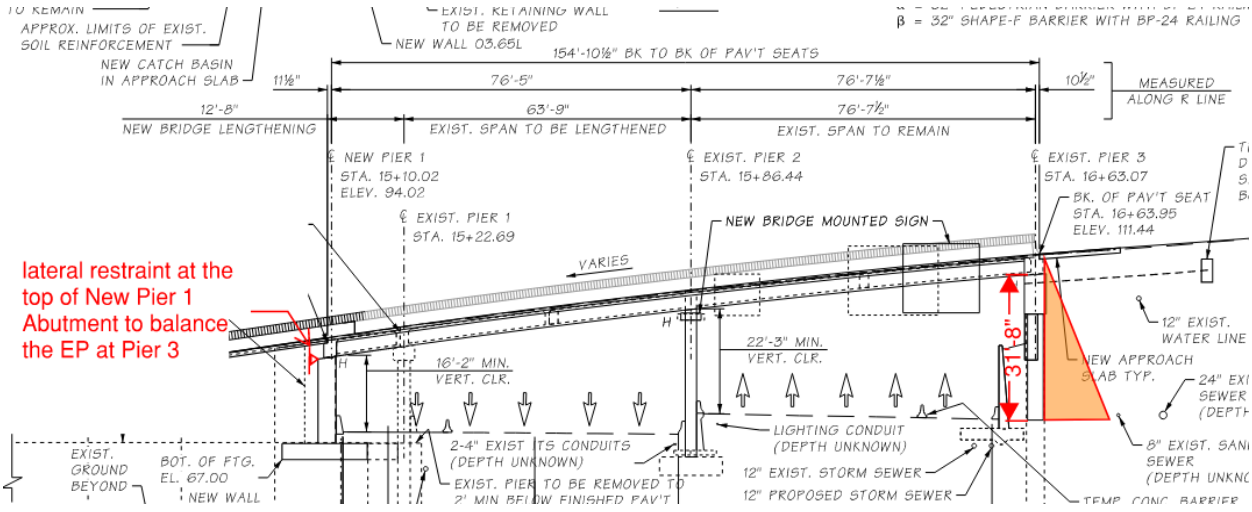
Skew Angel	trans. skew coeff.	long. Skew coeff.
0	1	0
15	0.933	0.16
30	0.867	0.373
45	0.627	0.547
60	0.32	0.667

6.6.3 Wind Load to Live Load

- As per WSDOT BDM 3.11.1, wind pressure on the vehicle are:
  - 0.1 kip/ft in the transverse direction
  - 0.04 kip/ft in the longitudinal direction

6.7 Earth Pressure

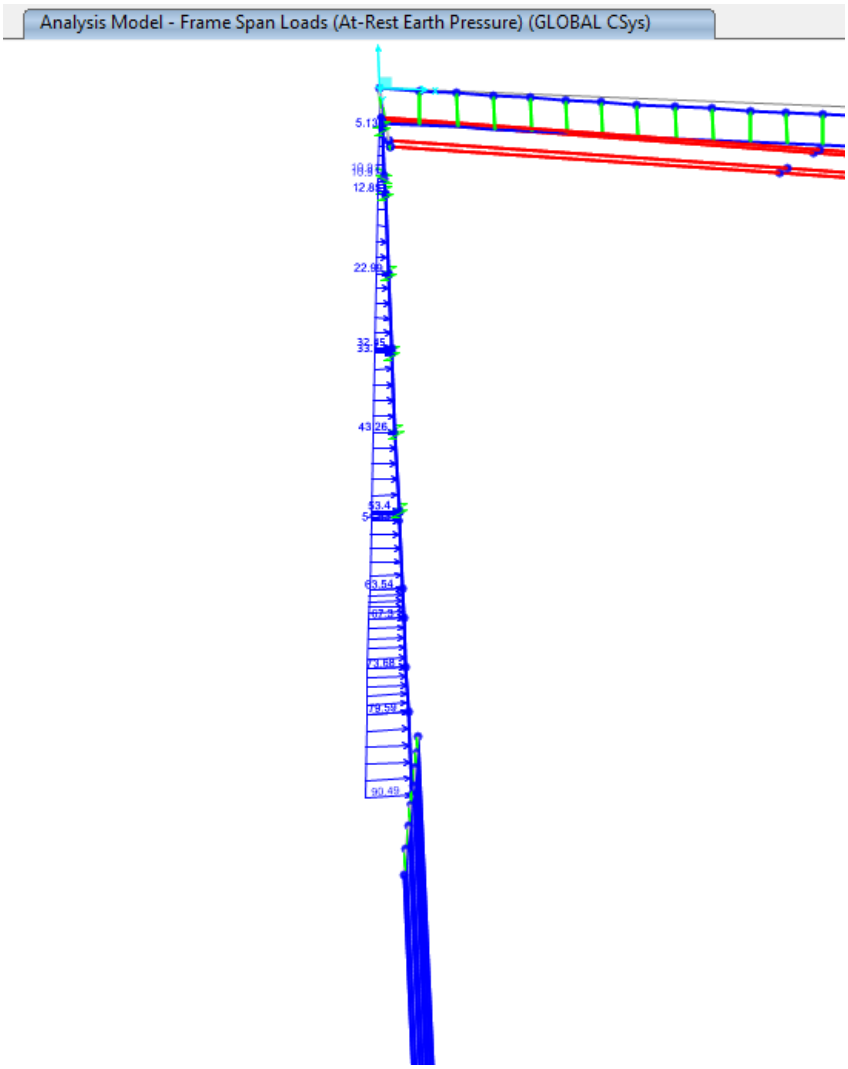
- As discussed with BOHU, as shown in the figure below, at-rest earth pressure is applied along the depth of East Abutment (Pier 3) and a lateral restraint is added at the top of the abutment at the new West Abutment (New Pier 1) to balance the lateral pressure.



Lateral Earth Pressure

K_0	0.38		*at-rest earth pressure coefficient assumed*
Y_s	0.13	kcf	*soil unit weight*
b	51	ft	*width of diaphragm, cap beam, and secant pile wall*

H	35.90	ft	*depth below ground*
p_t	0	kip/ft	*magnitude of distributed load applied at the top of diaphragm*
p_b	90.45	kip/ft	*magnitude of distributed load applied at the bottom*
ΣF	1623.5	kip	*total lateral force applied*



6.8 Settlement

- The settlement of 1.5" was applied at the new West Abutment (new Pier 1) after the construction is completed. This was based on the inputs from GeoEngineers sent on 2021/10/22. The settlement are changed to 0.59" on 2021/12/03 due to the change in pier 1 spring stiffnesses. Since the settlement of 1.5" leads to higher demand of negative bending moment at Pier 2, the analysis and the design are not updated. The settlements from GeoEngineers are attached in Appendix E.

6.9 Collison Forces

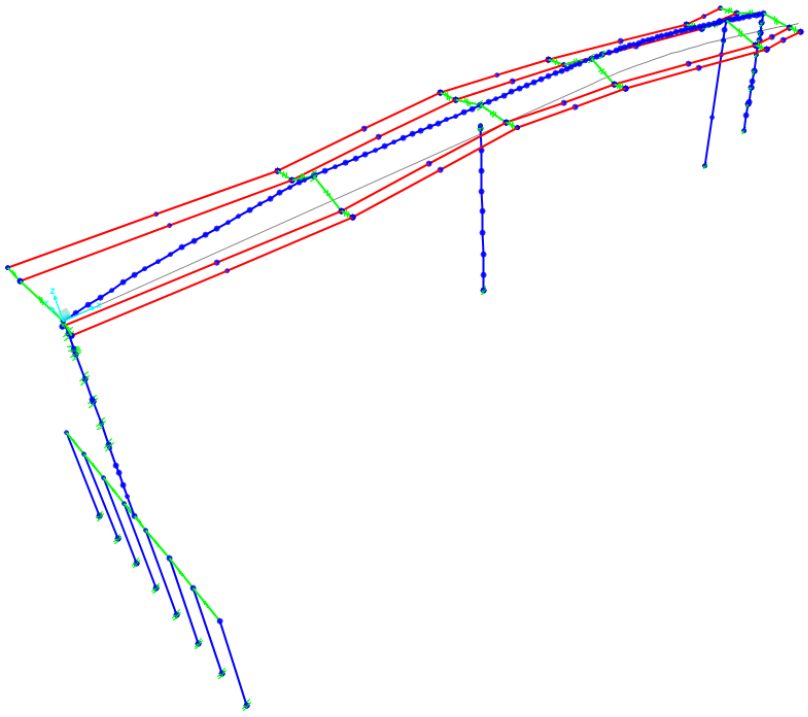
- As per AASHTO 3.6.5, the pier and abutment were subjected to 600 kips static force at a distance of 5 ft above ground. The load is applied in the direction of zero degree with the edge of the pavement to account for the worst-case scenario.

6.10 Staged Construction

- Stage construction load cases were created to define the sequence of construction and to evaluate time-dependent behavior. The stages considered include:

Construction Stage	Description
DL0_Short A_STG	East span constructed but not loaded
DL1_Short A_STG	East span loaded with dead load
DL1_Short B_STG	West span constructed but west span not loaded
DL2_Short B_STG	West span loaded with dead load
DL2_Short B_ Existing	Existing Condition – 28 years after west span is loaded
DL2_Short C_Sub	New Pier 1 (West Abutment) foundation added
DL2_ShortC_STG	New Pier 1 (West Abutment) superstructure added. Tendon not stressed
DL2_ShortC_(DC+PT)	Tendons stressed and self-weight of box girder added
DL3+PT_AbutRemove_Short_STG	Existing west abutment removed
DL3b+PT_Short _	Barrier and sidewalk self-weight are added
DL3b_Long_STG	75 years after the self-weight of barrier and sidewalk are applied
DL0_PT_Long_STG-1	75 years after the self-weight of barrier and sidewalk are applied. The structure is subjected to tendon force only. **The axial forces in tendons are used to calculate effective tendon stress for designs**

7. Finalized Model Screenshots



CEKH: These are direct output results from the analysis model. Spot checked results from some stations. No issues.

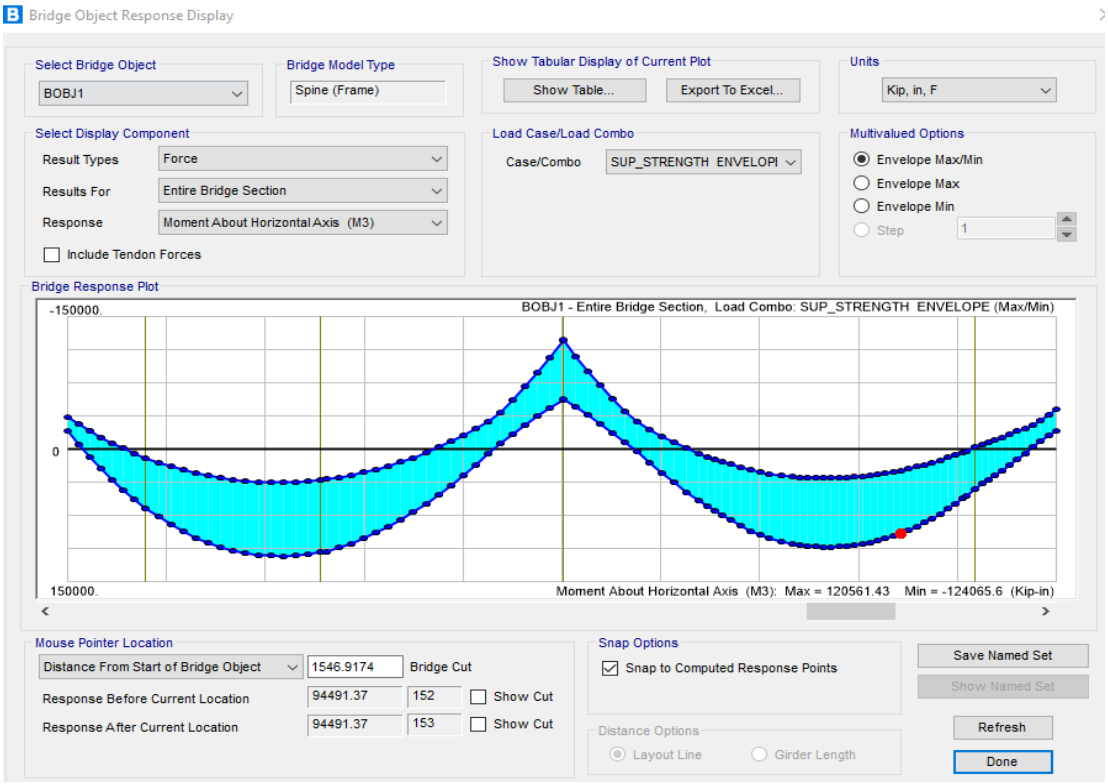
8. Analysis Results (Strength Limit States Envelope)

Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)
0.00	-353	-772	-181	-909	-1996	-3335	-969	-1048	-1863	-3553
1.73	-328	-730	-207	-996	-1300	-2106	-968	-971	-843	-2569
3.45	-304	-689	-232	-1080	-350	-1258	-967	-970	332	-1935
5.18	-280	-648	-258	-1157	532	-467	-966	-969	1429	-1364
6.91	-256	-608	-284	-1228	1345	268	-966	-969	2446	-844
8.64	-232	-568	-310	-1289	2093	947	-965	-968	3383	-373
10.36	-208	-529	-563	-1341	2785	2670	-965	-968	4255	57
12.09	-181	-489	-335	-935	3484	3546	-985	-1065	5166	463
14.02	-153	-450	-372	-985	4093	4294	-985	-1065	5943	846
15.95	-125	-403	-412	-1042	4524	4997	-983	-1064	6645	1183
17.88	-97	-363	-449	-1073	4972	5612	-983	-1063	7253	1474
19.82	-69	-324	-484	-1092	5350	6153	-982	-1062	7776	1721
21.75	-42	-285	-519	-1106	5658	6615	-981	-1061	8211	1924
23.68	-14	-247	-552	-1119	5900	7000	-980	-1061	8569	2083
25.61	13	-209	-562	-1127	6072	7340	-979	-982	8851	2199
27.54	42	-172	-602	-1134	6178	7618	-979	-982	9052	2272
29.47	70	-139	-641	-1150	6329	7822	-978	-981	9168	2302
31.40	99	-99	-685	-1126	6193	7971	-977	-980	9205	2291
33.34	133	-75	-1077	-794	8601	5696	-975	-978	9151	2238
35.27	169	-48	-1127	-781	8447	5686	-974	-977	9021	2145
37.20	204	-21	-1177	-769	8218	5613	-973	-976	8848	2010
39.13	239	11	125	162	7991	5298	-990	-1070	8695	1874
40.00	292	46	171	69	7684	4925	-1053	-1061	8290	1621
41.92	296	49	167	67	7597	4911	-1053	-1061	8211	1584
43.85	331	73	149	51	7215	4564	-1056	-1064	7791	1320
45.77	364	97	131	31	6733	4170	-1058	-1066	7279	1019
47.70	397	121	112	11	6177	3734	-1060	-1068	6691	682
49.62	431	144	93	-8	5552	3272	-1062	-1070	6043	309
51.54	464	167	74	-26	4856	2784	-1064	-1072	5327	-100
53.47	498	189	55	-45	4090	2253	-1066	-1074	4553	-546
55.39	531	212	32	-63	3253	1684	-1068	-1075	3738	-1059
57.32	564	234	10	-81	2346	1072	-1069	-1077	2868	-1623

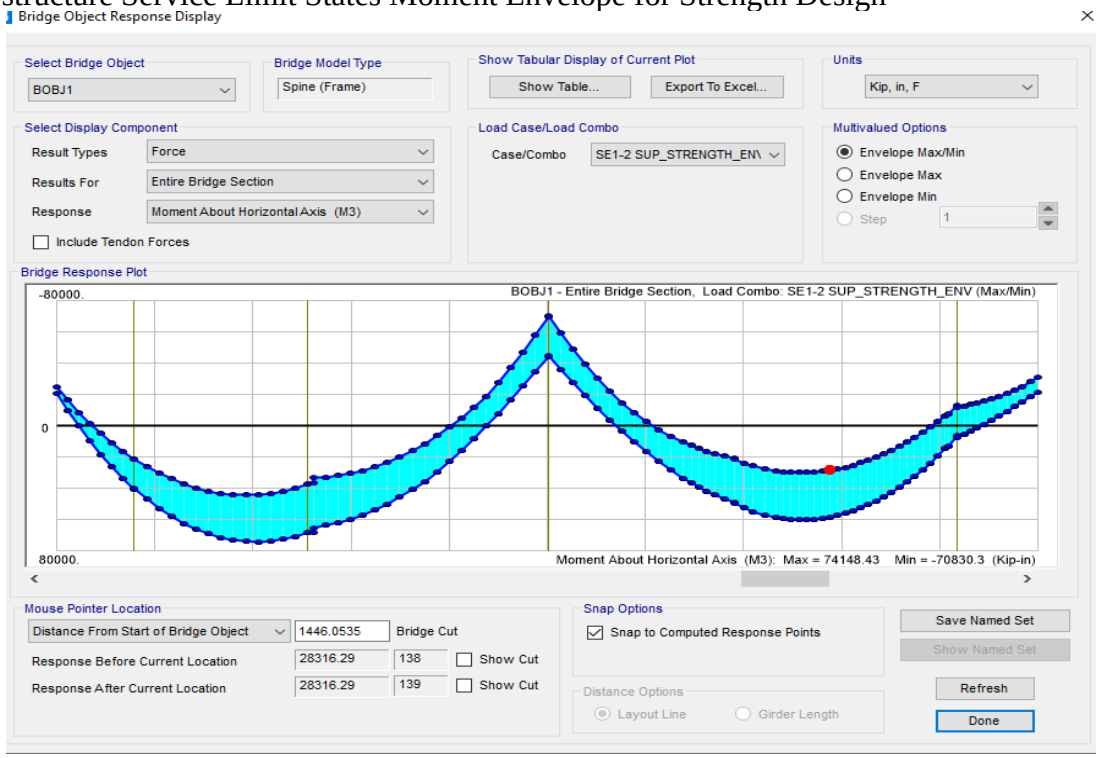
Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)
59.24	597	256	-13	-99	1370	420	-1071	-1079	1932	-2223
61.16	630	277	-37	-162	326	-1296	-1037	-1044	928	-2859
63.09	663	295	-61	-187	-785	-1913	-1037	-1044	-139	-3531
65.01	696	313	-84	-211	-1840	-2569	-1037	-1044	-1257	-4250
66.93	729	331	-108	-236	-2940	-3262	-1037	-1044	-2285	-5106
68.86	762	348	-132	-260	-4088	-3994	-1037	-1044	-3160	-6208
70.78	795	366	-157	-285	-5285	-4764	-1037	-1044	-4062	-7376
72.71	860	402	-206	-334	-7740	-6420	-1037	-1044	-5796	-9903
74.63	-418	-887	363	31	-6677	-10585	-575	-1044	-6657	-11422
76.63	827	384	-181	-309	-6489	-5573	-1037	-579	-4994	-8608
78.63	-399	-849	340	-25	-5860	-9116	-575	-579	-5839	-9895
80.54	-382	-816	319	-60	-5103	-7623	-581	-585	-5059	-8598
82.44	-364	-784	298	-92	-4380	-6192	-582	-586	-4139	-7365
84.35	-346	-751	277	-122	-3691	-4825	-582	-586	-3244	-6194
86.25	-329	-719	256	-149	-3037	-3523	-582	-586	-2378	-5087
88.16	-311	-686	236	-176	-2417	-2285	-582	-586	-1399	-4189
90.07	-293	-653	215	-202	-1831	-1114	-587	-591	-337	-3446
91.97	-272	-620	62	-226	-190	-10	-589	-593	706	-2755
93.88	-250	-587	41	-249	500	1026	-590	-594	1693	-2108
95.78	-228	-558	20	-267	1146	2071	-592	-596	2625	-1510
97.69	-207	-524	-3	-288	1768	2898	-593	-597	3481	-957
98.89	-193	-505	-4	-275	2192	3425	-594	-598	4012	-632
100.09	-179	-483	5	-253	2547	3865	-595	-599	4509	-342
101.29	-164	-463	15	-225	2914	4356	-596	-600	5012	-66
102.49	-151	-440	33	-189	3206	4768	-597	-601	5467	196
104.06	-131	-416	56	-143	3676	5341	-600	-604	6041	517
105.63	-87	-350	58	-17	3968	5860	-605	-610	6543	814
107.08	-68	-324	94	41	4238	6267	-606	-611	6940	1038
108.53	-40	-287	140	110	4524	6629	-607	-612	7294	1240
110.31	-16	-251	195	186	4755	6912	-608	-613	7640	1440
112.10	7	-221	243	250	4977	7215	-609	-614	7920	1606
113.30	24	-199	284	303	5101	7343	-610	-615	8069	1698
114.50	40	-177	327	356	5203	7438	-610	-615	8186	1774
115.70	56	-156	370	409	5281	7503	-611	-615	8275	1835
116.90	72	-134	415	460	5337	7535	-611	-616	8350	1880

Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)
118.10	90	-114	663	307	7433	5470	-610	-615	8394	1910
119.30	112	-97	723	340	7434	5446	-611	-615	8406	1924
120.50	133	-80	784	371	7404	5396	-611	-616	8387	1923
121.71	154	-63	843	400	7343	5321	-611	-616	8336	1906
122.91	176	-46	881	429	7245	5233	-608	-612	8252	1873
124.11	197	-30	938	455	7122	5121	-607	-612	8138	1825
125.31	219	-13	993	479	6967	4986	-607	-612	7990	1761
126.51	241	2	1045	497	6780	4873	-607	-612	7811	1682
127.71	262	18	1095	520	6562	4719	-606	-611	7599	1587
128.91	284	34	1142	541	6310	4542	-606	-611	7365	1476
130.11	306	49	1185	558	6027	4343	-605	-610	7103	1350
131.31	328	65	1225	572	5710	4123	-605	-610	6809	1208
132.51	350	80	1260	583	5361	3880	-604	-609	6482	1051
133.71	372	96	1291	590	4978	3617	-603	-608	6124	878
134.91	394	111	1316	593	4562	3331	-602	-607	5735	689
136.11	416	127	1336	592	4113	3025	416	127	5316	485
137.32	439	142	1350	586	3631	2697	439	142	4867	265
138.52	461	159	1350	554	3124	2262	461	159	4389	30
138.93	467	163	1368	567	2911	2187	467	163	4193	-56
140.43	517	196	1270	596	2396	1612	517	196	3561	-382
141.43	538	209	1258	337	1871	523	538	209	3068	-628
142.37	553	218	1203	295	1497	358	553	218	2715	-805
143.32	568	227	1149	252	1111	186	568	227	2349	-988
144.52	587	239	1074	198	605	-35	587	239	1871	-1228
145.72	606	250	993	141	84	-263	606	250	1380	-1479
146.92	625	262	909	83	-457	-500	625	262	868	-1742
148.12	644	273	817	24	-1010	-741	644	273	345	-2018
149.32	663	285	726	-37	-1584	-992	663	285	-199	-2314
150.52	680	295	653	-71	-1982	-1451	680	295	-750	-2684
151.72	696	303	611	-80	-2396	-1934	696	303	-1265	-3154
152.93	710	311	590	-75	-2818	-2428	710	311	-1649	-3726

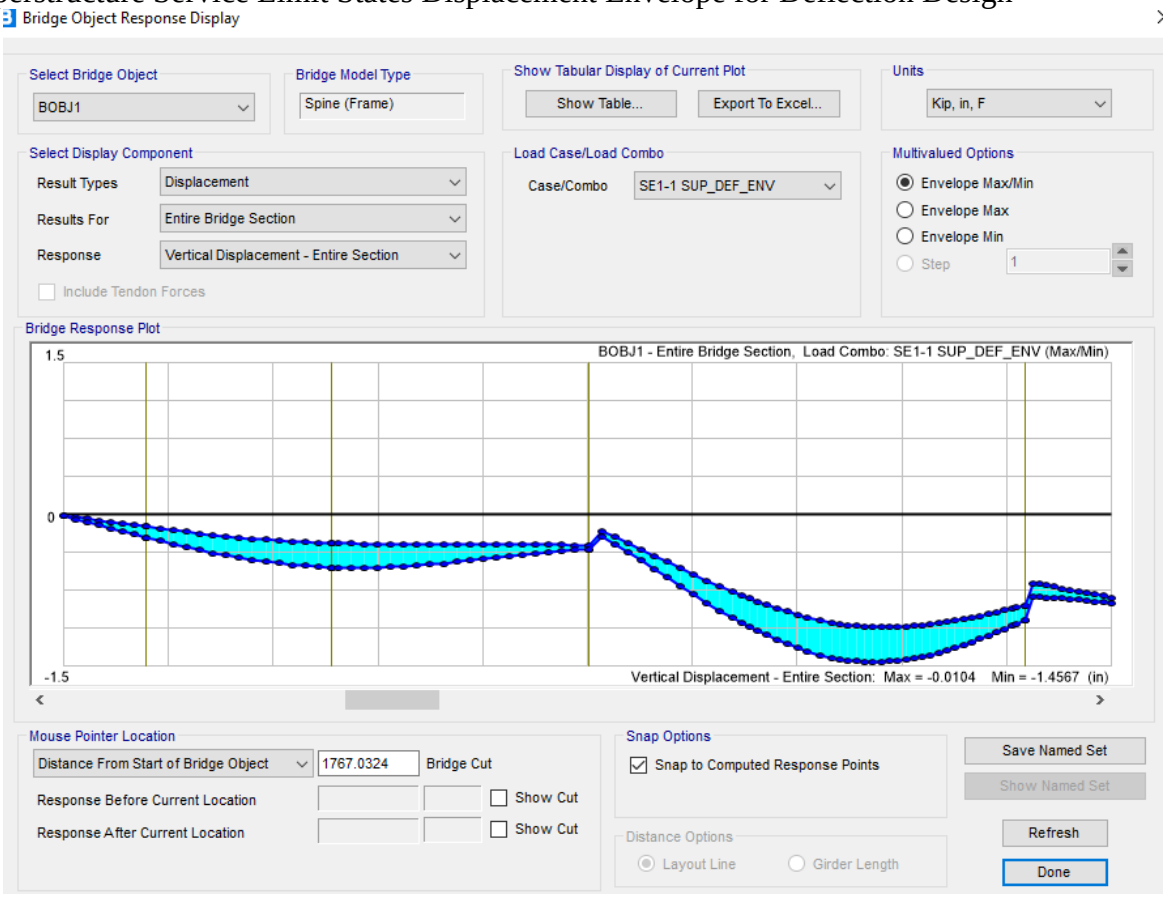
Superstructure Strength Limit States Moment Envelope



Superstructure Service Limit States Moment Envelope for Strength Design



Superstructure Service Limit States Displacement Envelope for Deflection Design

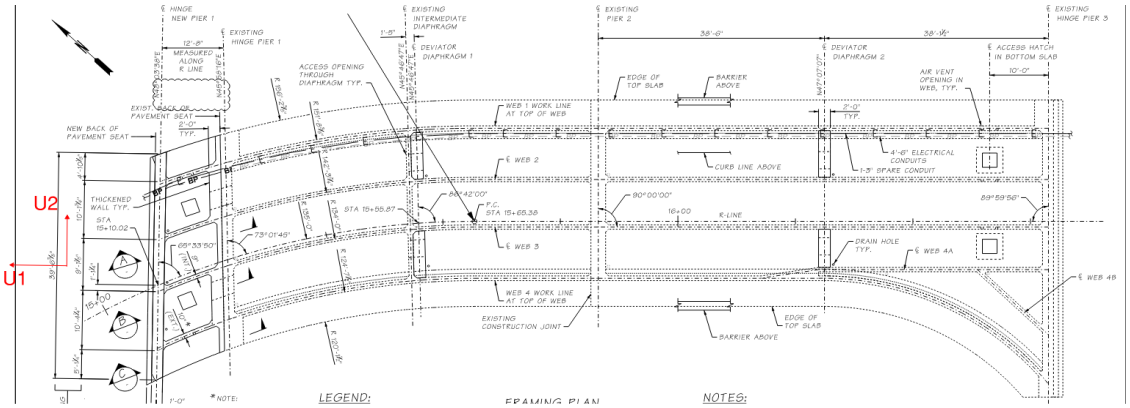


Bridge End Displacements

Longitudinal Displacements at Bridge Ends

Description: This sheet summarizes the longitudinal and transverse displacements at the end of the bridge due to unfactored temperature gradients, temperature rise and drop, and creep and shrinkage. The values are to be used in the expansion joints design.

Sign Convention (U1&U2)



Pier 3

Node	Load Case	U1 Long. Disp.	U2 Trans. Disp.
1	TG-	0.023 in	-0.0004 in
1	TG+	-0.076 in	0.0014 in
1	TU-	0.573 in	0.0017 in
1	TU+	-0.322 in	-0.0010 in
1	CR+SH	0.043 in	0.0001 in

New Pier 1

Node	Load Case	U1 Long. Disp.	U2 Trans. Disp.
140	TG-	-0.013 in	-0.001 in
140	TG+	0.042 in	0.003 in
140	TU-	-0.122 in	0.005 in
140	TU+	0.069 in	-0.003 in
140	CR+SH	-0.034 in	-0.002 in

Note: Transverse is normal to Pier 3 hinge CL and longitudinal is norm to the transverse direction

Forces and Displacements at Bottom of Box at New Pier 1

Note:

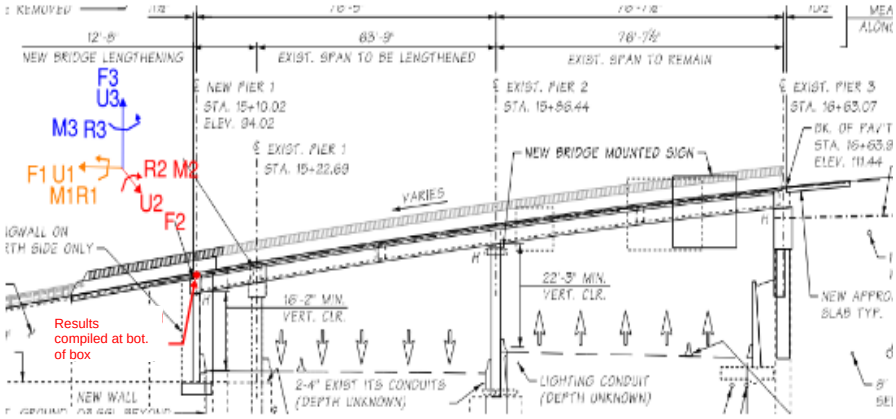
1. The size of the footing is assumed to be 24.5' (D) x40.4' (B) x 4' (T). The size of the pier wall is 37.5'(B)x3'(T)x19.5' (L). The heel soil area is 485.5 sq.ft and the height is 26'. The soil unit weight is taken as 0.13 kip/ft^3

2. The pier wall and footing centerlines are align with the centerline of the bridge, so the eccentricities in the transverse direction are ignored in the model. The eccentricity in the longituidnal direction between the wall and footing is also ignored.

3. The self weight of the soil on the heel is applied at the top of the footing with vertical load of 1639 kips and a moment of 10255 kip.ft due to the eccentricity between the CL of toe and CL of footing

1 Sign Convention

Positive Forces and Positive Displacement Directions



Note: Results are obtained from CSiBridge model at: \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\CSiBridge Model\CSIBridge Model\Model with New and Existing Structure. The load combinations are:
"DL2_ShortB_Existing" -> Existing structure
"DL3_ShortC_(DC+PT)" -> Extension+DC1+PT
"DL3+Soil on heel" -> Extension+DC1+PT+Soil on Heel
"DL3+PT_AbutRemove_Short_STG" -> Extension+DC1+PT+Pier 1 Removal
"DL3b+PT_Short" -> Extension+DC1+PT+Pier 1 Removal+DC2
"PT" -> PT

2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	0	0	0	0	0	0	~~Existing structure~~
2	-6	0	192	32	0	16	Extension+DC1+PT
2.5	-49	11	-53	561	0	14	Extension+DC1+PT+Soil on Heel
3	-28	3	404	-999	0	-4	Extension+DC1+PT+Pier 1 Removal
4	-28	3	490	-887	0	-3	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0	0	0	0	0	0	~~Existing structure~~
2	0.01	0	-0.03	0	-0.0003	0	Extension+DC1+PT
2.5	-0.10	0	-0.25	0	0.0012	0	Extension+DC1+PT+Soil on Heel
3	-0.13	0	-0.33	0	-0.0001	0	Extension+DC1+PT+Pier 1 Removal
4	-0.14	0	-0.34	0	-0.0001	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	0	0	0	0	0	0
PT	-6	1	-90	75	0	17
DC1	0	-1	283	-43	0	0
Soil on Heel	-44	11	-246	528	0	-3
Pier Removal	21	-8	457	-1559	0	-18
DC2	0	0	86	112	0	1

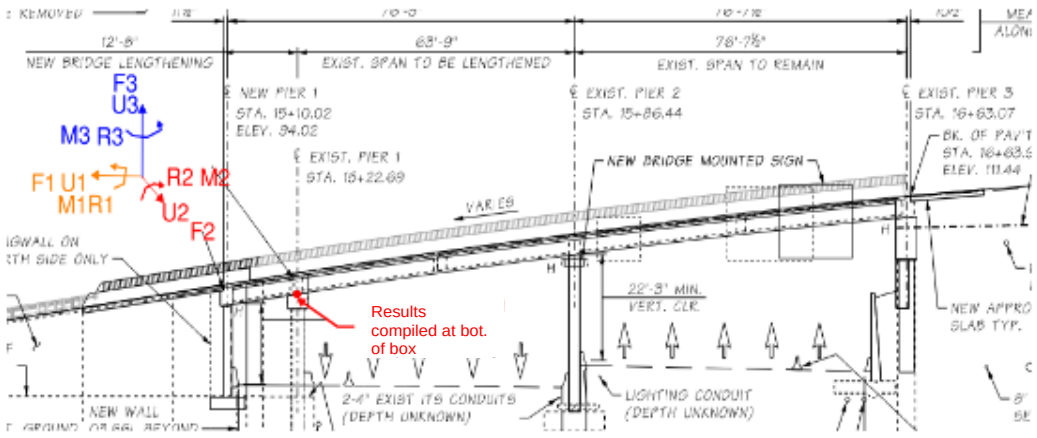
5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0	0	0	0	0	0
PT	-0.06	0	0.00	0	0.0002	0
DC1	0.07	0	-0.03	0	-0.0005	0
Soil on Heel	-0.11	0	-0.22	0	0.0015	0
Pier Removal	-0.04	0	-0.07	0	-0.0013	0
DC2	0	0	-0.01	0	0	0

Forces and Displacements at Bottom of Box at Existing Pier 1

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	0	-1	333	-439	0	6	Existing structure
2	-1	4	272	-568	0	18	Extension+DC1+PT
2.5	20	-5	567	-301	0	-16	Extension+DC1+PT+Soil on Heel
3	0	0	0	0	0	0	Extension+DC1+PT+Pier 1 Removal
4	0	0	0	0	0	0	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0.01	0	-0.26	0	0.0022	0	Existing structure
2	0.01	0	-0.25	0	0.0019	0	Extension+DC1+PT
2.5	-0.10	0	-0.34	0	0.0011	0	Extension+DC1+PT+Soil on Heel
3	-0.13	-0.01	-0.58	0	0.0020	0	Extension+DC1+PT+Pier 1 Removal
4	-0.13	-0.01	-0.59	0	0.0020	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	2	-7	-125	-7	-2	0
PT	-1	4	-92	-292	0	12
DC1	0	1	30	163	0	0
Soil on Heel	21	-9	295	267	5	-33
Pier Removal	-20	5	-567	301	-5	16
DC2	0	0	0	0	0	0

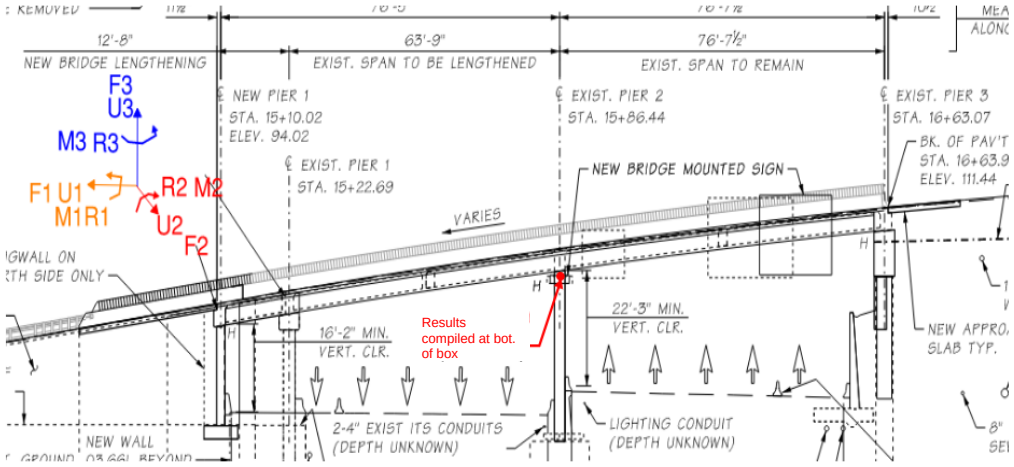
5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0.01	0	-0.26	0	0.0022	0
PT	-0.03	0	0	0	0	0
DC1	0.03	0	0.01	0	-0.0003	0
Soil on Heel	-0.11	0	-0.09	0	-0.0008	0
Pier Removal	-0.02	-0.01	-0.24	0	0.0008	0
DC2	0	0	-0.01	0	0	0

Forces and Displacements at Bottom of Box at Pier 2

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	4	4	931	487	0	27	Existing structure
2	6	-4	922	260	0	31	Extension+DC1+PT
2.5	-20	7	868	-146	0	-32	Extension+DC1+PT+Soil on Heel
3	19	-1	1000	320	0	21	Extension+DC1+PT+Pier 1 Removal
4	19	-1	1003	335	0	22	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	-0.03	0.01	-0.32	0	-0.0007	0	Existing structure
2	-0.06	0.02	-0.32	0	-0.0006	0	Extension+DC1+PT
2.5	-0.20	0.02	-0.31	0	-0.0007	0	Extension+DC1+PT+Soil on Heel
3	-0.19	0.01	-0.34	0	-0.0003	0	Extension+DC1+PT+Pier 1 Removal
4	-0.19	0.01	-0.34	0	-0.0002	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	4	4	931	487	0	27
PT	3	-7	-11	31	0	4
DC1	0	0	1	-258	0	0
Soil on Heel	-27	10	-53	-406	-5	-63
Pier Removal	39	-8	132	466	5	53
DC2	0	0	3	16	0	1

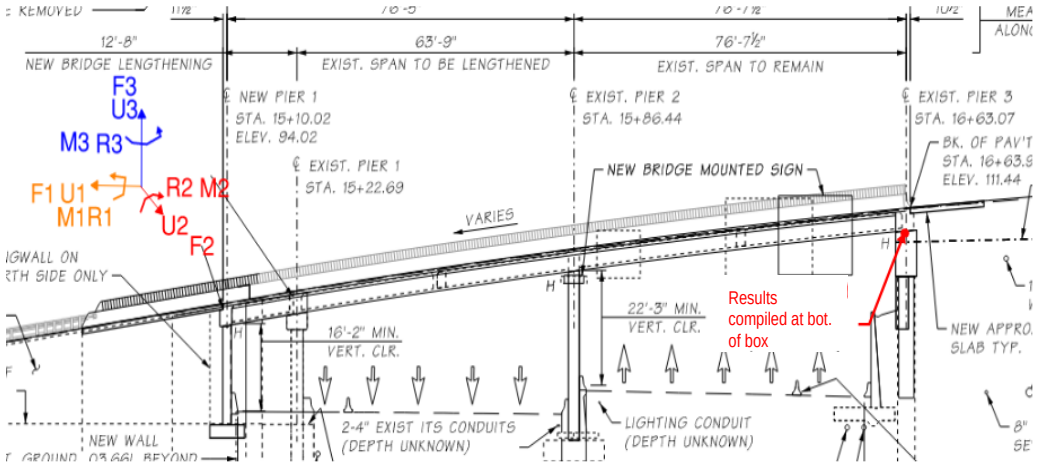
5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	-0.03	0.01	-0.32	0	-0.0007	0
PT	0	0	0	0	0	0
DC1	-0.03	0	0	0	0	0
Soil on Heel	-0.14	0	0.01	0	0	0
Pier Removal	0.01	0	-0.03	0	0.0004	0
DC2	0	0	0	0	0	0

Forces and Displacements at Bottom of Box at Pier 3

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	-3	-2	520	-981	0	48	Existing structure
2	0	0	532	-1114	0	131	Extension+DC1+PT
2.5	9	0	536	-1144	0	-133	Extension+DC1+PT+Soil on Heel
3	9	-2	514	-998	0	120	Extension+DC1+PT+Pier 1 Removal
4	9	-2	616	-990	0	122	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0.05	0	-0.01	0	-0.0019	0	Existing structure
2	-0.01	0	-0.01	0	-0.0018	0	Extension+DC1+PT
2.5	-0.15	0	-0.01	0	-0.0018	0	Extension+DC1+PT+Soil on Heel
3	-0.15	0	-0.01	0	-0.0016	0	Extension+DC1+PT+Pier 1 Removal
4	-0.15	0	-0.01	0	-0.0016	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	-3	-2	520	-981	0	48
PT	4	2	12	-167	0	80
DC1	0	0	-1	34	0	3
Soil on Heel	9	1	4	-30	2	-264
Pier Removal	0	-2	-22	146	-2	254
DC2	0	0	101	7	0	2

5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0.05	0	-0.01	0	0	0
PT	-0.01	0	0	0	0.0001	0
DC1	-0.05	0	0	0	0.0001	0
Soil on Heel	-0.14	0	0	0	0.0000	0
Pier Removal	-0.01	0	0	0	0.0002	0
DC2	0	0	0	0	0	0

Forces and Displacements at Bottom of Footing at New Pier 1

Note:

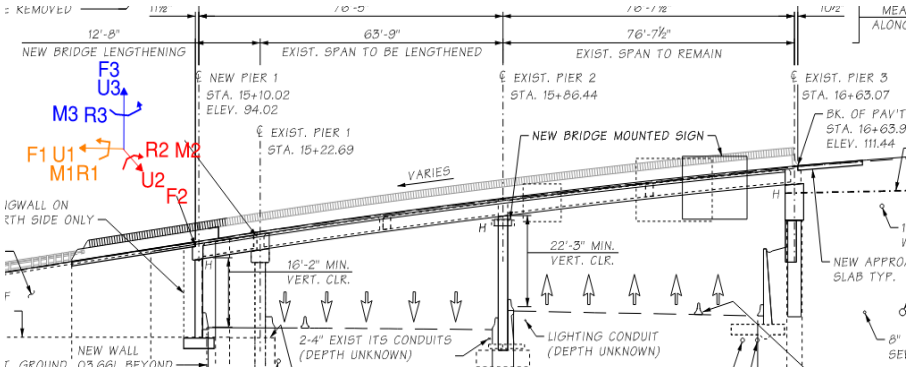
1. The size of the footing is assumed to be 24.5' (D) x40.4' (B) x 4' (T). The size of the pier wall is 37.5'(B)x3'(T)x19.5' (L). The heel soil area is 485.5 sq.ft and the height is 26'. The soil unit weight is taken as 0.13 kip/ft^3

2. The pier wall and footing centerlines are align with the centerline of the bridge, so the eccentricities in the transverse direction are ignored in the model. The eccentricity in the longituidnal direction between the wall and footing is also ignored.

3. The self weight of the soil on the heel is applied at the top of the footing with vertical load of 1639 kips and a moment of 10255 kip.ft due to the eccentricity between the CL of toe and CL of footing

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	0	0	0	0	0	0	~~Existing structure~~
2	-6	0	1127	36	-130	16	Extension+DC1+PT
2.5	-49	11	2522	298	9097	14	Extension+DC1+PT+Soil on Heel
3	-28	3	2979	-1071	9587	-4	Extension+DC1+PT+Pier 1 Removal
4	-28	3	3065	-956	9588	-3	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0	0	0	0	0	0	~~Existing structure~~
2	0	0	-0.18	0	0	0	Extension+DC1+PT
2.5	0.01	0	-0.40	0	-0.0006	0	Extension+DC1+PT+Soil on Heel
3	0	0	-0.47	0	-0.0006	0	Extension+DC1+PT+Pier 1 Removal
4	0	0	-0.49	0	-0.0006	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	0	0	0	0	0	0
PT	-6	1	-90	58	-132	17
DC1	0	-1	1217	-22	1	0
Soil on Heel	-44	11	1395	262	9227	-3
Pier Removal	21	-8	457	-1369	490	-18
DC2	0	0	86	114	1	1

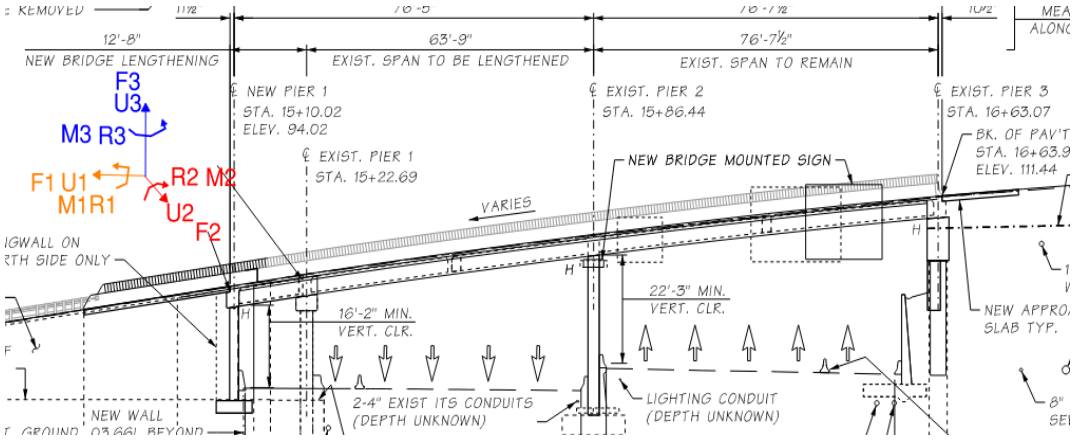
5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0	0	0	0	0	0
PT	0	0	-0.01	0	0.00001	0
DC1	0	0	-0.16	0	0	0
Soil on Heel	0.01	0	-0.22	0	-0.0006	0
Pier Removal	0	0	-0.07	0	0	0
DC2	0	0	0	0	0	0

Forces and Displacements at Bottom of Footing at Existing Pier 1

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	0	-1	875	-402	-8	6	Existing structure
2	-1	4	834	-691	-39	18	Extension+DC1+PT
2.5	20	-5	1129	-159	-601	-16	Extension+DC1+PT+Soil on Heel
3	0	0	0	0	0	0	Extension+DC1+PT+Pier 1 Removal
4	0	0	0	0	0	0	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0	0	-0.26	0	0	0	Existing structure
2	0	0	-0.25	0	0	0	Extension+DC1+PT
2.5	0	0	-0.32	0	0	0	Extension+DC1+PT+Soil on Heel
3	0	0	0	0	0	0	Extension+DC1+PT+Pier 1 Removal
4	0	0	0	0	0	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	2	-7	-125	-7	-2	0
PT	-1	4	92	-292	-28	12
DC1	0	1	-134	3	-2	0
Soil on Heel	21	-9	295	532	-562	-33
Pier Removal	-20	5	-1129	159	601	16
DC2	0	0	0	0	0	0

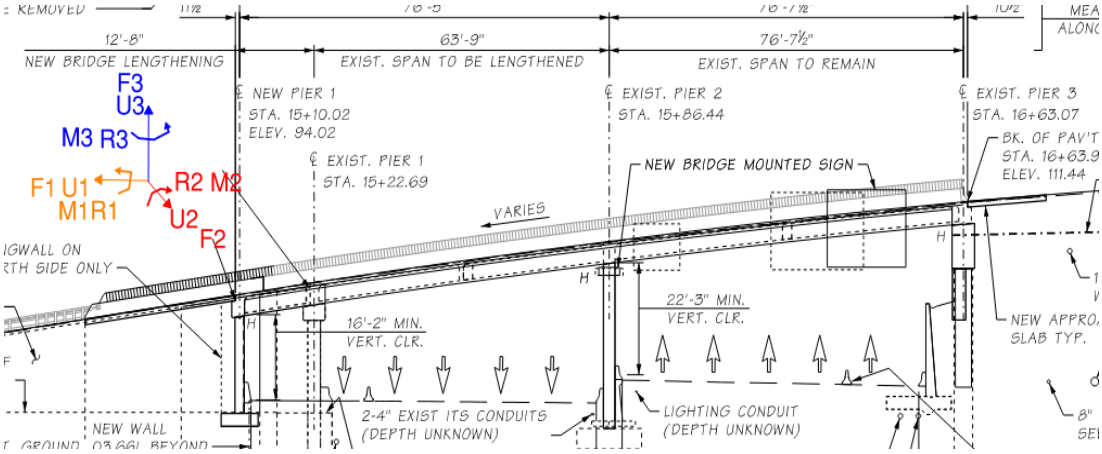
5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0	0	-0.26	0	0	0
PT	0	0	0.03	0	0	0
DC1	0	0	-0.01	0	0	0
Soil on Heel	0	0	-0.08	0	0	0
Pier Removal	0	0	0.32	0	0	0
DC2	0	0	0	0	0	0

Forces and Displacements at Bottom of Footing at Pier 2

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	4	4	1592	357	130	27	Existing structure
2	6	-4	1583	389	233	31	Extension+DC1+PT
2.5	20	-7	1529	390	742	32	Extension+DC1+PT+Soil on Heel
3	19	-1	1661	320	686	21	Extension+DC1+PT+Pier 1 Removal
4	19	-1	1664	335	684	22	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0	0	-0.31	0	0	0	Existing structure
2	0	0	-0.31	0	0	0	Extension+DC1+PT
2.5	0	0	-0.30	0	-0.0001	0	Extension+DC1+PT+Soil on Heel
3	0	0	-0.33	0	0	0	Extension+DC1+PT+Pier 1 Removal
4	0	0	-0.33	0	0	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	4	4	1592	357	130	27
PT	3	-7	11	31	104	4
DC1	0	0	-20	1	0	0
Soil on Heel	14	-3	-53	0	509	1
Pier Removal	-2	5	132	-70	-56	-11
DC2	0	0	3	16	-2	1

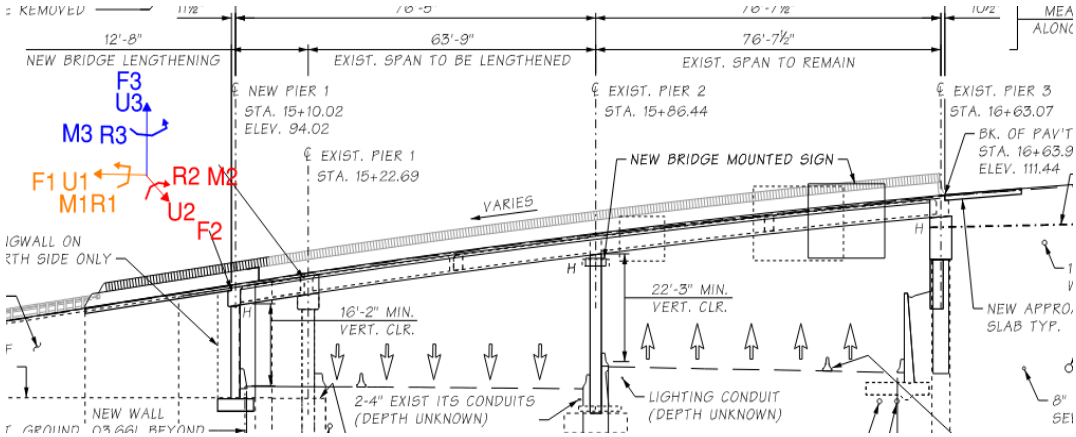
5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0	0	-0.31	0	0	0
PT	0	0	0	0	0	0
DC1	0	0	0	0	0	0
Soil on Heel	0	0	0.01	-0.0001	-0.0002	0
Pier Removal	0	0	-0.03	0	0.0002	0
DC2	0	0	0	0	0	0

Forces and Displacements at Bottom of Secant Shaft at Pier 3

1 Sign Convention

Positive Forces and Positive Displacement Directions



2 Cumulative Forces

DC1: Box girder sw+utilities sw + diaphragm sw+tendon sw +substructure sw

DC2: Barrier sw+ sidewalk sw + approach slab reaction

PT: PT stressing

Stage	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft	Stage Description
1	-3	-2	1415	-905	-104	48	Existing structure
2	0	0	1427	-1106	12	131	Extension+DC1+PT
2.5	9	0	1430	-1155	282	133	Extension+DC1+PT+Soil on Heel
3	9	-2	1409	-945	295	120	Extension+DC1+PT+Pier 1 Removal
4	9	-2	1510	-933	296	122	Extension+DC1+PT+Pier 1 Removal+DC2

3 Cumulative Deformation

Stage	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad	Stage Description
1	0.01	0	-0.01	0	-0.0001	0	Existing structure
2	0	0	-0.01	0	0	0	Extension+DC1+PT
2.5	-0.03	0	-0.01	0	-0.0002	0	Extension+DC1+PT+Soil on Heel
3	-0.03	0	-0.01	0	0	0	Extension+DC1+PT+Pier 1 Removal
4	-0.03	0	-0.01	0	0	0	Extension+DC1+PT+Pier 1 Removal+DC2

4 Forces due to Single Load Case

	F1 kip	F2 kip	F3 kip	M1 kip.ft	M2 kip.ft	M3 kip.ft
Existing	-3	-2	1415	-905	-104	48
PT	4	2	-12	-208	115	83
DC1	0	0	24	6	1	0
Soil on Heel	9	1	4	-48	270	3
Pier Removal	0	-2	-22	210	13	-13
DC2	0	0	101	12	0	2

5 Deformation due to Single Load Case

	U1 in	U2 in	U3 in	R1 rad	R2 rad	R3 rad
Existing	0.01	0	0	0	-0.0001	0
PT	-0.01	0	0	0	0.0001	0
DC1	0	0	0	0	0	0
Soil on Heel	-0.03	0	0	0	-0.0002	0
Pier Removal	-0.001	0	0	0	0	0
DC2	0	0	0	0	0	0

9. Appendix

Appendix A- Drawings

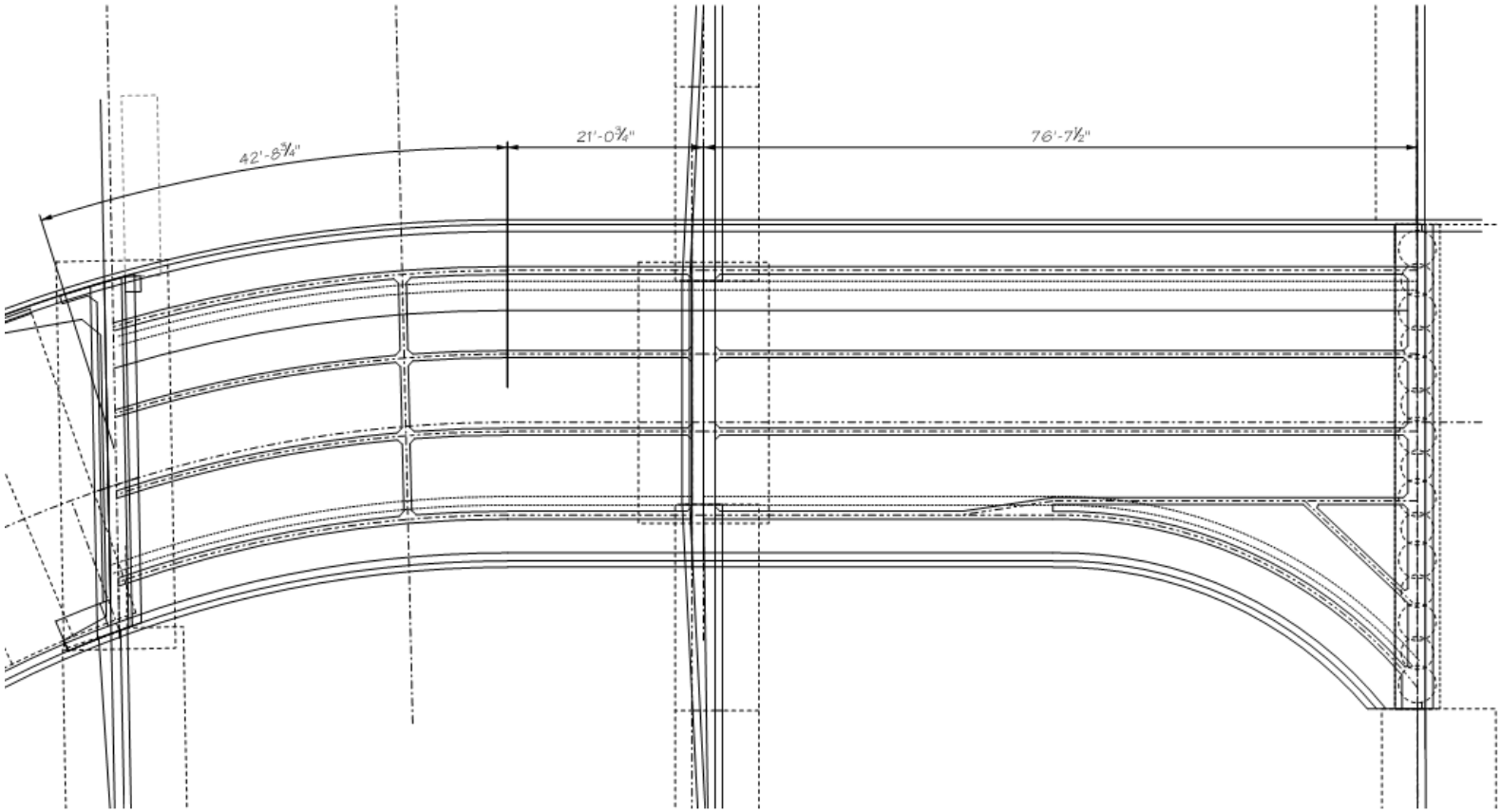


Figure A-1: Existing Bridge Stations Measured along Centerline

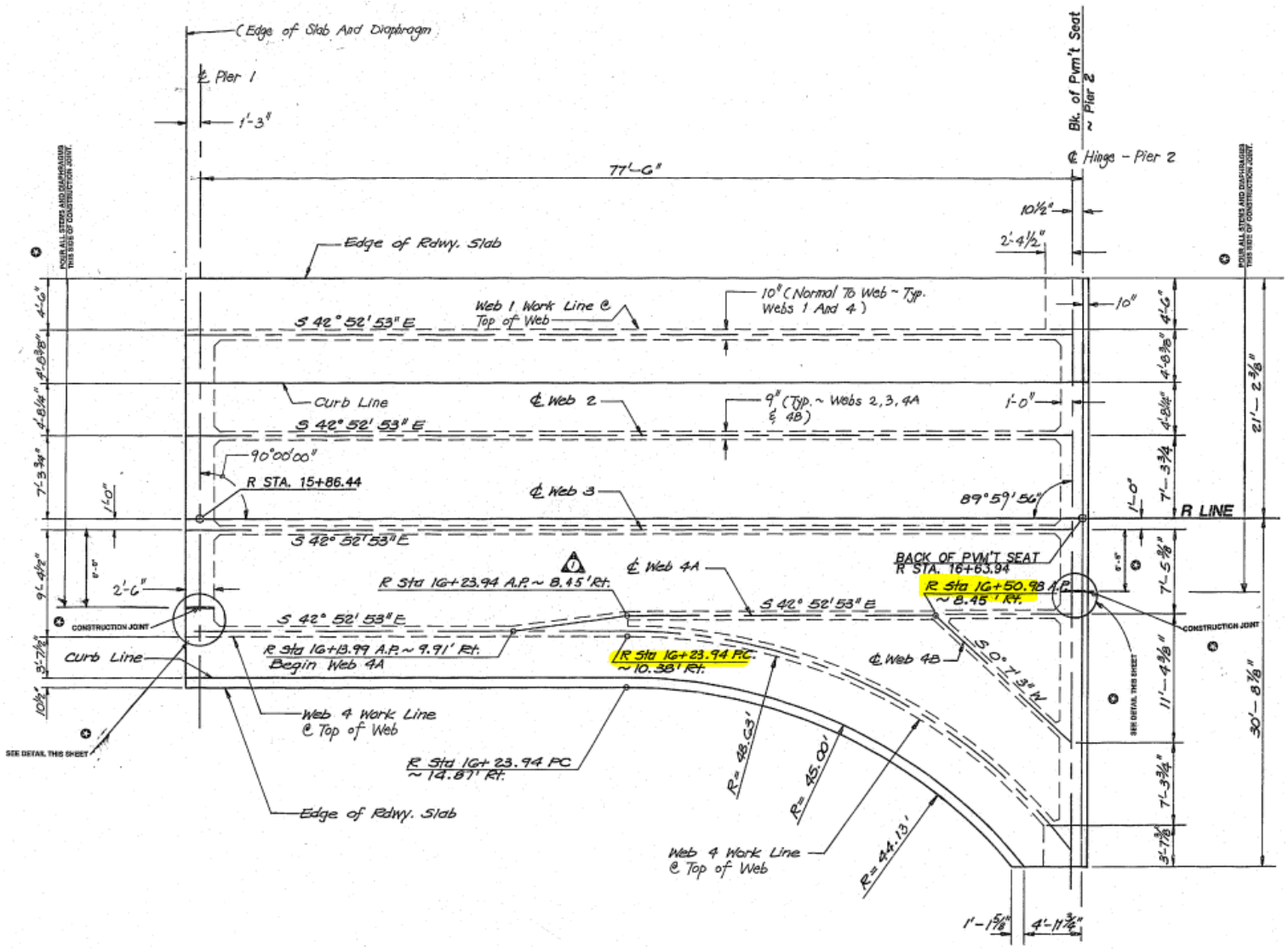


Figure A-2: Framing Plan for East Span with Stations in Flare Region Highlighted

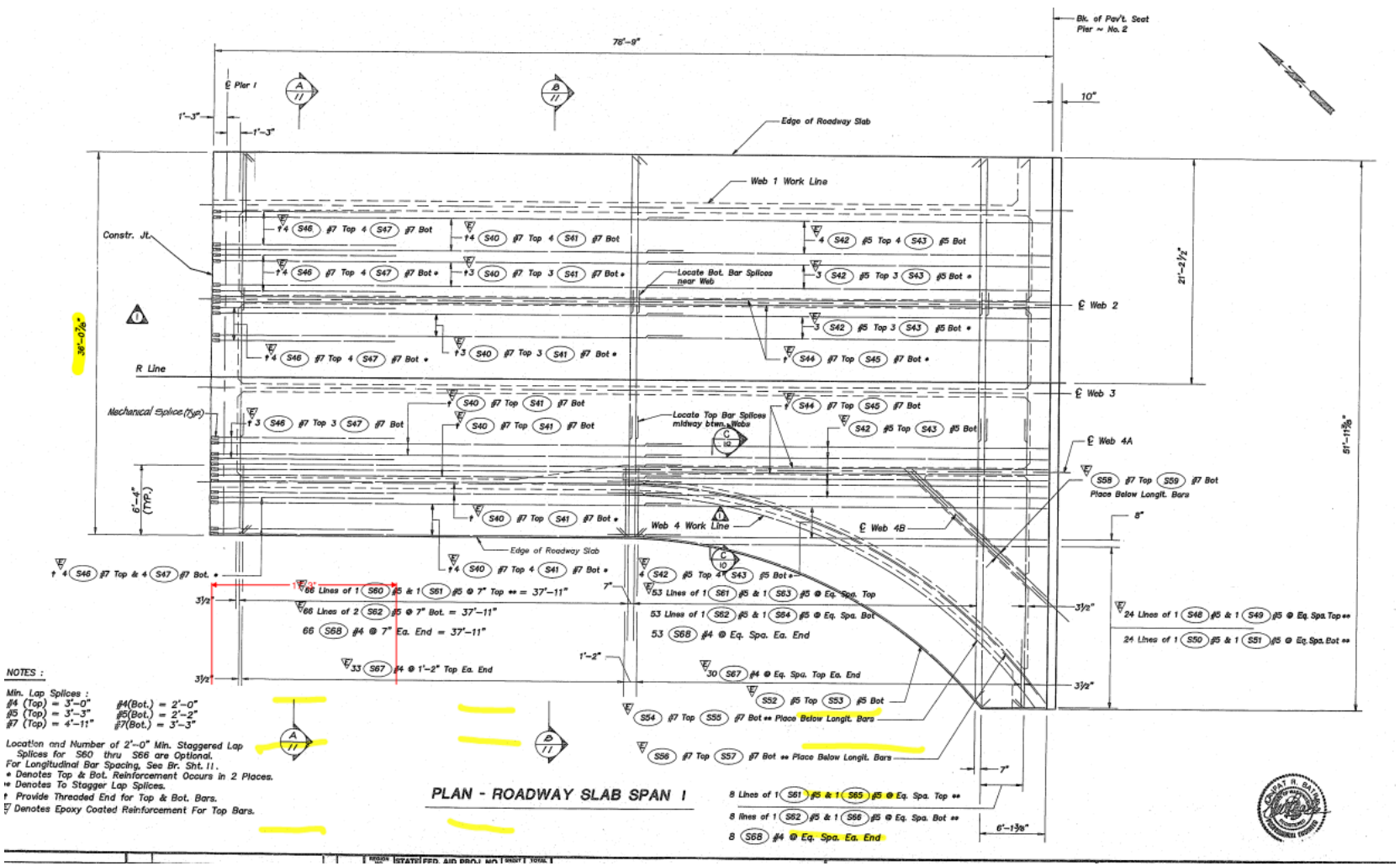


Figure 3: A-3 Plan of Roadway Slab

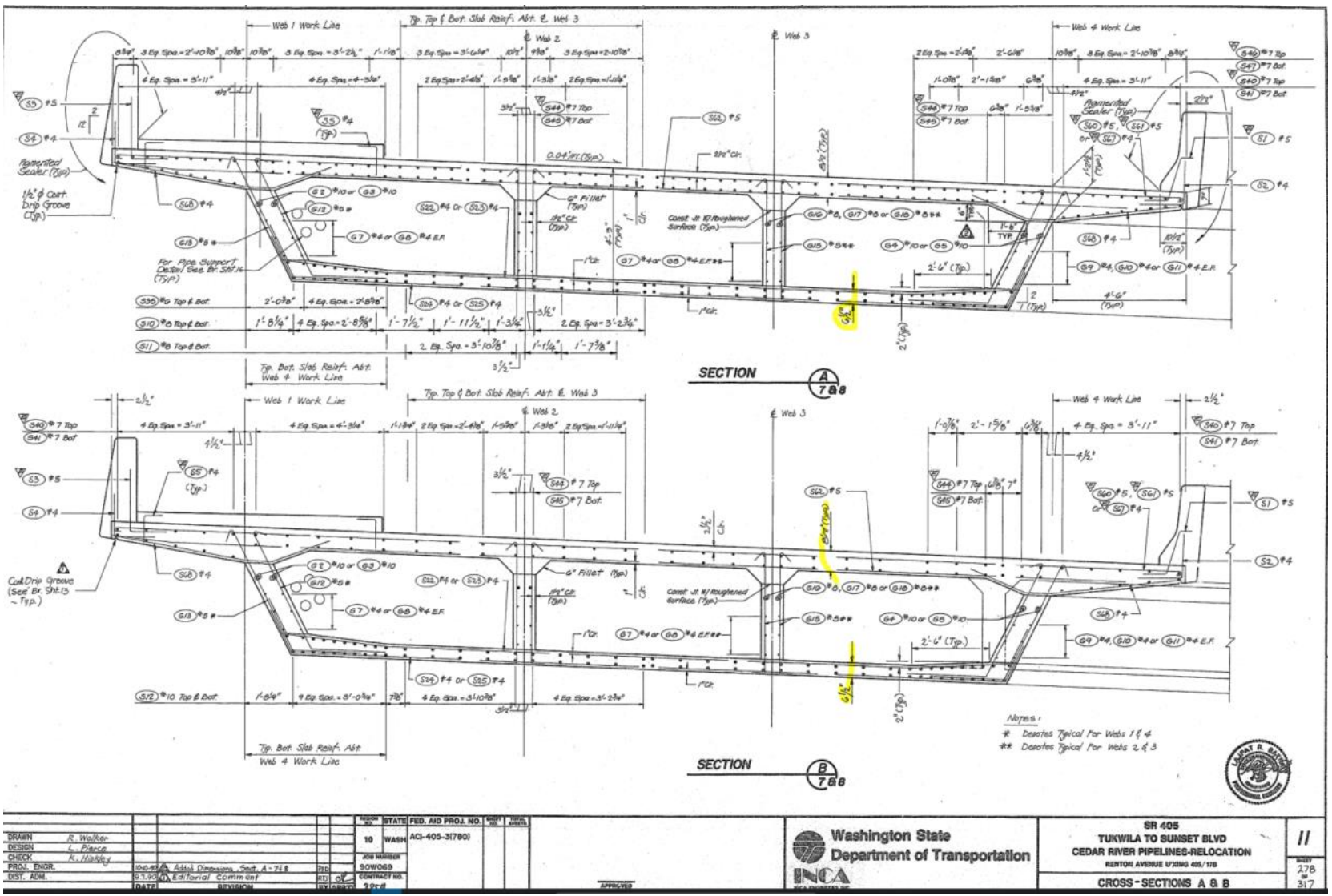


Figure A-4 East Span Cross-sections

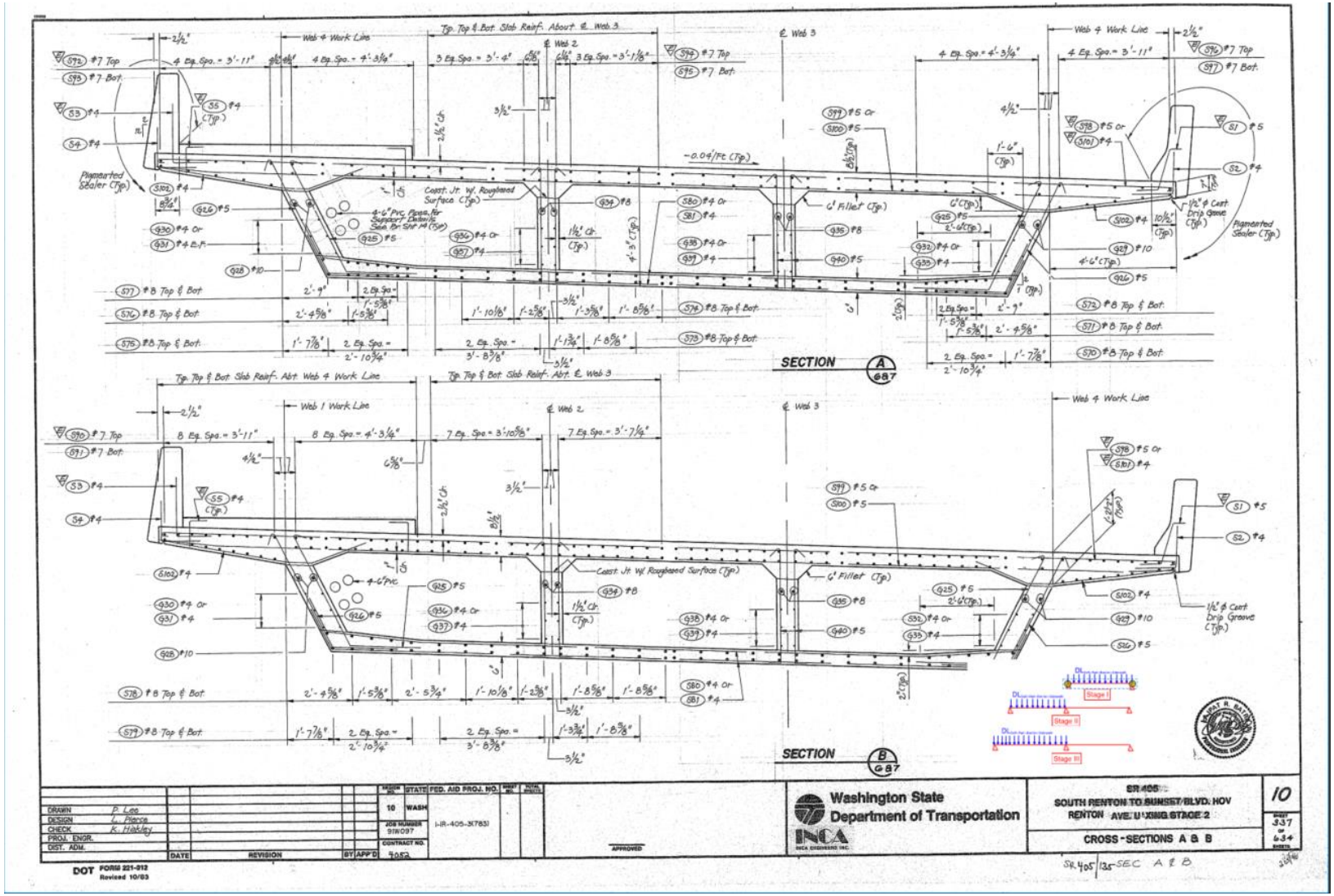


Figure A-5: West Span Cross-sections

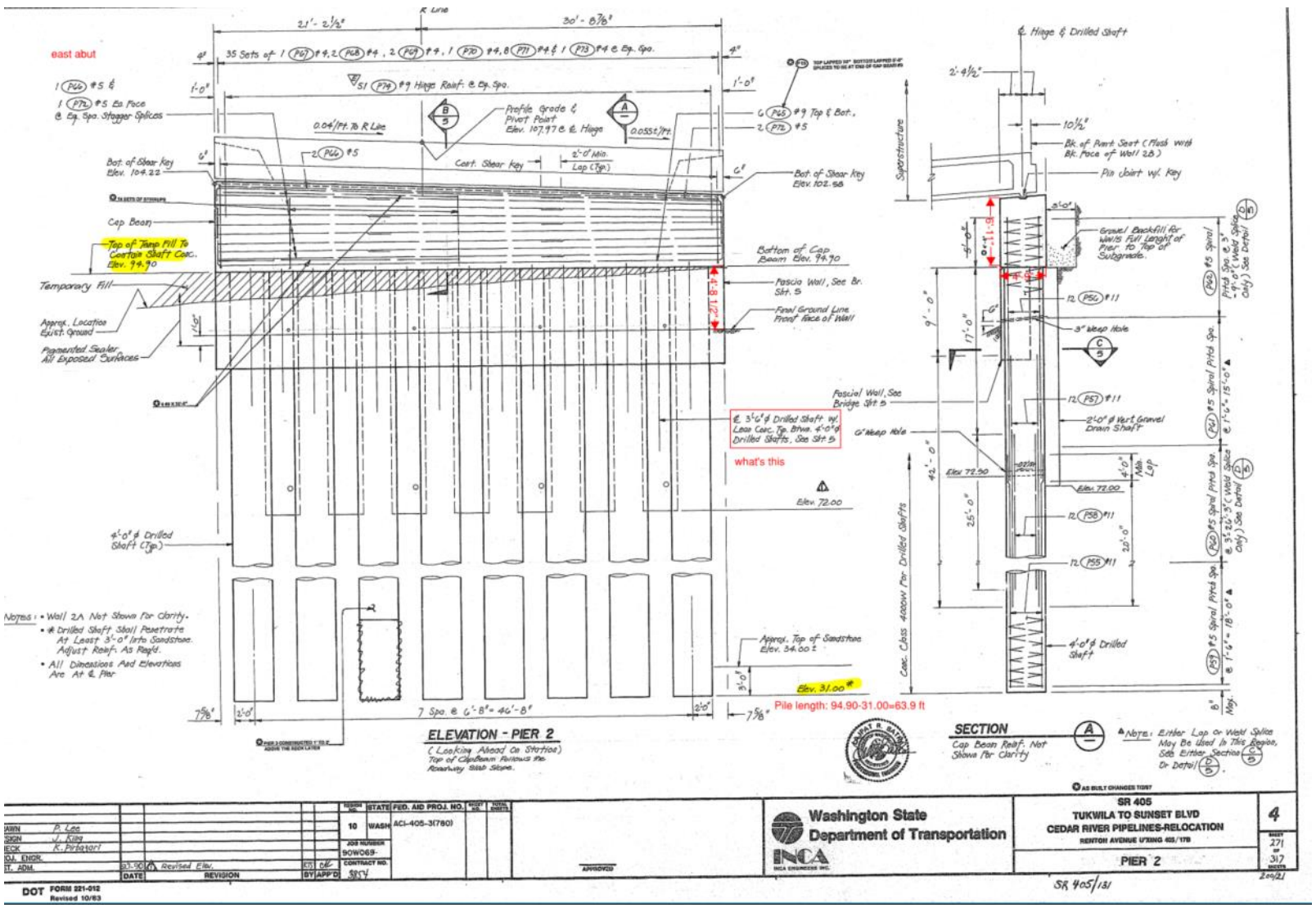


Figure A-6: East Abutment

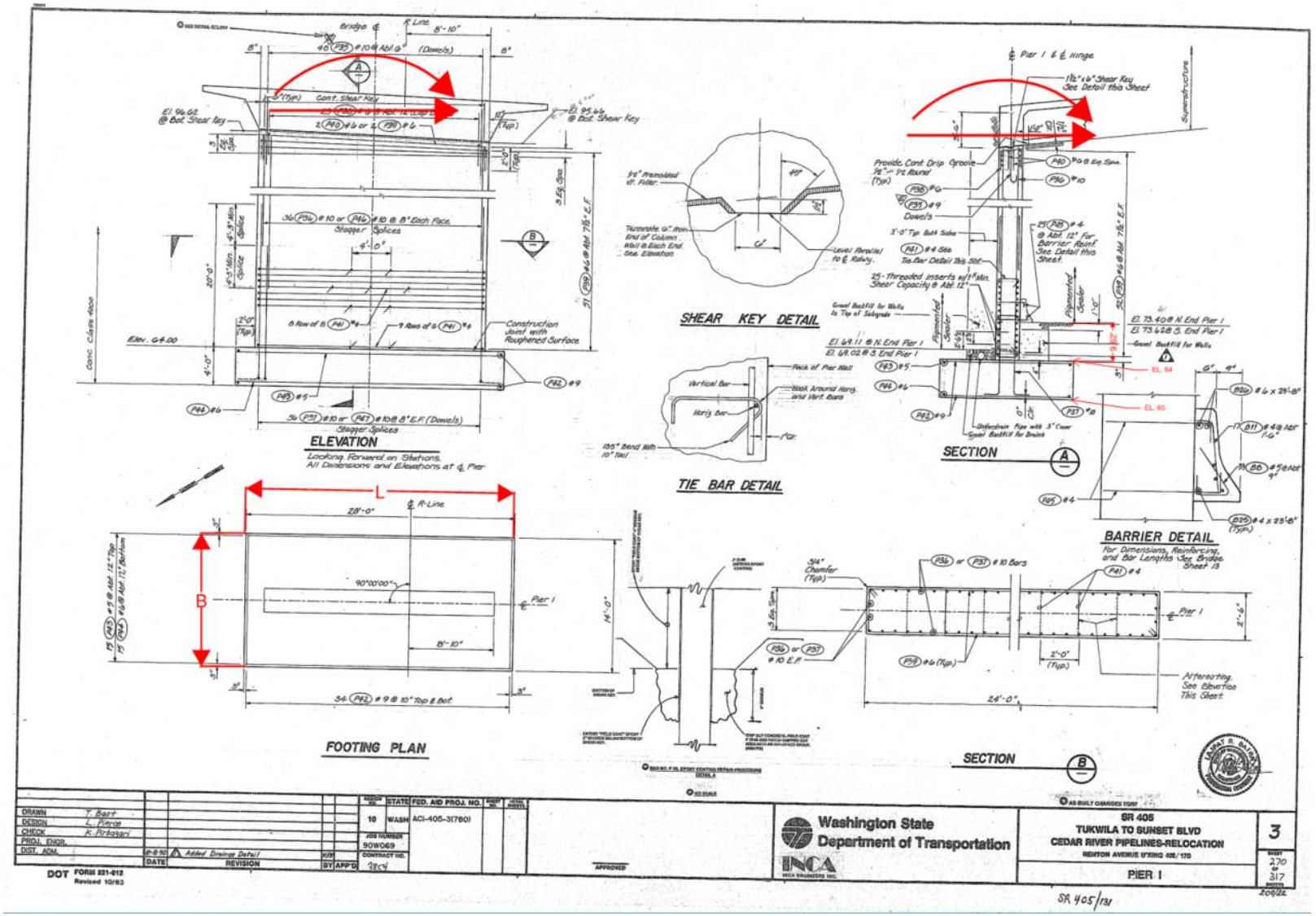


Figure A-7: Intermediate Pier

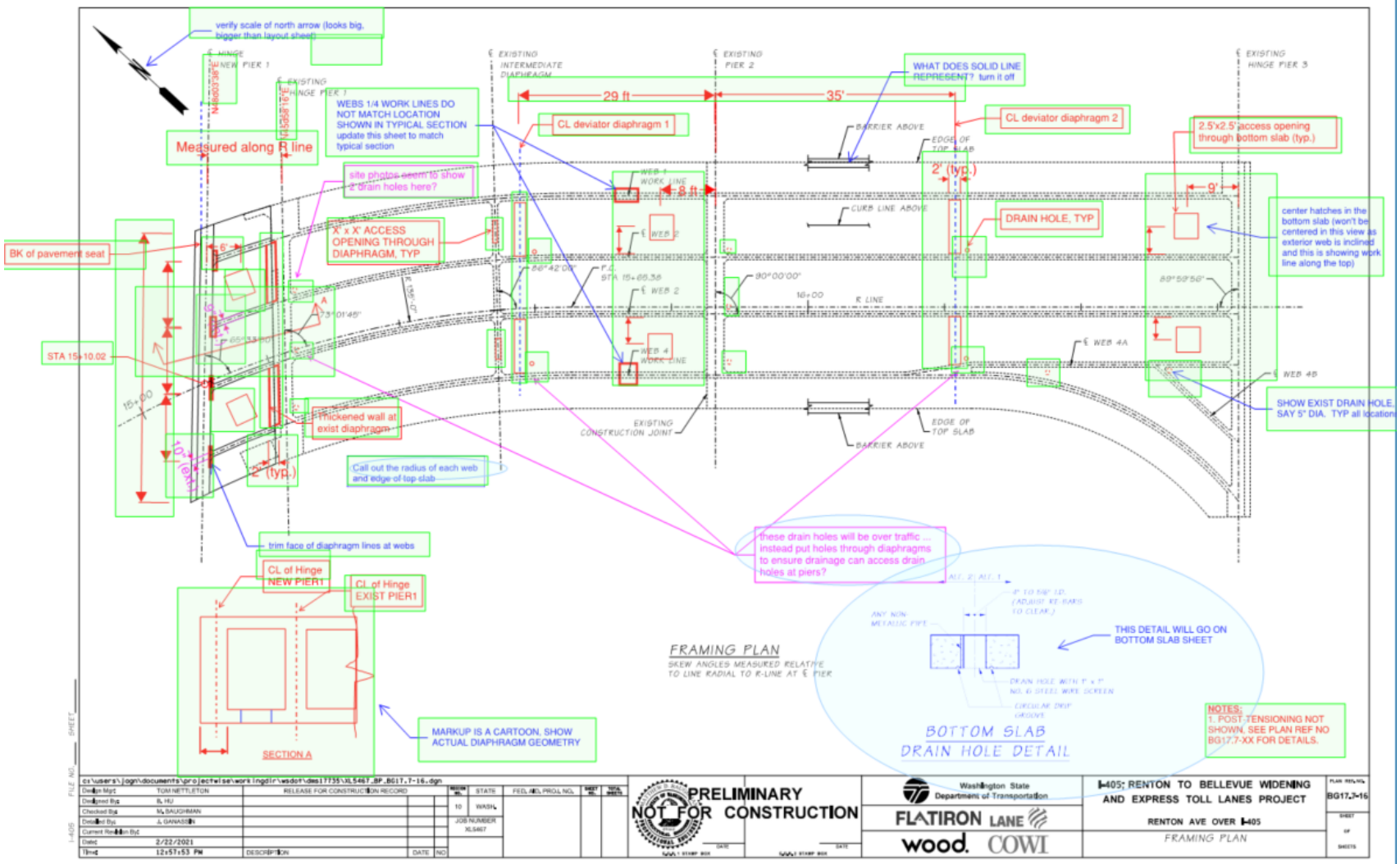


Figure A-8a: Diaphragm Locations

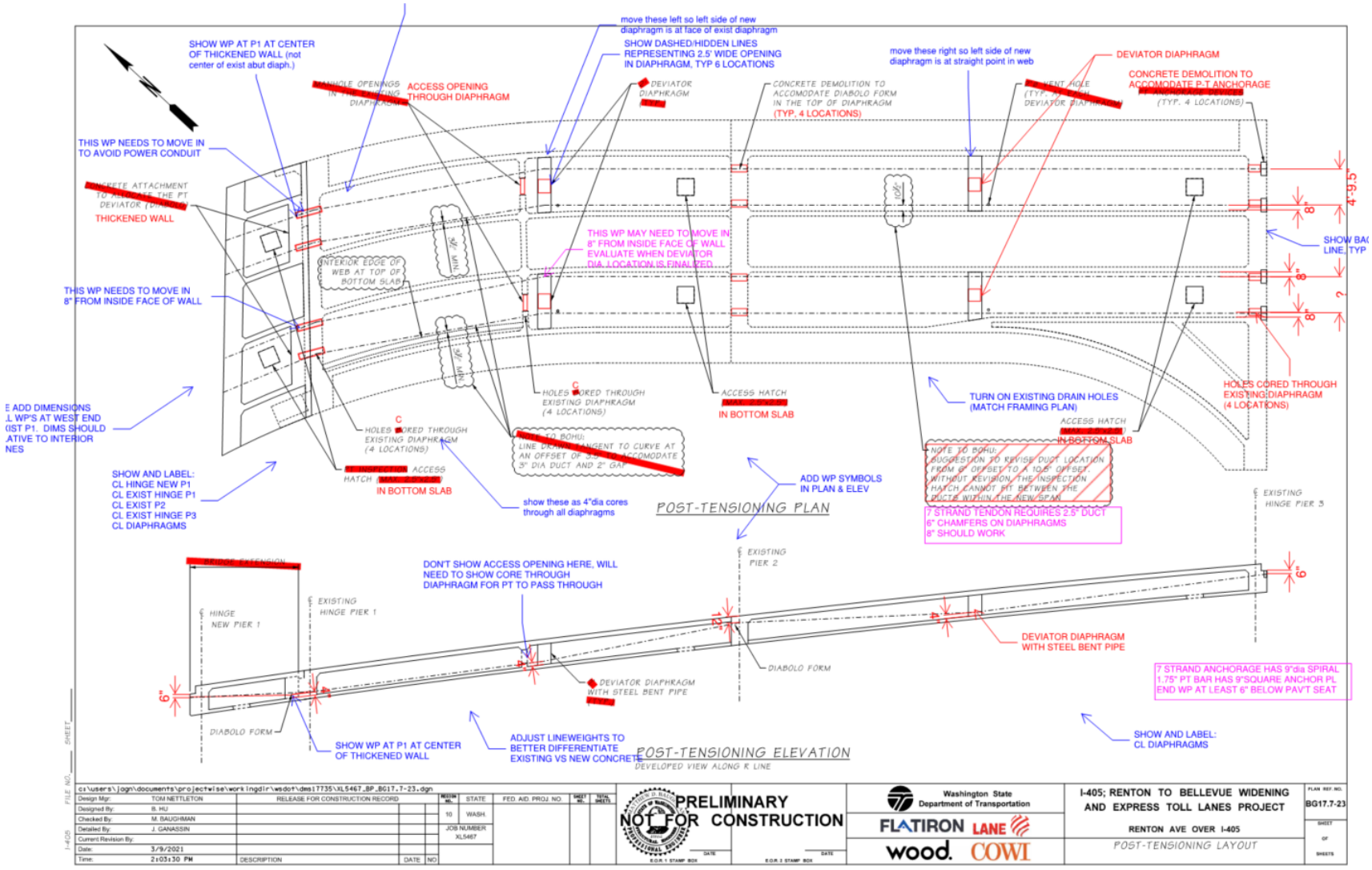


Figure A-8b: Tendon Layout

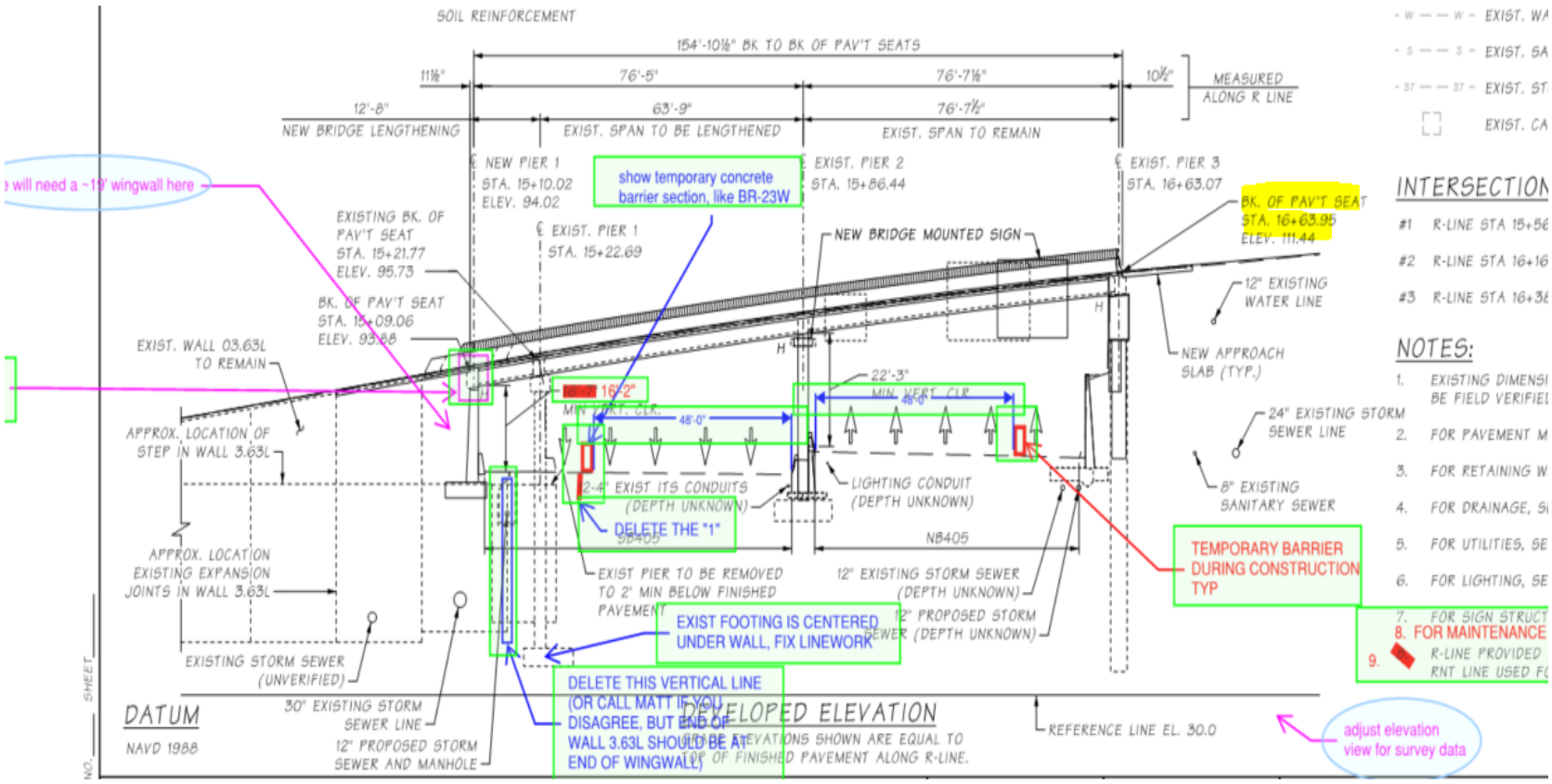


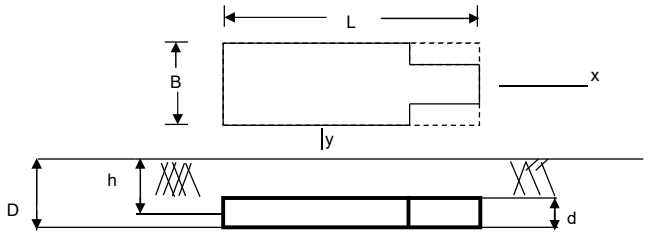
Figure A-9: Elevation View (Mark-up Dwg

Appendix B– Footing Spring Stiffness at New West Abutment (New Pier 1)

Foundation Spring Constants

Project:	**WSDOT I-405 Renton to Bellevue Widening**
Owner:	**WSDOT**
Job Number:	**00180-366-01**
Analysis by:	**MSH**
Date/Time:	**10/27/2021**
Foundation soil:	**ESU 1B**
Foundation:	**Pier 1 Proposed Spread Footing - 45.5 x 24.7**
EQ Return Interval	**SEE**
FEMA 356	

B	24.7	ft
L	45.5	ft
D	6	ft
d	4	ft
h	4	ft



υ	0.36	
G_0	1511.1	ksf
$(N_1)_{60}$	21	
γ	125	pcf
D_w	29	ft
σ'_o	750	psf
Site Class	C	
$S_{XS}/2.5$	0.51	
G/G_0	0.34	*From Table 4-7 (FEMA 356)
G	513.8	ksf

h	Distance of the effective sidewall centroid from ground surf
Vso	Shear wave velocity
υ	Poissons ratio
G_o	Initial Shear Modulus
$S_{XS}/2.5$	Effective Peak Acceleration
S_{XS}	Spectral response acceleration at short periods for the sele hazard level and damping adjusted for site class

Foundation on surface of homogeneous halfspace

Lateral x		
$K_{x,sur}$	4.842E+04	kips/ft
	4.035E+03	kips/in
Lateral y		
$K_{y,sur}$	5.103E+04	kips/ft
	4.252E+03	kips/in
Vertical		
$K_{z,sur}$	6.446E+04	kips/ft
	5.372E+03	kips/in
Rocking rx		
$K_{xx,sur}$	1.012E+07	kip-ft
	1.215E+08	kip-in
Rocking ry		
$K_{yy,sur}$	2.505E+07	kip-ft
	3.005E+08	kip-in
Torsional		
$K_{zz,sur}$	2.228E+07	kip-ft
	2.673E+08	kip-in

Foundation embedded in homogeneous halfspace

Lateral x		
Beta	1.49	
$K_{x,emb}$	7.199E+04	kips/ft
$K_{x,emb}$	6.000E+03	kips/in
Lateral y		
Beta	1.49	
$K_{y,emb}$	7.587E+04	kips/ft
$K_{y,emb}$	6.322E+03	kips/in
Vertical		
Beta	1.17	
$K_{z,emb}$	7.551E+04	kips/ft
$K_{z,emb}$	6.292E+03	kips/in
Rocking rx		
Beta	1.51	
$K_{xx,emb}$	1.528E+07	kip-ft
$K_{xx,emb}$	1.834E+08	kip-in
Rocking ry		
Beta	1.50	
$K_{yy,emb}$	3.765E+07	kip-ft
$K_{yy,emb}$	4.518E+08	kip-in
Torsional		
Beta	1.78	
$K_{zz,emb}$	3.964E+07	kip-ft
$K_{zz,emb}$	4.757E+08	kip-in

Appendix C– Footing Spring Stiffness at Intermediate Pier (Pier 2)

Bridge 17.7
Foundation Six Degree of Freedom Springs

Pier 2 - SEE

Input Summary	
Bridge 17.7 - Foundation Six Degree of Freedom Springs (SEE)	
Property	**Pier 2**
Footing Width, B (ft)	14
Footing Length, L (ft)	28
Footing Depth, d (ft)	4
Footing Embedment Depth, D_f (ft)	8.4
Depth to Groundwater, D_w (ft)	17
Engineering Soil Unit (ESU)	ESU 2A - Native Sand and Gravel
Unit Weight, γ (pcf)	120.0
$(N_1)_{60}$	29
Effective Vertical Stress, σ_o' (psf)	1008
Poisson's Ratio, v [a]	0.33
Site Class	D
S_{as} [b] / 2.5 (SEE)	0.56
G / G_0 [c]	0.340
G_0 [d] (ksf)	1950.86
G [e] (ksf)	663.293

a. Poisson's ratio correlated using Table C10.4.6.3-1 (AASHTO 2017)
b. Spectral response acceleration at short period for the selected hazard level and damping adjusted for site class
c. Effective shear modulus ratio from FEMA Table 4-7
d. The initial shear modulus calcuated using $G_0 = 20{,}000 * (N_1)_{60}^{1/3} * \text{Sqrt}(\sigma_o')$
e. Effective shear modulus

Parameter	Return Period	
	210-year (FEE)	1,000-year (SEE)
Site Class	D	D
Site Class Adjusted Peak Ground Acceleration (A_c)	0.27g	0.56g
Mean Earthquake Magnitude (Mw)	6.77	7.0

Abbreviations:
FEE = functional evaluation earthquake
SEE = safety evaluation earthquake

Table 4-7 Effective Shear Modulus Ratio (G/G_0)

	Effective Peak Acceleration, $S_{XS}/2.5$			
Site Class	$S_{XS}/2.5$ =0	$S_{XS}/2.5$ =0.1	$S_{XS}/2.5$ =0.4	$S_{XS}/2.5$ =0.8
A	1.00	1.00	1.00	1.00
B	1.00	1.00	0.95	0.90
C	1.00	0.95	0.75	0.60
D	1.00	0.90	0.50	0.10
E	1.00	0.60	0.05	*
F	*	*	*	*

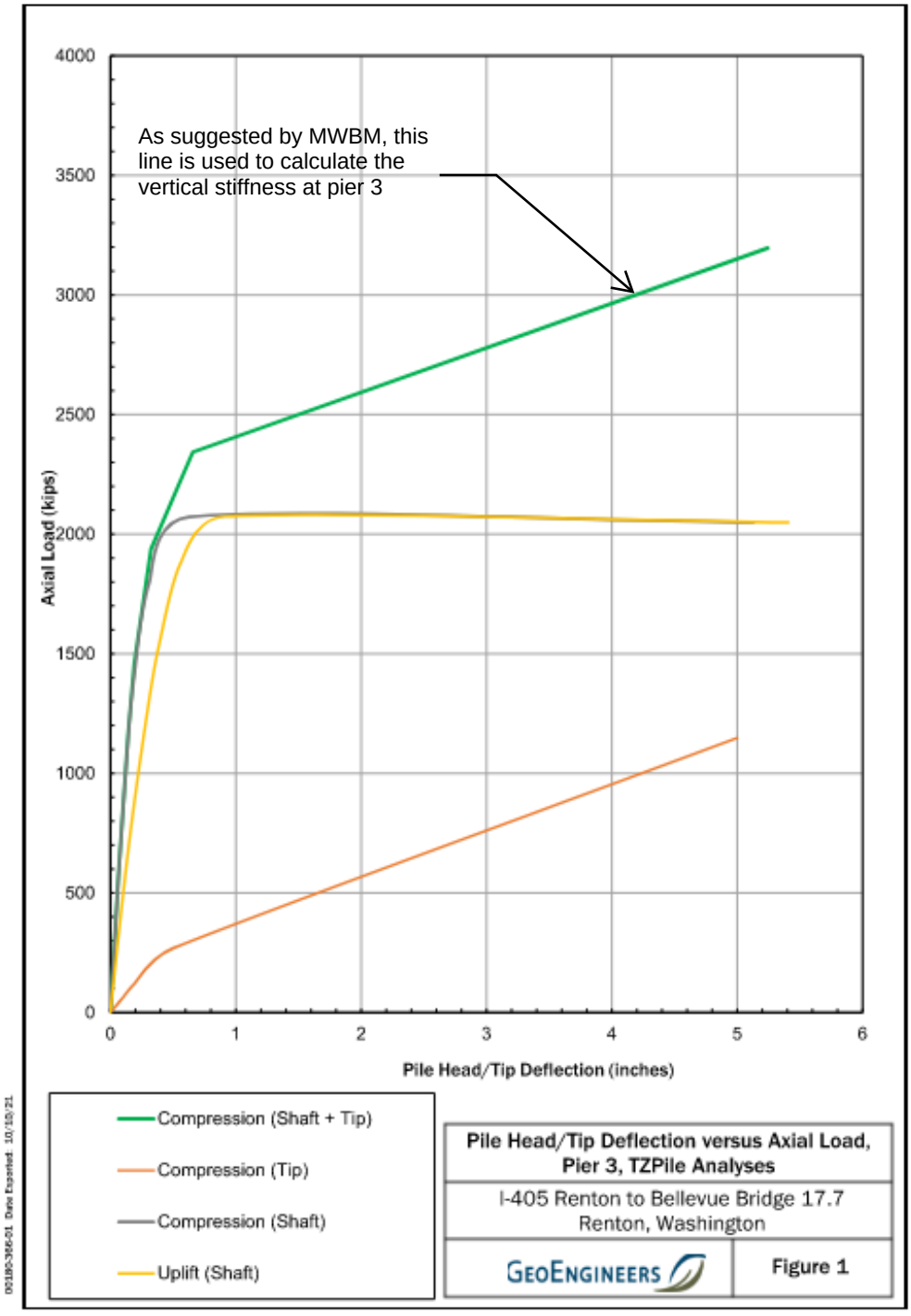
Note: The straight line interpolation for intermediate values of $S_{XS}/2.5$
* Site specific geotechnical investigation and dynamic site response analyses shall be performed.

DRAFT

							Dimensions of Footing			
Scenario	**ESU**	**Ground Elevation**	**Groundwater Elevation**	**BOF Elevation**	**Embedded Depth**	**Depth to Groundwater**	**Length**	**Width**	**Equivalent Square B**	**Footing Depth**
		feet	**feet**	**feet**	**feet**	**feet**	**feet**	**feet**	**feet**	**feet**
Pier 2	ESU 2A - Native Sand and Gravel	72	55	63.6	8.4	17	28	14	20	4

Results Summary							
Bridge 17.7 - Foundation Six Degree of Freedom Springs (SEE)							
Foundation Embedded in Homogeneous Halfspace							
Scenario	**ESU**	**$K_{x,emb}$**	**$K_{y,emb}$**	**$K_{z,emb}$**	**$K_{xx,emb}$**	**$K_{yy,emb}$**	**$K_{zz,emb}$**
		kips/inch	**kips/inch**	**kips/inch**	**kips/inch**	**kips/inch**	**kips/inch**
Pier 2	ESU 2A - Native Sand and Gravel	5.745E+03	6.097E+03	5.089E+03	6.012E+07	1.406E+08	1.683E+08

Appendix D– Footing Spring Stiffness at East Abutment (Pier 3)



Appendix E – Settlements

Sent on 2021/10/22

WSDOT/I-405 Renton to Bellevue
Bridge 17.7 - Staged Settlement Analysis
60180-366-01; Task 200

		Pier 1		
		Load (kips)	Pressure at Bottom of Footing	Δ Pressure per Stage (ksf)
1	Remove Existing Hilfiker Wall (27-feet)	-	0.0	0.00
2	Extension+DC1+PT (4' Thick Footing)	922	1.13	1.13
2.5	Extension+DC1+PT (4' Thick Footing), & New Hilfiker Wall Over Footing (23-foot Fill)	1807	2.22	2.22
3	New Hilfiker Wall, & Extension+DC1+PT+Pier 1 Removal	2110	2.59	1.46
4	New Hilfiker Wall, & Extension+DC1+PT+Pier 1 Removal+DC2	2198	2.70	0.11
	Footing Length (ft)		37.0	
	Footing Width (ft)		22.0	
	Footing Size (ft^2)		814	
	Final Total Pressure Increase (ksf)		4.9	

	Pier 1				
	Stage 1	Stage 2	Stage 2.5	Stage 3	Case 4
q_o (ksf)	0.00	1.13	2.22	1.46	0.11
v	0.36	0.36	0.36	0.36	0.36
A (ft^2)	814	814	814	814	814
E_s (ksf)	888	888	888	888	888
L/B	1.7	1.7	1.7	1.7	1.7
β_z	1.094	1.094	1.094	1.094	1.094
S_o(in.) per Stage	0.00	0.35	0.68	0.45	0.03
Cummulative S_o (in.)	0.00	0.35	1.03	1.48	1.51

		Pier 2		
		Load (kips)	Pressure (ksf)	Δ Pressure (ksf)
0	Prior to Existing Bridge Construction	0	0.00	0.00
1	Existing	1513	3.86	3.86
2	Extension+DC1+PT	1494	3.81	-0.05
3	Extension+DC1+PT+Pier 1 Removal	1556	3.97	0.16
4	Extension+DC1+PT+Pier 1 Removal+DC2	1560	3.98	0.01
	Footing Length (ft)		28.0	
	Footing Width (ft)		14.0	
	Footing Size (ft^2)		392	
	Final Total Pressure Increase (ksf)		3.98	

	Pier 2				
	Stage 0	Stage 1	Stage 2	Stage 3	Stage 4
q_o (ksf)	0.00	3.86	-0.05	0.16	0.01
v	0.40	0.40	0.40	0.40	0.40
A (ft^2)	392	392	392	392	392
E_s (ksf)	1000	1000	1000	1000	1000
L/B	2.0	2.0	2.0	2.0	2.0
β_z	1.10	1.10	1.10	1.10	1.10
S_o(in.) per Stage	0.00	0.70	-0.01	0.03	0.00
Cummulative S_o (in.)	0.00	0.70	0.69	0.72	0.72

Variable Key	
Variables	Description
q_o (ksf)	Load intensity at bottom of foundation
v	Poisson's Ratio
A (ft^2)	Area of bottom of foundation
E_s (ksf)	Young's Modulus of Subgrade soil
β_z	Shape factor
S_o (in.)	Predicted cumulative elastic settlement

Appendix E – Settlements

Sent on 2021/12/03

WSDOT/I-405 Renton to Bellevue
Bridge 17.7 - Staged Settlement Analysis
00180-366-01; Task 200

		Pier 1		
		Cumulative Load (kips)	Cumulative Pressure (ksf)	Δ Pressure per Stage (ksf)
1	Existing (Existing Hilfiker Wall, 27-feet)	-	3.2	0.00
2	Extension+DC1+PT	1127	1.18	1.18
2.5	Build New Hilfiker Wall (27-feet Tall)	2522	2.64	1.46
3	Extension+DC1+PT+Pier 1 Removal	2979	3.12	0.48
4	Extension+DC1+PT+Pier 1 Removal+DC2	3065	3.21	0.09
	Footing Length (ft)	38.7		
	Footing Width (ft)	24.7		
	Footing Size (ft^2)	956		
	Final Total Pressure Increase (ksf)	3.2		

	Pier 1				
	Stage 1	Stage 2	Stage 2.5	Stage 3	Stage 4
q_o (ksf)	0.00	1.18	1.46	0.48	0.09
ν	0.25	0.25	0.25	0.25	0.25
A (ft^2)	956	956	956	956	956
E_s (ksf)	1720	1720	1720	1720	1720
L/B	1.6	1.6	1.6	1.6	1.6
β_z	1.091	1.091	1.091	1.091	1.091
S_e(in.) per Stage	0.00	0.22	0.27	0.09	0.02
Cumulative S_e (in.)	0.00	0.22	0.49	0.58	0.59

		Pier 2		
		Cumulative Load (kips)	Cumulative Pressure (ksf)	Δ Pressure per Stage (ksf)
1	Existing	1592	4.06	0.00
2	Extension+DC1+PT	1583	4.04	-0.02
3	Extension+DCI+PT+Pier 1 Removal	1661	4.24	0.20
4	Extension+DCI+PT+Pier 1 Removal+DC2	1664	4.24	0.01
	Footing Length (ft)	28.0		
	Footing Width (ft)	14.0		
	Footing Size (ft^2)	392		
	Final Total Pressure Increase (ksf)	0.18		

	Pier 2			
	Stage 1	Stage 2	Stage 3	Stage 4
q_o (ksf)	0.00	-0.02	0.20	0.01
ν	0.35	0.35	0.35	0.35
A (ft^2)	392	392	392	392
E_s (ksf)	979	979	979	979
L/B	2.0	2.0	2.0	2.0
β_z	1.10	1.10	1.10	1.10
S_e(in.) per Stage	0.00	0.00	0.04	0.00
Cumulative S_e (in.)	0.00	0.00	0.03	0.04

Variable Key	
Variables	Description
q_o (ksf)	Load intensity at bottom of foundation
ν	Poisson's Ratio
A (ft^2)	Area of bottom of foundation
E_s (ksf)	Young's Modulus of Subgrade soil
β_z	Shape factor
S_e (in.)	Predicted cumulative elastic settlement

COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I2

Title: Girders

CALC BOOK COMPLETION DATE: **2021 Nov 21**

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I2-0	-	I2-0	Calc Register (this sheet)	N/A	N/A	
2	I2-1	-	I2-14	1 Shear and Flexure Demand and Capacity Ratios along Bridge	LDLI	CUPG	
4	I2-15	-	I2-51	2 Service Limit State Check at Critical Sections	LDLI	CUPG	
5	I2-52	-	I2-55	3 Existing and New Structure Interface Check	LDLI	CUPG	
6	I2-56	-	I2-72	4 Web Interface Check for Critical Sections	LDLI	CUPG	
7	I2-73	-	I2-113	5 Access Opening Check	LDLI	CUPG	
8	I2-74	-	I2-85	5.1 Permanent Bottom Slab Access Opening	LDLI	CUPG	
9	I2-86	-	I2-113	5.2 Temporary Deck Access Opening	LDLI	CUPG	
10	I2-114	-	I2-114	6 Development Length	LDLI	FCCA	
12	I2-115	-	I2-131	7 Tendon Stresses	LDLI	BOHU	

Shear and Flexural Capacity Check

Description

The following calculations check the shear and flexural capacity along the bridge span. The D/C ratios are calculated at 98 stations with maximum spacing of 2.96 ft and minimum spacing of 0.41 ft. Typical cross-section was used in the calculations to be conservative. An independent check was conducted to study the flexural and shear capacities at the diaphragms. Between the diaphragms and supports, the maximum D/C for flexure is 0.6 and the maximum D/C for shear is 0.6.

COWI

Project I-405 Bridge 17.7
Title Shear Capacity Calculation Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 10/27/2021
Date 10/27/2021

Description:
This sheet summarized the inputs of the shear capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons along the bridge superstructure at each cross-sections along the superstructure. The torsion, moment, and axial force effects for the corresponding maximum or minimum shear forces are obtained from csibridge load combination 'SUP_STRENGTH ENVELOPE', which is the governing demands in all strength limit state combinations. The mild reinforcement areas in the extended portion are assumed to match with the existing.
Notation Definition:

$V_{2\ max}$	Maximum Vertical Shear Force Demand at the Cross-section	V_p	Tendon Shear Force	d_{s_bottom}	Vertical Distance from Centroid of Reinforcements in Bottom Slab to NA
$V_{2\ min}$	Minimum Vertical Shear Force Demand at the Cross-section	y_{tendon}	Vertical Distance from Top of Superstructure to Tendon	A_{s_bottom}	Area of Reinforcement in Bottom Slab
T	Torsion Demand	s_{ext}	Trans. Reinf. Spacing in Exterior Webs	P_e	Tendon Force in Service
$M_{3\ max}$	Corresponding Moment about Longitudinal Axis when V2 is max	s_{int}	Trans. Reinf. Spacing in Interior Webs		
$M_{3\ min}$	Corresponding Moment about Longitudinal Axis when V2 is min	d_{s_top}	Vertical Distance from Centroid of Reinforcements in Top Slab to NA		
P	Axial Force Demand	A_{s_top}	Area of Reinforcement in Top Slab		

Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	V_p (kip)	y_{tendon} (ft)	s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
0.00	-353.00	-772.13	-181.36	-908.58	-1995.87	-3334.89	-968.84	-1048.11	42.96	1.70	30.00	30.00	0.61	0.181	3.979	0.244	254.000
1.73	-328.38	-730.22	-206.63	-996.40	-1300.15	-2105.75	-967.63	-970.55	42.96	1.78	6.00	8.00	0.615	0.181	3.979	0.244	254.000
3.45	-304.01	-688.85	-232.11	-1079.61	-349.59	-1258.22	-967.06	-969.98	42.96	1.85	6.00	8.00	0.615	0.181	3.979	0.244	254.000
5.18	-279.90	-647.95	-257.78	-1157.43	531.74	-467.21	-966.46	-969.39	42.96	1.92	6.00	8.00	0.615	0.181	3.979	0.244	254.000
6.91	-256.05	-607.57	-283.60	-1227.99	1345.43	267.76	-965.83	-968.76	42.96	1.99	6.00	8.00	0.615	0.181	3.979	0.244	254.000
8.64	-232.46	-567.72	-309.54	-1288.77	2093.20	947.15	-965.16	-968.09	42.96	2.07	6.00	8.00	0.615	0.181	3.979	0.244	254.000
10.36	-208.44	-529.30	-562.69	-1341.18	2785.26	2669.51	-964.59	-967.51	42.96	2.14	6.00	8.00	0.615	0.181	3.979	0.244	254.000
12.09	-181.14	-488.92	-334.70	-934.92	3484.04	3546.00	-984.64	-1064.99	42.96	2.21	8.00	13.00	0.615	0.181	3.979	0.244	254.000
14.02	-153.26	-449.51	-371.79	-985.23	4093.40	4293.66	-985.04	-1065.39	42.96	2.30	8.00	13.00	0.615	0.181	3.979	0.706	254.000
15.95	-125.05	-402.52	-411.96	-1042.01	4524.31	4997.07	-983.41	-1063.76	42.96	2.38	8.00	13.00	0.615	0.181	3.979	0.706	254.000
17.88	-97.15	-362.84	-448.85	-1072.74	4971.80	5611.66	-982.68	-1063.04	42.96	2.46	8.00	13.00	0.615	0.181	3.979	0.706	254.000
19.82	-69.34	-323.71	-483.76	-1091.92	5350.02	6152.97	-981.89	-1062.25	42.96	2.54	13.00	15.00	0.615	0.181	3.979	0.706	254.000
21.75	-41.78	-285.07	-518.54	-1106.24	5658.42	6614.55	-981.11	-1061.46	42.96	2.62	13.00	15.00	0.615	0.181	3.979	0.706	254.000
23.68	-14.40	-246.99	-552.21	-1119.42	5899.52	6999.59	-980.30	-1060.65	42.96	2.70	15.00	15.00	0.615	0.181	3.979	0.706	254.000
25.61	13.45	-209.30	-562.46	-1127.21	6071.57	7340.22	-979.42	-982.43	42.96	2.79	15.00	15.00	0.615	0.181	3.979	0.706	254.000
27.54	41.78	-172.10	-602.23	-1133.85	6177.51	7617.60	-978.52	-981.53	42.96	2.87	15.00	15.00	0.615	0.181	3.979	0.706	254.000
29.47	70.34	-139.43	-641.45	-1149.77	6329.20	7821.89	-978.15	-981.16	42.96	2.95	15.00	15.00	0.615	0.181	3.979	0.706	254.000
31.40	99.06	-99.01	-685.31	-1125.60	6193.22	7970.74	-976.64	-979.65	42.96	3.03	15.00	15.00	0.615	0.350	3.979	0.706	254.000
33.34	132.52	-75.09	-1077.08	-793.81	8601.19	5695.52	-974.85	-977.86	42.96	3.11	15.00	15.00	0.615	0.350	3.979	0.706	254.000
35.27	168.65	-47.85	-1127.04	-780.90	8446.96	5686.41	-973.90	-976.91	42.96	3.19	15.00	15.00	0.615	0.350	3.979	0.706	254.000
37.20	204.48	-21.02	-1177.09	-768.61	8218.18	5613.42	-972.93	-975.95	42.96	3.28	15.00	15.00	0.615	0.350	3.979	0.706	254.000
39.13	239.47	10.62	125.49	161.60	7990.56	5297.52	-989.95	-1070.13	61.25	3.25	15.00	15.00	0.615	0.350	3.979	0.706	254.000
40.00	292.21	46.15	170.50	68.87	7683.92	4925.30	-1053.41	-1061.24	-61.25	3.20	15.00	15.00	0.615	0.350	3.979	0.706	262.000
41.92	296.21	49.03	167.38	66.63	7597.46	4911.31	-1053.41	-1061.24	-61.25	3.09	15.00	15.00	0.615	0.350	3.979	0.706	262.000
43.85	330.66	73.29	149.16	51.49	7214.61	4563.62	-1055.69	-1063.52	-61.25	2.98	15.00	15.00	0.615	0.350	3.979	0.706	262.000
45.77	364.05	97.32	130.79	31.48	6732.61	4170.46	-1057.76	-1065.59	-61.25	2.86	13.00	15.00	0.615	0.350	3.979	0.706	262.000
47.70	397.37	121.11	112.16	11.46	6177.28	3734.46	-1059.77	-1067.60	-61.25	2.75	13.00	15.00	0.615	0.350	3.979	0.706	262.000
49.62	430.76	144.29	93.14	-7.64	5552.14	3272.16	-1061.77	-1069.60	-61.25	2.64	8.00	13.00	0.615	0.350	3.979	0.706	262.000
51.54	464.14	166.94	74.09	-26.18	4856.08	2783.73	-1063.73	-1071.57	-61.25	2.53	8.00	13.00	0.615	0.350	3.979	0.706	262.000
53.47	497.54	189.44	54.58	-44.62	4090.43	2252.64	-1065.68	-1073.51	-61.25	2.41	8.00	13.00	0.615	0.350	3.979	0.706	262.000
55.39	530.81	211.77	32.05	-63.12	3252.80	1683.76	-1067.60	-1075.44	-61.25	2.30	8.00	13.00	0.615	0.350	3.979	0.706	262.000
57.32	564.03	233.90	9.71	-81.28	2345.82	1071.78	-1069.49	-1077.32	-61.25	2.19	8.00	8.00	0.615	0.350	3.979	0.706	262.000
59.24	597.20	255.88	-13.39	-99.33	1369.92	420.14	-1071.34	-1079.18	-61.25	2.08	8.00	8.00	0.615	0.350	3.979	0.706	262.000
61.16	630.28	276.95	-36.99	-162.13	325.77	-1295.89	-1036.58	-1044.42	-61.25	1.96	6.00	8.00	0.615	0.567	3.979	0.706	262.000
63.09	663.31	294.82	-60.59	-186.67	-785.42	-1913.15	-1036.58	-1044.42	-61.25	1.85	6.00	8.00	0.615	0.567	3.979	0.706	262.000
65.01	696.26	312.69	-84.21	-211.22	-1839.99	-2568.64	-1036.58	-1044.42	-61.25	1.74	6.00	8.00	0.615	0.567	3.979	0.390	262.000
66.93	729.14	330.56	-107.82	-235.76	-2939.63	-3262.36	-1036.58	-1044.42	-61.25	1.63	6.00	8.00	0.615	0.567	3.979	0.390	262.000
68.86	761.91	348.43	-132.26	-260.31	-4088.17	-3994.31	-1036.58	-1044.42	-61.25	1.51	6.00	8.00	0.615	0.567	3.979	0.390	262.000
70.78	794.57	366.30	-156.80	-284.86	-5284.65	-4764.49	-1036.58	-1044.42	-61.25	1.40	6.00	8.00	0.615	0.567	3.979	0.390	262.000
72.71	859.61	402.04	-205.79	-333.95	-7740.07	-6419.52	-1036.58	-1044.42	-61.25	1.29	6.00	8.00	0.615	0.567	3.979	0.390	262.000
74.63	-417.75	-886.79	363.33	30.97	-6677.40	-10584.90	-574.90	-1044.42	-61.25	1.18	6.00	8.00	0.615	0.567	3.979	0.390	262.000
76.63	827.13	384.17	-181.32	-309.40	-6489.37	-5572.89	-1036.58	-578.90	79.37	1.06	30.00	30.00	0.615	0.567	3.979	0.390	273.000
78.63	-399.50	-848.79	339.53	-24.67	-5860.03	-9115.92	-575.05	-579.05	76.68	1.21	6.00	8.00	0.615	0.567	4.000	0.329	273.000

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7
Title Shear Capacity Calculation Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 10/27/2021
Date 10/27/2021

Description:
This sheet summarized the inputs of the shear capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons along the bridge superstructure at each cross-sections along the superstructure. The torsion, moment, and axial force effects for the corresponding maximum or minimum shear forces are obtained from csibridge load combination 'SUP_STRENGTH ENVELOPE', which is the governing demands in all strength limit state combinations. The mild reinforcement areas in the extended portion are assumed to match with the existing.
Notation Definition:

$V_{2\ max}$	Maximum Vertical Shear Force Demand at the Cross-section	V_p	Tendon Shear Force	d_{s_bottom}	Vertical Distance from Centroid of Reinforcements in Bottom Slab to NA
$V_{2\ min}$	Minimum Vertical Shear Force Demand at the Cross-section	y_{tendon}	Vertical Distance from Top of Superstructure to Tendon	A_{s_bottom}	Area of Reinforcement in Bottom Slab
T	Torsion Demand	s_{ext}	Trans. Reinf. Spacing in Exterior Webs	P_e	Tendon Force in Service
$M_{3\ max}$	Corresponding Moment about Longitudinal Axis when V2 is max	s_{int}	Trans. Reinf. Spacing in Interior Webs		
$M_{3\ min}$	Corresponding Moment about Longitudinal Axis when V2 is min	d_{s_top}	Vertical Distance from Centroid of Reinforcements in Top Slab to NA		
P	Axial Force Demand	A_{s_top}	Area of Reinforcement in Top Slab		

Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	V_p (kip)	y_{tendon} (ft)	s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
80.54	-381.83	-816.48	318.70	-59.55	-5102.87	-7622.54	-581.40	-585.40	76.68	1.34	6.00	8.00	0.615	0.567	4.000	0.329	273.000
82.44	-364.15	-784.01	297.92	-92.43	-4379.97	-6192.41	-581.65	-585.65	34.12	1.47	6.00	8.00	0.615	0.567	4.000	0.329	273.000
84.35	-346.48	-751.43	277.11	-121.59	-3691.33	-4825.36	-581.91	-585.91	119.24	1.53	8.00	8.00	0.615	0.567	4.000	0.329	273.000
86.25	-328.81	-718.74	256.31	-149.47	-3036.97	-3522.55	-582.19	-586.19	76.68	1.74	8.00	8.00	0.615	0.567	4.000	0.329	273.000
88.16	-311.14	-685.95	235.50	-176.03	-2416.87	-2284.91	-582.48	-586.48	76.68	1.88	8.00	13.00	0.615	0.567	4.000	0.329	273.000
90.07	-293.46	-653.07	214.70	-201.54	-1831.04	-1113.79	-587.07	-591.07	76.68	2.01	8.00	13.00	0.615	0.567	4.000	0.329	273.000
91.97	-271.98	-620.13	61.70	-225.86	-189.82	-9.58	-588.65	-592.65	76.68	2.14	8.00	13.00	0.615	0.567	4.000	0.329	273.000
93.88	-250.08	-587.11	40.58	-249.29	500.17	1026.11	-590.23	-594.23	76.68	2.28	13.00	13.00	0.615	0.333	4.000	0.329	273.000
95.78	-228.06	-557.96	20.02	-266.86	1146.35	2070.78	-591.65	-595.65	76.68	2.41	13.00	15.00	0.615	0.333	4.000	0.329	273.000
97.69	-207.13	-524.38	-2.91	-287.54	1768.45	2897.95	-593.15	-597.15	76.68	2.54	13.00	15.00	0.615	0.333	4.000	0.395	273.000
98.89	-192.74	-505.01	-4.43	-274.91	2191.66	3424.91	-594.39	-598.39	76.68	2.63	13.00	15.00	0.615	0.333	4.000	0.395	273.000
100.09	-179.34	-482.93	4.51	-252.55	2547.20	3865.25	-595.23	-599.23	76.68	2.71	16.00	15.00	0.615	0.333	4.000	0.395	273.000
101.29	-164.37	-462.62	14.90	-224.88	2914.17	4356.01	-596.43	-600.44	76.68	2.80	16.00	15.00	0.615	0.333	4.000	0.395	273.000
102.49	-150.54	-439.68	33.07	-189.42	3205.63	4768.03	-597.31	-601.31	76.68	2.88	16.00	15.00	0.615	0.333	4.000	0.395	273.000
104.06	-130.66	-415.58	55.86	-143.41	3675.56	5341.36	-600.19	-604.19	76.68	2.99	16.00	15.00	0.615	0.333	4.000	0.395	273.000
105.63	-86.93	-349.92	57.96	-16.75	3968.20	5859.93	-605.31	-610.12	-156.62	3.10	16.00	15.00	0.615	0.333	4.000	0.395	273.000
107.08	-68.04	-323.66	94.14	40.85	4237.76	6266.79	-606.29	-611.11	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
108.53	-39.89	-286.82	139.67	110.05	4523.72	6629.18	-607.03	-611.84	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
110.31	-16.20	-251.05	194.51	185.74	4755.24	6912.04	-608.22	-613.04	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
112.10	7.49	-220.89	242.81	250.31	4976.50	7215.04	-609.15	-613.98	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
113.30	23.59	-199.14	284.17	303.38	5100.92	7342.64	-609.71	-614.53	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
114.50	39.77	-177.40	326.81	356.30	5202.72	7438.48	-610.22	-615.04	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
115.70	56.02	-155.67	370.39	408.70	5281.33	7502.55	-610.67	-615.50	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
116.90	72.37	-133.95	414.62	460.22	5336.72	7534.83	-611.07	-615.90	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
118.10	90.46	-113.92	662.60	307.21	7433.46	5470.07	-610.27	-615.09	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
119.30	111.69	-96.96	723.24	339.99	7434.01	5445.83	-610.54	-615.37	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
120.50	132.99	-80.02	783.61	370.91	7403.96	5396.07	-610.76	-615.58	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
121.71	154.37	-63.09	843.30	399.58	7343.14	5320.85	-610.90	-615.73	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
122.91	175.83	-46.26	881.47	428.79	7245.01	5232.65	-607.62	-612.44	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
124.11	197.36	-29.81	937.94	455.21	7121.96	5121.49	-607.33	-612.16	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
125.31	218.98	-13.38	992.82	478.92	6967.09	4986.22	-607.11	-611.93	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
126.51	240.66	2.45	1045.34	497.21	6780.39	4873.27	-606.79	-611.60	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
127.71	262.41	18.06	1095.24	520.28	6561.57	4718.71	-606.44	-611.25	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
128.91	284.22	33.63	1142.08	540.56	6310.38	4542.03	-605.96	-610.77	0.00	3.31	16.00	15.00	0.615	0.333	4.000	0.395	273.000
130.11	306.10	49.19	1185.48	557.93	6026.65	4343.33	-605.44	-610.25	0.00	3.31	16.00	13.00	0.615	0.333	4.000	0.329	273.000
131.31	328.03	64.71	1225.02	572.03	5710.11	4122.75	-604.81	-609.62	0.00	3.31	16.00	13.00	0.615	0.333	4.000	0.329	273.000
132.51	350.03	80.21	1260.31	582.72	5360.66	3880.45	-604.14	-608.94	0.00	3.31	16.00	13.00	0.615	0.333	4.000	0.329	273.000
133.71	372.09	95.67	1290.91	589.67	4978.13	3616.57	-603.33	-608.12	0.00	3.31	13.00	13.00	0.615	0.333	4.000	0.329	273.000
134.91	394.20	111.11	1316.40	592.75	4562.32	3331.33	-602.49	-607.28	0.00	3.31	8.00	13.00	0.615	0.333	4.000	0.329	273.000
136.11	416.36	126.51	1336.35	591.57	4113.18	3024.71	-601.51	-606.30	0.00	3.31	8.00	13.00	0.615	0.333	4.000	0.329	273.000
137.32	438.58	141.88	1350.36	586.22	3630.52	2697.10	-600.49	-605.27	0.00	3.31	8.00	13.00	0.615	0.333	4.000	0.329	273.000
138.52	461.24	159.03	1350.46	554.16	3123.89	2261.86	-600.04	-604.81	0.00	3.31	8.00	13.00	0.615	0.333	4.000	0.329	273.000
138.93	467.07	163.23	1368.11	567.48	2911.24	2187.43	-598.48	-603.25	94.90	3.31	8.00	13.00	0.615	0.333	4.000	0.329	273.000
140.43	517.20	196.00	1270.24	596.41	2395.94	1611.70	-673.17	-684.75	-146.76	3.44	30.00	30.00	0.615	0.333	4.000	0.329	273.000

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7
Title Shear Capacity Calculation Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 10/27/2021
Date 10/27/2021

Description:
This sheet summarized the inputs of the shear capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons along the bridge superstructure at each cross-sections along the superstructure. The torsion, moment, and axial force effects for the corresponding maximum or minimum shear forces are obtained from csibridge load combination 'SUP_STRENGTH ENVELOPE', which is the governing demands in all strength limit state combinations. The mild reinforcement areas in the extended portion are assumed to match with the existing.

Notation Definition:

$V_{2\ max}$	Maximum Vertical Shear Force Demand at the Cross-section	V_p	Tendon Shear Force	d_{s_bot}tom	Vertical Distance from Centroid of Reinforcements in Bottom Slab to NA
$V_{2\ min}$	Minimum Vertical Shear Force Demand at the Cross-section	y_{tendon}	Vertical Distance from Top of Superstructure to Tendon	A_{s_bottom}	Area of Reinforcement in Bottom Slab
T	Torsion Demand	s_{ext}	Trans. Reinf. Spacing in Exterior Webs	P_e	Tendon Force in Service
$M_{3\ max}$	Corresponding Moment about Longitudinal Axis when V2 is max	s_{int}	Trans. Reinf. Spacing in Interior Webs		
$M_{3\ min}$	Corresponding Moment about Longitudinal Axis when V2 is min	d_{s_top}	Vertical Distance from Centroid of Reinforcements in Top Slab to NA		
P	Axial Force Demand	A_{s_top}	Area of Reinforcement in Top Slab		

Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	V_p (kip)	y_{tendon} (ft)	s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
141.43	537.83	208.61	1257.59	336.59	1870.78	523.13	-672.38	-683.95	-134.40	3.31	30.00	30.00	0.615	0.333	4.000	0.329	250.000
142.37	552.82	217.81	1203.33	294.82	1496.88	358.26	-671.73	-683.28	-134.40	3.18	8.00	13.00	0.615	0.333	4.000	0.329	250.000
143.32	567.95	227.04	1148.72	252.47	1110.78	186.46	-670.71	-682.25	-134.40	3.05	8.00	13.00	0.615	0.333	4.000	0.329	250.000
144.52	587.02	238.67	1073.82	197.60	604.93	-35.26	-669.71	-681.23	-134.40	2.89	8.00	13.00	0.615	0.333	4.000	0.329	250.000
145.72	606.04	250.27	993.22	141.47	84.19	-263.06	-668.76	-680.25	-134.40	2.73	8.00	13.00	0.615	0.333	4.000	0.329	250.000
146.92	624.83	261.88	909.43	83.03	-457.27	-499.55	-667.34	-678.81	-134.40	2.57	8.00	13.00	0.615	0.333	4.000	0.329	250.000
148.12	644.15	273.42	816.92	24.08	-1010.46	-740.66	-666.48	-677.92	-134.40	2.41	8.00	13.00	0.615	0.333	4.000	0.329	250.000
149.32	662.52	284.99	726.47	-37.11	-1584.37	-992.03	-665.66	-677.09	-134.40	2.24	8.00	13.00	0.615	0.333	4.000	0.329	250.000
150.52	680.42	294.77	652.51	-70.65	-1982.11	-1451.45	-664.76	-676.16	-134.40	2.08	8.00	13.00	0.615	0.333	4.000	0.329	250.000
151.72	695.74	303.09	610.87	-80.13	-2396.48	-1933.86	-664.41	-675.79	-134.40	1.92	8.00	13.00	0.615	0.333	4.000	0.329	250.000
152.93	709.58	310.63	590.01	-75.39	-2817.59	-2428.05	-664.78	-676.16	-134.40	1.76	30.00	30.00	0.615	0.333	4.000	0.329	250.000

COWI

Project I-405 Bridge 17.7 **Project No** A207833
Title Shear Capacity Check (Sample Calcs)
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:This spreadsheet calculates the shear d/c ratio of a box girder at a single cross-section. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'shear capacity input' using vba for all stations.

Station	V_{umax}(kip)	V_{umin}(kip)	T_u for V_{umax} (kip.ft)	T_u for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P_u for V_{umax} (kip)	P_u for V_{umin} (kip)	V_p (kip)	y_{tendon} (ft)
152.93	709.58	310.63	590.01	-75.39	-2817.59	-2428.05	-664.78	-676.16	-134.40	1.76

s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
30.00	30.00	0.615	0.333	4.000	0.329	250.000

Section Location West Abutment

1 Material Properties

f'_c (ksi)	5	*concrete compressive strength*
f_{pu} (ksi)	270	*specified tendon tensile strength*
f_{p0} (ksi)	179.582	*assumes equal to fps* *(5.7.4.3.2)*
f_{ps} (ksi)	179.582	*average stress in presstressing steel* *(5.6.3.1)*
f_y (ksi)	60	*reinforcement yield strength*
E_s (ksi)	29000	*reinforcement modulus of elasticity*
E_p (ksi)	29000	*tendon modulus of elasticity*
E_c (ksi)	4903	*concrete modulus of elasticity*

2 Structural Properties

Girder

b_t (ft)	36.073	*deck width*
b_b (ft)	23.873	*bottom slab width*
h_{ft} (ft)	0.708	*deck thickness*
h_{fb} (ft)	0.500	*bottom slab thickness*
b_w (ft)	3.363	*total web thickness*
h (ft)	4.25	*total height*
A_0 (ft^2)	110.6	*area enclosed by the shear flow path, measured using bluebeam* *(5.7.3.4.2)*
h_{fillet} (ft)	0.5	*height of fillet*
h_w (ft)	2.542	*height of web*
h_{tw}(ft)	0.333	*spacing of web longitudinal reinforcements*
A_{ct}_negtive (sf)	30.316	
A_{ct}_positive (sf)	17.402	

Tendon

A_{ps} (ft^2)	0.00151	*strand area*
N_s	7	*amount of strand per tendon*
N_t	4	*amount of tendon*
$A_{ps.total}$(ft^2)	0.0422	*total tendon area*
d_{pmax} (ft)	2.490	*distance from extreme compression fiber to the centroid of the tendons for max moment*
d_{pmin} (ft)	2.490	*distance from extreme compression fiber to the centroid of the tendons for min moment*

Mild Reinforcement

C_t (ft)	0.208	*concrete cover*
C_b (ft)	0.083	*concrete cover*
$D_{N.5}$ (ft)	0.052	*diameter of #5*
$A_{N.5}$ (sf)	0.00215	*area of #5*
$N_{N.5}$	8	*amount of #5*
A_s for $V_{u\,max}$(sf)	0.333	*area of moment resisting steel when shear is at maximum*
A_s for $V_{u\,min}$ (sf)	0.333	*area of moment resisting steel when shear is at minimum*
d_s for V_{umax} (ft)	3.635	*vertical distance from extreme compression fiber to the centroid of the mild tensile reinforcement in slab*
d_s for V_{umin} (ft)	3.635	*vertical distance from extreme compression fiber to the centroid of the mild tensile reinforcement in slab*
A_v (sf)	0.0172	*area of shear reinforcement*
α (deg)	90.0	*angle of inclination of transverse reinforcement to longitudinal axes*

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Project I-405 Bridge 17.7 **Project No** A207833
Title Shear Capacity Check (Sample Calcs)
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:This spreadsheet calculates the shear d/c ratio of a box girder at a single cross-section. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'shear capacity input' using vba for all stations.

Station	V_{umax}(kip)	V_{umin}(kip)	T_u for V_{umax} (kip.ft)	T_u for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P_u for V_{umax} (kip)	P_u for V_{umin} (kip)	V_p (kip)	y_{tendon} (ft)
152.93	709.58	310.63	590.01	-75.39	-2817.59	-2428.05	-664.78	-676.16	-134.40	1.76

s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
30.00	30.00	0.615	0.333	4.000	0.329	250.000

3 Effective Shear Depth

if the station is in east span:	
d_e for $V_{u\ max}$ (ft)	3.321
d_e for $V_{u\ min}$ (ft)	3.321
d_v for V_{umax} (ft)	3.060
d_v for V_{umin} (ft)	3.060

depth of center of gravity of steel (5.7.2.8)
depth of center of gravity of steel (5.7.2.8)
effective shear depth (5.7.2.8)
effective shear depth (5.7.2.8)

4 Minimum Transverse Reinforcement Check

λ	1
A_{vmin} (in^2)	1.426
A_v (in^2)	2.48
Min. Trans. Rein.?	yes

concrete density modification factor (5.4.2.8)
minimum transverse reinforcement area within distance s, where s is spacing of reinforcement
transverse reinforcement area within distance s

5 Maximum Spacing of Transverse Reinforcement

	for V_{max}	for V_{min}		
Resistance factor	0.9	0.9	*resistance factor*	(5.5.4.2)
v_u (ksi)	0.441	0.142	*shear stress on concrete*	(5.7.2.8)
s_{max} (in)	24.0	24.0	*maximum permitted spacing*	(5.7.2.6)
check s_{max}	no	no	*Does this check pass for both Vumax and Vumin?* no	

5 Nominal Shear Resistance

	for V_{max}	for V_{min}		
s_x (in)	11.00	11.00	*cracking spacing parameter*	(5.7.3.4.2)
a_g (in)	0.75	0.75	*aggregate size*	(5.7.3.4.2)
S_{xe}(in)	11.00	11.00	*cracking spacing parameter influenced by aggregate size*	(5.7.3.4.2)
V_{eff} (kip)	719	312	*factored shear considering torsion effect*	(5.7.3.4.2)
ε_s_positive	0.00005	-0.00029	*net longitudinal tensile strain*	(5.7.3.4.2)
ε_s_negative	3.58E-06	-1.99E-05	*net longitudinal tensile strain*	(5.7.3.4.2)
ε_s	0.000052	-0.00002	*net longitudinal tensile strain*	(5.7.3.4.2)
β	4.619	4.873	*shear resistance factor*	(5.7.3.4.2)
V_c (kip)	483.654	510.286	*concrete shear resistance*	(5.7.3.4.2)
θ (deg)	29.183	27.977	*angle of inclination of diagonal compressive stress*	(5.7.3.4)
V_s (kip)	326.109	342.868	*shear resistance*	

6 Longitudinal Reinforcement Check

$$A_{ps}f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v\phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

(5.7.3.5)

	for V_{max}	for V_{min}	
LHS of eqn. (kip	3971.14	3971.14	
RHS of eqn. (kip	1532.86	580.00	
long. Reinf. Chec	yes	yes	*Does this check pass for both Vumax and Vumin?* yes

6 Summary

	for V_{max}	for V_{min}	
V_n (kip)	944.2	987.6	
V_u (kip)	709.581	310.626	
D/C	0.835	0.349	*Governed D/C 0.8350504*

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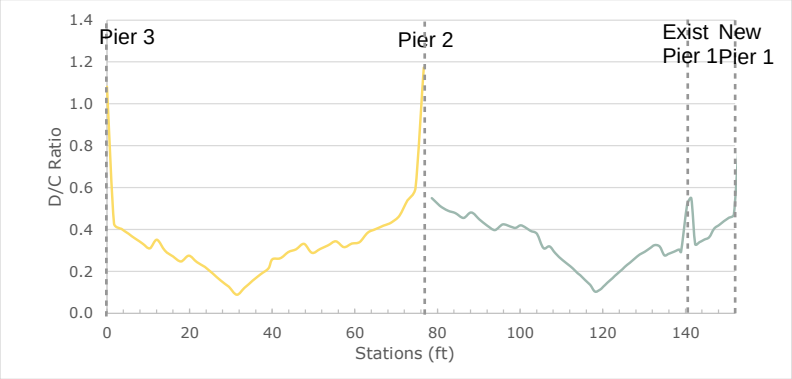
Project I-405 Bridge 17.7 **Project No** A207833
Title Shear Capacity D/C
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:
This sheet summarized the inputs of the shear capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons along the bridge superstructure at each cross-sections along the superstructure. D/C ratios at the locations of diaphgrams are overestimated because the 'shear capacity check' sheet assumes the superstructure is box girder with 3 cells. An independent shear check will be conducted to check for the D/C at the diaphgrams.



Station (ft)	Min Reinf.?	Max Spacing	Long. Reinf.?	Governed D/C
0.00	yes	no	yes	1.1
1.73	yes	yes	yes	0.4
3.45	yes	yes	yes	0.4
5.18	yes	yes	yes	0.4
6.91	yes	yes	yes	0.4
8.64	yes	yes	yes	0.3
10.36	yes	yes	yes	0.3
12.09	yes	yes	yes	0.4
14.02	yes	yes	yes	0.3
15.95	yes	yes	yes	0.3
17.88	yes	yes	yes	0.2
19.82	yes	yes	yes	0.3
21.75	yes	yes	yes	0.2
23.68	yes	yes	yes	0.2
25.61	yes	yes	yes	0.2
27.54	yes	yes	yes	0.2
29.47	yes	yes	yes	0.1
31.40	yes	yes	yes	0.1
33.34	yes	yes	yes	0.1
35.27	yes	yes	yes	0.2
37.20	yes	yes	yes	0.2
39.13	yes	yes	yes	0.2
40.00	yes	yes	yes	0.3
41.92	yes	yes	yes	0.3
43.85	yes	yes	yes	0.3
45.77	yes	yes	yes	0.3
47.70	yes	yes	yes	0.3
49.62	yes	yes	yes	0.3
51.54	yes	yes	yes	0.3
53.47	yes	yes	yes	0.3
55.39	yes	yes	yes	0.3
57.32	yes	yes	yes	0.3
59.24	yes	yes	yes	0.3
61.16	yes	yes	yes	0.3
63.09	yes	yes	yes	0.4
65.01	yes	yes	yes	0.4
66.93	yes	yes	yes	0.4
68.86	yes	yes	yes	0.4
70.78	yes	yes	yes	0.5
72.71	yes	yes	yes	0.5
74.63	yes	yes	yes	0.6
76.63	yes	no	yes	1.2

Station (ft)	Min Reinf.?	Max Spacing	Long. Reinf.	Governed D/C
78.63	yes	yes	yes	0.6
80.54	yes	yes	yes	0.5
82.44	yes	yes	yes	0.5
84.35	yes	yes	yes	0.5
86.25	yes	yes	yes	0.5
88.16	yes	yes	yes	0.5
90.07	yes	yes	yes	0.4
91.97	yes	yes	yes	0.4
93.88	yes	yes	yes	0.4
95.78	yes	yes	yes	0.4
97.69	yes	yes	yes	0.4
98.89	yes	yes	yes	0.4
100.09	yes	yes	yes	0.4
101.29	yes	yes	yes	0.4
102.49	yes	yes	yes	0.4
104.06	yes	yes	yes	0.4
105.63	yes	yes	yes	0.3
107.08	yes	yes	yes	0.3
108.53	yes	yes	yes	0.3
110.31	yes	yes	yes	0.3
112.10	yes	yes	yes	0.2
113.30	yes	yes	yes	0.2
114.50	yes	yes	yes	0.2
115.70	yes	yes	yes	0.2
116.90	yes	yes	yes	0.1
118.10	yes	yes	yes	0.1
119.30	yes	yes	yes	0.1
120.50	yes	yes	yes	0.1
121.71	yes	yes	yes	0.2
122.91	yes	yes	yes	0.2
124.11	yes	yes	yes	0.2
125.31	yes	yes	yes	0.2
126.51	yes	yes	yes	0.2
127.71	yes	yes	yes	0.3
128.91	yes	yes	yes	0.3
130.11	yes	yes	yes	0.3
131.31	yes	yes	yes	0.3
132.51	yes	yes	yes	0.3
133.71	yes	yes	yes	0.3
134.91	yes	yes	yes	0.3
136.11	yes	yes	yes	0.3
137.32	yes	yes	yes	0.3
138.52	yes	yes	yes	0.3
138.93	yes	yes	yes	0.3
140.43	yes	no	yes	0.5
141.43	yes	no	yes	0.5
142.37	yes	yes	yes	0.3
143.32	yes	yes	yes	0.3
144.52	yes	yes	yes	0.4
145.72	yes	yes	yes	0.4
146.92	yes	yes	yes	0.4
148.12	yes	yes	yes	0.4
149.32	yes	yes	yes	0.4
150.52	yes	yes	yes	0.5
151.72	yes	yes	yes	0.5
152.93	yes	no	yes	0.8

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7
Title Flexural Capacity Calculation Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 10/27/2021
Date 10/27/2021

Description:
This sheet summarizes the inputs of the flexural capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons and the mild reinforcement along the bridge superstructure at each cross-sections along the superstructure. The mild reinforcements in the extension are assumed to match with the existing.

Notation Definitions:

I_{22} (ft^4)	box girder cross-section about	As.1, As.2, and As.3	Longitudinal rein. area in top slab for layers 1, 2, and 3
y (ft)	Vertical distance from top of box girder to centroid	As.1, As.2, and As.3	Longitudinal rein. area in bot slab for layers 8 and 9
A (ft^2)	Box girder cross-section area	ds.1, ds.2, ds.3, ds.8, and ds.9	Vertical distance from top of deck to centroid of reinf.
y_{tendon} (ft)	Vertical Distance from Top of Superstructure to Tendon	M_{u_max} (kip.ft)	Max. moment demand
P_e (kip)	Service	M_{u_min} (kip.ft)	Min. moment demand

Station (ft)	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)	As.1 (in^2)	As.2 (in^2)	As.3 (in^2)	As.8 (in^2)	As.9 (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
0.00	127.82	1.65	52.74	1.70	254.000	-1862.940	-3552.910	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
1.73	127.82	1.65	52.74	1.78	254.000	-842.989	-2569.170	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
3.45	127.82	1.65	52.74	1.85	254.000	332.395	-1934.970	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
5.18	127.82	1.65	52.74	1.92	254.000	1429.476	-1363.720	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
6.91	127.82	1.65	52.74	1.99	254.000	2445.722	-843.976	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
8.64	127.82	1.65	52.74	2.07	254.000	3383.216	-372.577	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
10.36	127.82	1.65	52.74	2.14	254.000	4254.570	56.791	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
12.09	127.82	1.65	52.74	2.21	254.000	5166.348	462.986	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
14.02	127.82	1.65	52.74	2.30	254.000	5943.330	845.740	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
15.95	127.82	1.65	52.74	2.38	254.000	6644.632	1182.704	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
17.88	127.82	1.65	52.74	2.46	254.000	7253.418	1474.447	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
19.82	127.82	1.65	52.74	2.54	254.000	7775.904	1721.469	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
21.75	127.82	1.65	52.74	2.62	254.000	8211.120	1924.254	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
23.68	127.82	1.65	52.74	2.70	254.000	8568.802	2083.276	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
25.61	127.82	1.65	52.74	2.79	254.000	8851.060	2199.005	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
27.54	127.82	1.65	52.74	2.87	254.000	9051.736	2271.909	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
29.47	127.82	1.65	52.74	2.95	254.000	9168.391	2302.455	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
31.40	127.82	1.65	52.74	3.03	254.000	9205.137	2291.109	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
33.34	127.82	1.65	52.74	3.11	254.000	9151.047	2238.336	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
35.27	127.82	1.65	52.74	3.19	254.000	9021.003	2144.598	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
37.20	127.82	1.65	52.74	3.28	254.000	8847.977	2010.359	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
39.13	127.82	1.65	52.74	3.25	254.000	8695.382	1874.263	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
40.00	127.82	1.65	52.74	3.20	262.000	8290.301	1621.334	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
41.92	127.82	1.65	52.74	3.09	262.000	8210.845	1584.161	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
43.85	127.82	1.65	52.74	2.98	262.000	7791.469	1319.890	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
45.77	127.82	1.65	52.74	2.86	262.000	7279.325	1019.310	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
47.70	127.82	1.65	52.74	2.75	262.000	6691.276	682.422	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
49.62	127.82	1.65	52.74	2.64	262.000	6042.540	309.226	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
51.54	127.82	1.65	52.74	2.53	262.000	5326.914	-100.277	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
53.47	127.82	1.65	52.74	2.41	262.000	4553.109	-546.087	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
55.39	127.82	1.65	52.74	2.30	262.000	3737.600	-1059.230	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
57.32	127.82	1.65	52.74	2.19	262.000	2868.342	-1622.770	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
59.24	127.82	1.65	52.74	2.08	262.000	1932.002	-2222.620	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
61.16	127.82	1.65	52.74	1.96	262.000	928.422	-2858.770	40.80	33.60	7.20	50.800	50.800	3.56	7.063	11.50	46.76	48.740
63.09	127.82	1.65	52.74	1.85	262.000	-139.025	-3531.230	40.80	33.60	7.20	50.800	50.800	3.56	7.063	11.50	46.76	48.740

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7
Title Flexural Capacity Calculation Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833

Date 10/27/2021
Date 10/27/2021

Description:
This sheet summarizes the inputs of the flexural capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons and the mild reinforcement along the bridge superstructure at each cross-sections along the superstructure. The mild reinforcements in the extension are assumed to match with the existing.

Notation Definitions:

I_{22} (ft^4)	box girder cross-section about	As.1, As.2, and As.3	Longitudinal rein. area in top slab for layers 1, 2, and 3
y (ft)	Vertical distance from top of box girder to centroid	As.1, As.2, and As.3	Longitudinal rein. area in bot slab for layers 8 and 9
A (ft^2)	Box girder cross-section area	ds.1, ds.2, ds.3, ds.8, and ds.9	Vertical distance from top of deck to centroid of reinf.
y_{tendon} (ft)	Vertical Distance from Top of Superstructure to Tendon	M_{u_max} (kip.ft)	Max. moment demand
P_e (kip)	Service	M_{u_min} (kip.ft)	Min. moment demand

Station (ft)	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)	As.1 (in^2)	As.2 (in^2)	As.3 (in^2)	As.8 (in^2)	As.9 (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
65.01	127.82	1.65	52.74	1.74	262.000	-1257.180	-4250.370	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
66.93	127.82	1.65	52.74	1.63	262.000	-2284.790	-5106.170	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
68.86	127.82	1.65	52.74	1.51	262.000	-3159.660	-6208.280	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
70.78	127.82	1.65	52.74	1.40	262.000	-4062.480	-7376.060	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
72.71	127.82	1.65	52.74	1.29	262.000	-5796.270	-9903.140	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
74.63	127.82	1.65	52.74	1.18	262.000	-6656.570	-11421.500	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
76.63	127.82	1.65	52.74	1.06	273.000	-4993.730	-8608.080	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
78.63	123.83	1.61	51.84	1.21	273.000	-5838.630	-9894.760	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
80.54	123.83	1.61	51.84	1.34	273.000	-5059.090	-8598.430	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
82.44	123.83	1.61	51.84	1.47	273.000	-4139.250	-7364.500	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
83.29	123.83	1.61	51.84	1.53	273.000	-3244.350	-6193.880	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
86.25	123.83	1.61	51.84	1.74	273.000	-2377.920	-5087.450	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
88.16	123.83	1.61	51.84	1.88	273.000	-1398.680	-4188.520	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
90.07	123.83	1.61	51.84	2.01	273.000	-336.998	-3446.470	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
91.97	123.83	1.61	51.84	2.14	273.000	706.204	-2754.600	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
93.88	123.83	1.61	51.84	2.28	273.000	1692.842	-2107.800	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
95.78	123.83	1.61	51.84	2.41	273.000	2625.387	-1509.520	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
97.69	123.83	1.61	51.84	2.54	273.000	3481.092	-957.313	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
98.89	123.83	1.61	51.84	2.63	273.000	4012.390	-632.325	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
100.09	123.83	1.61	51.84	2.71	273.000	4508.660	-341.942	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
101.29	123.83	1.61	51.84	2.80	273.000	5012.403	-65.821	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
102.49	123.83	1.61	51.84	2.88	273.000	5466.943	195.961	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
104.06	123.83	1.61	51.84	2.99	273.000	6041.192	516.501	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
105.63	123.83	1.61	51.84	3.10	273.000	6542.771	814.005	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
107.08	123.83	1.61	51.84	3.31	273.000	6940.381	1038.365	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
108.53	123.83	1.61	51.84	3.31	273.000	7293.695	1240.038	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
110.31	123.83	1.61	51.84	3.31	273.000	7640.479	1439.864	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
112.10	123.83	1.61	51.84	3.31	273.000	7920.446	1605.517	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
113.30	123.83	1.61	51.84	3.31	273.000	8068.898	1697.553	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
114.50	123.83	1.61	51.84	3.31	273.000	8185.603	1774.026	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
115.70	123.83	1.61	51.84	3.31	273.000	8275.107	1834.930	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
116.90	123.83	1.61	51.84	3.31	273.000	8350.311	1880.257	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
118.10	123.83	1.61	51.84	3.31	273.000	8393.922	1910.005	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
119.30	123.83	1.61	51.84	3.31	273.000	8406.233	1924.169	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
120.50	123.83	1.61	51.84	3.31	273.000	8386.772	1922.745	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7
Title Flexural Capacity Calculation Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833

Date 10/27/2021
Date 10/27/2021

Description:
This sheet summarizes the inputs of the flexural capacity check for Bridge 17.7 superstructure. The inputs include the forces and the location of the tendons and the mild reinforcement along the bridge superstructure at each cross-sections along the superstructure. The mild reinforcements in the extension are assumed to match with the existing.

Notation Definitions:

I_{22} (ft^4)	box girder cross-section about	As.1, As.2, and As.3	Longitudinal rein. area in top slab for layers 1, 2, and 3
y (ft)	Vertical distance from top of box girder to centroid	As.1, As.2, and As.3	Longitudinal rein. area in bot slab for layers 8 and 9
A (ft^2)	Box girder cross-section area	ds.1, ds.2, ds.3, ds.8, and ds.9	Vertical distance from top of deck to centroid of reinf.
y_{tendon} (ft)	Vertical Distance from Top of Superstructure to Tendon	M_{u_max} (kip.ft)	Max. moment demand
P_e (kip)	Service	M_{u_min} (kip.ft)	Min. moment demand

Station (ft)	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)	As.1 (in^2)	As.2 (in^2)	As.3 (in^2)	As.8 (in^2)	As.9 (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
121.71	123.83	1.61	51.84	3.31	273.000	8335.550	1905.735	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
122.91	123.83	1.61	51.84	3.31	273.000	8252.499	1873.135	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
124.11	123.83	1.61	51.84	3.31	273.000	8137.505	1824.944	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
125.31	123.83	1.61	51.84	3.31	273.000	7990.200	1761.162	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
126.51	123.83	1.61	51.84	3.31	273.000	7810.903	1681.787	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
127.71	123.83	1.61	51.84	3.31	273.000	7599.385	1586.820	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
128.91	123.83	1.61	51.84	3.31	273.000	7365.418	1476.260	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
130.11	123.83	1.61	51.84	3.31	273.000	7102.961	1350.104	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
131.31	123.83	1.61	51.84	3.31	273.000	6808.519	1208.347	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
132.51	123.83	1.61	51.84	3.31	273.000	6482.271	1050.983	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
133.71	123.83	1.61	51.84	3.31	273.000	6124.350	877.998	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
134.91	123.83	1.61	51.84	3.31	273.000	5735.186	689.374	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
136.11	123.83	1.61	51.84	3.31	273.000	5316.290	485.076	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
137.32	123.83	1.61	51.84	3.31	273.000	4867.250	265.053	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
138.52	123.83	1.61	51.84	3.31	273.000	4388.848	29.885	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
138.93	123.83	1.61	51.84	3.31	273.000	4193.340	-56.057	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
140.43	123.83	1.61	51.84	3.44	273.000	3561.383	-382.185	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
141.43	123.83	1.61	51.84	3.31	250.000	3068.396	-628.084	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
142.37	123.83	1.61	51.84	3.18	250.000	2715.219	-805.099	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
143.32	123.83	1.61	51.84	3.05	250.000	2349.148	-988.046	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
144.52	123.83	1.61	51.84	2.89	250.000	1871.442	-1228.440	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
145.72	123.83	1.61	51.84	2.73	250.000	1379.694	-1479.130	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
146.92	123.83	1.61	51.84	2.57	250.000	867.523	-1741.630	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
148.12	123.83	1.61	51.84	2.41	250.000	345.471	-2017.730	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
149.32	123.83	1.61	51.84	2.24	250.000	-199.013	-2313.690	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
150.52	123.83	1.61	51.84	2.08	250.000	-749.622	-2683.670	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
151.72	123.83	1.61	51.84	1.92	250.000	-1265.360	-3153.530	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
152.93	123.83	1.61	51.84	1.76	250.000	-1648.76	-3726.22	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7 **Project No** A207833
Title Flexure Capacity Check (Sample Calcs)
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:This spreadsheet calculates the flexural d/c ratio of a box girder at a single cross-section based on AASHTO 2017. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'flexural capacity input' using vba for all stations. Definitions of notations and sample calculations at critical section are shown in mathcad sheets.

Station	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)
152.93	123.83	1.61	51.84	1.76	250.000	-1648.76	-3726.22

$As.1$ (in^2)	$As.2$ (in^2)	$As.3$ (in^2)	$As.8$ (in^2)	$As.9$ (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875

Section Location West Abutment

1 Material Properties

f'_c (ksi)	5	*concrete compressive strength*
f_r (ksi)	0.537	*concrete modulus of fracture* *(5.4.2.6)*
f_{pu} (ksi)	270	*prestressed tendon ultimate tensile strength*
f_{pe} (ksi)	165	*effective stress in prestressing steel after losses*
f_y (ksi)	60	*reinforcement tensile yield strength*

2 Structural Properties

Girder

b_t (ft)	36.067	*deck width*
b_b (ft)	24	*bottom slab width*
h_{ft} (ft)	0.708	*deck thickness*
h_{fb} (ft)	0.500	*bottom slab thickness*
b_w (ft)	3.363	*total thickness of webs*
h (ft)	4.25	*total height*
h_{fillet} (ft)	0.5	*height of fillet*
h_w (ft)	2.542	*height of web*

Tendon

A_{ps} (ft^2)	0.00151	*strand area*
N_s	7	*amount of strand per tendon*
N_t	4	*amount of tendon*
$A_{ps.total}$ (ft^2)	0.0422	*total tendon area*
d_{p_max} (in)	29.88	*distance from extreme compression fiber to the centroid of the tendons for max mome*
d_{p_min} (in)	29.88	*distance from extreme compression fiber to the centroid of the tendons for min momer*

Mild Reinforcement

E_s (ksf)	4177087.2	*reinforcement modulus of elasticity*

3 Flexural Demand

γ_1	1.6	*flexural cracking variability factor* *(5.6.3.3)*
γ_2	1	*prestress variability factor* *(5.6.3.3)*
γ_3	0.67	*ratio of specified minimum yield strenght to ultimate tensile strenght of mild (5.6.3.3)*

	for M_{u_max}	for M_{u_min}	
$1.33M_u$(kip.ft)	-2192.85	-4955.87	
S_c (ft)	76.913	76.913	*section modulus for the extreme fiber*
e (ft)	-0.1499	-0.1499	*tendon vertical eccentricity*
f_{cpe}(ksi)	-0.120	-0.120	*compressive stress in concrete due to effective prestress forces*
M_{cr} (kip.ft)	-7265.24	-7265.24	*cracking moment*

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Project I-405 Bridge 17.7 **Project No** A207833
Title Flexure Capacity Check (Sample Calcs)
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:This spreadsheet calculates the flexural d/c ratio of a box girder at a single cross-section based on AASHTO 2017. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'flexural capacity input' using vba for all stations. Definitions of notations and sample calculations at critical section are shown in mathcad sheets.

Station	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)
152.93	123.83	1.61	51.84	1.76	250.000	-1648.76	-3726.22

$As.1$ (in^2)	$As.2$ (in^2)	$As.3$ (in^2)	$As.8$ (in^2)	$As.9$ (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875

4 Factored Flexural Resistance

β_1	0.8	*stress block factor*	*(5.6.2.2)*
#of plastic hinges	1	*number of plastic hinges at supports*	*(5.6.3.1.2)*
l_i (ft)	152.93	*length of tendon between anchorages*	*(5.6.3.1.2)*
l_e (ft)	101.95	*effecitve tendon length*	*(5.6.3.1.2)*

	for M_{u_max}	for M_{u_min}	
f_{ps} (ksi)	183.56	183.56	*average tendon strength*
c (in)	4.08	4.08	*distance from compression extreme fibre to neutral*
a (in)	3.264	3.264	*depth of equivalent rectangular stress block*
Φ	0.9	0.9	*flexural resistance factor*
M_n (kip.ft)	-13065.73	-13065.73	*moment resistance*

Note:
1. compression reinforcements and all the reinforcements in the webs are ignored in the calculation of c
2. the tensile reinforcements in the slabs are assumed to yield in the calculation of c

5 Summary

for M_{u_max}

	Demand	Resistance	D/C
Minimum Reinforcement	-2192.851	-11759.153	0.186
Ultimate Capcity	-1648.760	-11759.153	0.140

for M_{u_min}

	Demand	Resistance	D/C
Minimum Reinforcement	-4955.873	-11759.153	0.421
Capcity	-3726.220	-11759.153	0.317

For maximum demand
This cross-section is governed by:
minimum reinforcement check
D/C ratio for this cross section is:
0.186

For minimum demand
This cross-section is governed by:
minimum reinforcement check
D/C ratio for this cross section is:
0.421

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Project I-405 Bridge 17.7 **Project No** A207833
Title Flexural Capacities
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:
This sheet summarizes the flexural capacities and demands for the proposed structure superstructure.

Station (ft)	Demand for Max. M	Demand for Min. M	Capacity for Max. M	Capacity for Min. M	RESPONSE 2000 Positive Capacity
0.00	-2478	-4725	-7724	-7724	-12613.00
1.73	-1121	-3417	-7641	-7641	-12613.00
3.45	442	-2574	11231	-7558	-12613.00
5.18	1901	-1814	11310	-7476	-12613.00
6.91	3253	-1122	11389	-7394	17107.00
8.64	4500	-496	11469	-7313	17107.00
10.36	5038	76	11550	11550	17107.00
12.09	5166	616	11631	11631	17107.00
14.02	5943	1125	23078	23078	
15.95	6645	1573	23168	23168	
17.88	7253	1961	23259	23259	
19.82	7776	2290	23351	23351	
21.75	8211	2559	23443	23443	
23.68	8569	2771	23536	23536	
25.61	8851	2925	23629	23629	
27.54	9052	3022	23723	23723	
29.47	9168	3062	23818	23818	
31.40	9205	3047	23913	23913	
33.34	9151	2977	24009	24009	
35.27	9021	2852	24106	24106	
37.20	8848	2674	24203	24203	
39.13	8695	2493	24175	24175	
40.00	8290	2156	24197	24197	
41.92	8211	2107	24061	24061	
43.85	7791	1755	23926	23926	
45.77	7279	1356	23793	23793	
47.70	6691	908	23660	23660	
49.62	6043	411	23529	23529	
51.54	5340	-133	23399	-11408	
53.47	5261	-726	23270	-11529	
55.39	4971	-1409	23143	-11653	
57.32	3815	-2158	23016	-11777	
59.24	2570	-2956	22891	-11902	
61.16	1235	-3802	22767	-17866	
63.09	-185	-4697	-17993	-17993	
65.01	-1672	-5653	-18120	-18120	
66.93	-3039	-6791	-18249	-18249	
68.86	-4202	-7555	-18379	-18379	
70.78	-5403	-7634	-18510	-18510	
72.71	-7709	-9903	-18643	-18643	
74.63	-7792	-11422	-18776	-18776	
76.63	-6642	-8608	-19026	-19026	

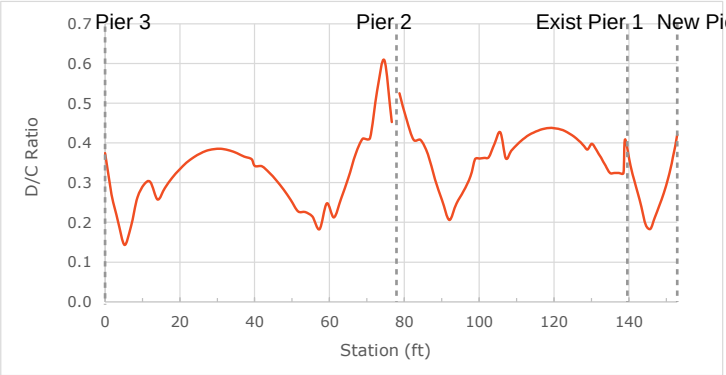
Station (ft)	Demand for Max. M	Demand for Min. M	Capacity for Max. M	Capacity for Min. M	RESPONSE 2000 Positive Capacity
78.63	-7753	-9895	-18846	-18846	
80.54	-6729	-8598	-18682	-18682	
82.44	-5505	-7557	-18520	-18520	
84.35	-4315	-7513	-18449	-18449	
86.25	-3163	-6766	-18201	-18201	
88.16	-1860	-5571	-18044	-18044	
90.07	-448	-4584	-17889	-17889	
91.97	939	-3664	12210	-17735	
93.88	2251	-2803	12367	-11338	
95.78	3492	-2008	12527	-11186	
97.69	4630	-1273	14631	-11036	
98.89	5293	-841	14733	-10942	
100.09	5355	-455	14835	-10849	
101.29	5417	-88	14939	-10756	
102.49	5478	261	15043	15043	
104.06	6041	687	15180	15180	
105.63	6543	1083	15318	15318	
107.08	6940	1381	15582	15582	19198.000
108.53	7294	1649	15582	15582	19198.000
110.31	7640	1915	15582	15582	19198.000
112.10	7920	2135	15582	15582	19198.000
113.30	8069	2258	15582	15582	19198.000
114.50	8186	2359	15582	15582	19198.000
115.70	8275	2440	15582	15582	19198.000
116.90	8350	2501	15582	15582	19198.000
118.10	8394	2540	15582	15582	19198.000
119.30	8406	2559	15582	15582	19198.000
120.50	8387	2557	15582	15582	19198.000
121.71	8336	2535	15582	15582	19198.000
122.91	8252	2491	15582	15582	19198.000
124.11	8138	2427	15582	15582	19198.000
125.31	7990	2342	15582	15582	19198.000
126.51	7811	2237	15582	15582	19198.000
127.71	7599	2110	15582	15582	19198.000
128.91	7365	1963	15582	15582	19198.000
130.11	7103	1796	13640	13640	17844.000
131.31	6809	1607	13640	13640	17844.000
132.51	6482	1398	13640	13640	17844.000
133.71	6124	1168	13640	13640	17844.000
134.91	5792	917	13640	13640	17844.000
136.11	5792	645	13640	13640	17844.000
137.32	5792	353	13640	13640	17844.000
138.52	5792	40	13640	13640	17844.000
138.93	5577	-75	13640	-10208	
140.43	4737	-508	13808	-10074	
141.43	4081	-835	13377	-10158	
142.37	3611	-1071	13225	-10281	
143.32	3124	-1314	13075	-10406	
144.52	2489	-1634	12887	-10566	
145.72	1835	-1967	12701	-10729	
146.92	1154	-2316	12517	-10894	
148.12	459	-2684	12337	-11062	
149.32	-265	-3077	-11233	-11233	
150.52	-997	-3569	-11406	-11406	
151.72	-1683	-4194	-11581	-11581	
152.93	-2193	-4956	-11759	-11759	

Note: station starts from CL of Pier 3

COWI

Project I-405 Bridge 17.7 **Project No** A207833
Title Flexure Capacity D/C
PREPARED BY LDLI **Date** 10/27/2021
CHECKED BY CUPG **Date** 10/27/2021

Description:
This sheet summarizes the flexure D/C ratios for the proposed structure superstructure.



Note: the longitudinal reinforcements in the flare region when station between 12.09 and 39.13 ft are ignored in the capacity calculation, so the actual D/C ratios in cross-sections within the flare region are less than shown in the plot

Station (ft)	D/C Ratio for Max. M	D/C Ratio for Min. M	Governed D/C
0.00	0.20	0.37	0.4
1.73	0.09	0.27	0.3
3.45	0.03	0.20	0.2
5.18	0.11	0.14	0.1
6.91	0.19	0.04	0.2
8.64	0.26	0.04	0.3
10.36	0.29	0.00	0.3
12.09	0.30	0.04	0.3
14.02	0.26	0.05	0.3
15.95	0.29	0.07	0.3
17.88	0.31	0.08	0.3
19.82	0.33	0.10	0.3
21.75	0.35	0.11	0.4
23.68	0.36	0.12	0.4
25.61	0.37	0.12	0.4
27.54	0.38	0.13	0.4
29.47	0.38	0.13	0.4
31.40	0.38	0.13	0.4
33.34	0.38	0.12	0.4
35.27	0.37	0.12	0.4
37.20	0.37	0.11	0.4
39.13	0.36	0.10	0.4
40.00	0.34	0.09	0.3
41.92	0.34	0.09	0.3
43.85	0.33	0.07	0.3
45.77	0.31	0.06	0.3
47.70	0.28	0.04	0.3
49.62	0.26	0.02	0.3
51.54	0.23	0.01	0.2
53.47	0.23	0.06	0.2
55.39	0.21	0.12	0.2
57.32	0.17	0.18	0.2
59.24	0.11	0.25	0.2
61.16	0.05	0.21	0.2
63.09	0.01	0.26	0.3
65.01	0.09	0.31	0.3
66.93	0.17	0.37	0.4
68.86	0.23	0.41	0.4
70.78	0.29	0.41	0.4
72.71	0.41	0.53	0.5
74.63	0.41	0.61	0.6
76.63	0.35	0.45	0.5

Note: Response 2000 capacities are used to calculate D/C ratio for Max M for stations 0-12.09 and 107.08-138.52

Station (ft)	D/C Ratio for Max. M	D/C Ratio for	Governed D/C
78.63	0.41	0.53	0.5
80.54	0.36	0.46	0.5
82.44	0.30	0.41	0.4
84.35	0.23	0.41	0.4
86.25	0.17	0.37	0.4
88.16	0.10	0.31	0.3
90.07	0.03	0.26	0.3
91.97	0.08	0.21	0.2
93.88	0.18	0.25	0.2
95.78	0.28	0.18	0.3
97.69	0.32	0.12	0.3
98.89	0.36	0.08	0.4
100.09	0.36	0.04	0.4
101.29	0.36	0.01	0.4
102.49	0.36	0.02	0.4
104.06	0.40	0.05	0.4
105.63	0.43	0.07	0.4
107.08	0.36	0.07	0.4
108.53	0.38	0.09	0.4
110.31	0.40	0.10	0.4
112.10	0.41	0.11	0.4
113.30	0.42	0.12	0.4
114.50	0.43	0.12	0.4
115.70	0.43	0.13	0.4
116.90	0.43	0.13	0.4
118.10	0.44	0.13	0.4
119.30	0.44	0.13	0.4
120.50	0.44	0.13	0.4
121.71	0.43	0.13	0.4
122.91	0.43	0.13	0.4
124.11	0.42	0.13	0.4
125.31	0.42	0.12	0.4
126.51	0.41	0.12	0.4
127.71	0.40	0.11	0.4
128.91	0.38	0.10	0.4
130.11	0.40	0.10	0.4
131.31	0.38	0.09	0.4
132.51	0.36	0.08	0.4
133.71	0.34	0.07	0.3
134.91	0.32	0.05	0.3
136.11	0.32	0.04	0.3
137.32	0.32	0.02	0.3
138.52	0.32	0.00	0.3
138.93	0.41	0.01	0.4
140.43	0.34	0.05	0.3
141.43	0.31	0.08	0.3
142.37	0.27	0.10	0.3
143.32	0.24	0.13	0.2
144.52	0.19	0.15	0.2
145.72	0.14	0.18	0.2
146.92	0.09	0.21	0.2
148.12	0.04	0.24	0.2
149.32	0.02	0.27	0.3
150.52	0.09	0.31	0.3
151.72	0.15	0.36	0.4
152.93	0.19	0.42	0.4

Note: station starts from CL of Pier 3

Serviceability Check

Description

The purpose of the following calculations is to calculate and check the deformation and reinforcement distribution of the proposed structure in the service limit state. The check is conducted at east span where the bending moment is at maximum in magnitude, intermediate pier (Pier 2), west span where the bending moment is at maximum in magnitude, and interface between the existing and new box girder.

The calculations and checks are based upon AASHTO 8th Edition, 2017.

1. Purpose and Assumptions

1. The purpose is to calculate and check the deformation and the reinforcement distribution for the service limit states in east span
2. The calculation is based on AASHTO 8th Edition, 2017

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}} \right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi} \right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft$
Bottom Width	$b_b := 24.021ft$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6.5in$
Web Thickness	$b_w := (10in \cdot 2 + 9in \cdot 2) = 3.167 \cdot ft$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 127.82ft^4$
Vertical Distance from Top of Deck to Centroid of Cross-section	$y := 1.647ft$
Cross-section Area	$A := 52.739ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$

West Span Length $L_{west} := 76.295ft$

East Span Length $L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #10 $D_{N10} := 1.27in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

Young's Modulus of Reinforcement $E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	37	$A_{s.1} := 37 \cdot 0.6in^2 = 22.2 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	27	$A_{s.2} := 27 \cdot 0.6in^2 = 16.2 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#10	40	$A_{s.8} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#10	40	$A_{s.9} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N10}}{2}\right) = 46.635 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N10}}{2}\right) = 48.865 \cdot in$

Maximum Spacing in the Layer Closest to the Tension Face $S_{max} := \frac{(3ft + 10.875in)}{4} = 11.719 \cdot in$

Minimum Spacing in the Layer Closest to the Tension Face $S_{min} := \frac{(3ft + 0.75in)}{9} = 4.083 \cdot in$

3. Reinforcement Distribution

Exposure Condition: Class 2

Exposure Factor (5.6.7) $\gamma_e := 0.75$

Aggregate Size $a_g := 0.75in$ — Note: as per AASHTO C5.4.2.1

Positive Moment at SLS $M_{sls} := 5438kip \cdot ft$ — Note: maximum moment at east span from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination

Axial Force at SLS $P_{sls} := -1672kip$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_c := c_b + D_{N4} + \frac{D_{N10}}{2} = 2.135 \cdot in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_s := 1 + \frac{d_c}{0.7 \cdot \left(h - d_c\right)} = 1.062$

Tensile strain in nonprestressed reinforcement at SLS $\varepsilon_s := 0.34 \cdot 10^{-3}$ — Note: From Response 2000 Tensile Stress in Nonprestressed Reinforcement at SLS

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss} := \varepsilon_s \cdot E_s = 9.863 \cdot ksi$$

Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7)

$$S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 45.834 \cdot in$$

Minimum Spacing for CIP Concrete (5.10.3.1.1)

$$S_{min.limit} := \max\left(1.5 \cdot D_{N10}, 1.5in, 1.5 \cdot a_g\right) = 1.905 \cdot in$$

Maximum Allowable Spacing (5.10.3.2)

$$S_{max.limit} := S_{max.limit} = 45.834 \cdot in$$

4. Deformations

Deflection due to Vehicle and Pedestrian Load

$$\Delta := 0.0119ft = 0.143 \cdot in$$

Note: deflection is the maximum absolute deflection from csibridge load combination "LL_Sup_Def"

Cracking Moment (5.6.3.5.2)

$$M_{cr} := f_r \cdot \frac{I_{22}}{h - y} = 3.795 \times 10^3 \cdot kip \cdot ft$$

Maximum Moment in a Component at the Stage for which Deformation is Computed

$$M_a := 5927kip \cdot ft$$

Note: M.sls is from csibridge. It's the maximum moment in mid span in load combintation 'SE1-1 SUP_DEF_ENV'

Maximum Moment in a Component at the Stage for which Deformation is Computed

$$P_a := -1627kip$$

Curvature at Ma (from Response 2000)

$$\rho := \frac{12.07}{10^6} \frac{rad}{in}$$

Effective Moment of Inertia (5.6.3.5.2)

$$I_{eff} := \frac{M_a}{E_c \cdot \rho} = 1.202 \times 10^6 \cdot in^4$$

Adjustment Factor for Deflection

$$\alpha := \frac{I_{eff}}{I_{22}} = 0.453$$

Adjusted Deflection due to Vehicle and Pedestrian Load

$$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.315 \cdot in$$

Maximum Allowable Deflection (2.5.2.6.2)

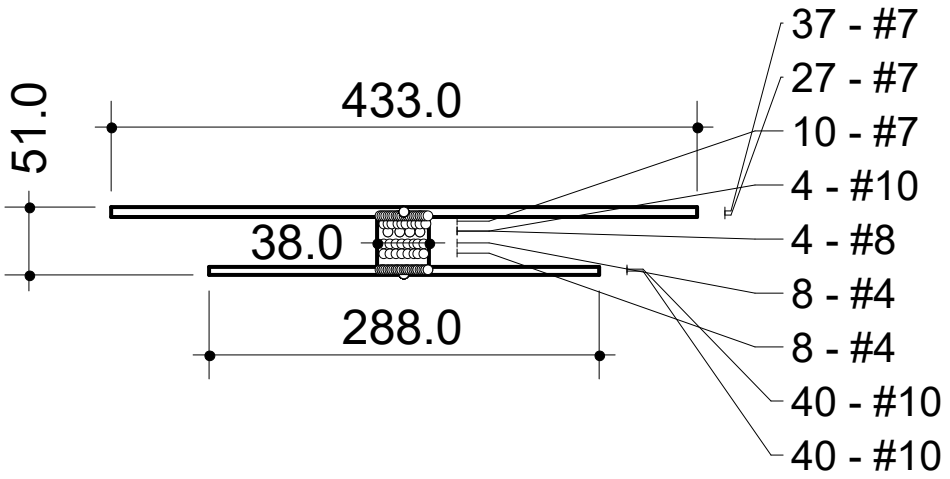
$$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$$

5. Summary

Maximum Spacing in Design	$S_{max} = 11.719 \cdot in$	$\frac{((S_{max}))}{(S_{max.limit})} = 0.256$
Maximum Allowable Spacing	$S_{max.limit} = 45.834 \cdot in$	
Maximum Spacing Check	$if(S_{max} \le S_{max.limit}, "ok", "fail") = "ok"$	
Minimum Spacing in Design	$S_{min} = 4.083 \cdot in$	
Minimum Allowable Spacing	$S_{min.limit} = 1.905 \cdot in$	$\frac{(S_{min.limit})}{S_{min}} = 0.467$
Minimum Spacing Check	$if(S_{min} \ge S_{min.limit}, "ok", "fail") = "ok"$	
Adjusted Deflection due to Vehicle and Pedestrian Load	$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.315 \cdot in$	$\frac{\Delta_{adjusted}}{\Delta_{max.limit}} = 0.343$
Maximum Allowable Deflection (2.5.2.6.2)	$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$	
Deflection Check	$if(\Delta_{adjusted} \le \Delta_{max.limit}, "ok", "fail") = "ok"$	

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6920.5	8029.1
Inertia (in^4)	2587651.1	3167868.7
y_t (in)	20.4	22.3
y_b (in)	30.6	28.7
S_t (in^3)	126752.4	142110.0
S_b (in^3)	84605.3	110346.7



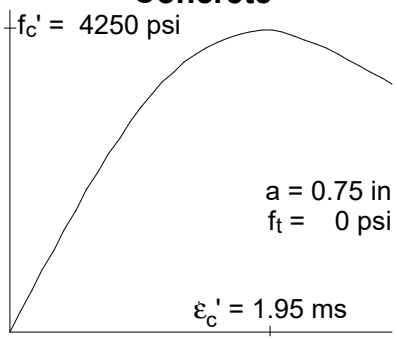
Crack Spacing

2 x dist + 0.1 d_b /ρ

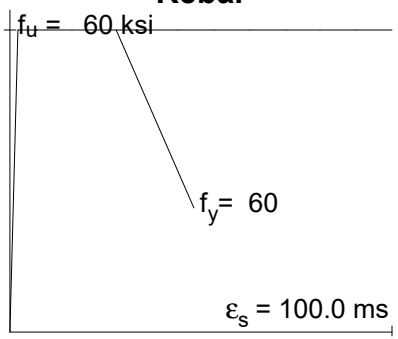
Loading (N,M,V + dN,dM,dV)

-1672 , 6179 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.36 in

Serviceability Check for East Span

Response-2000 v 1.0.5

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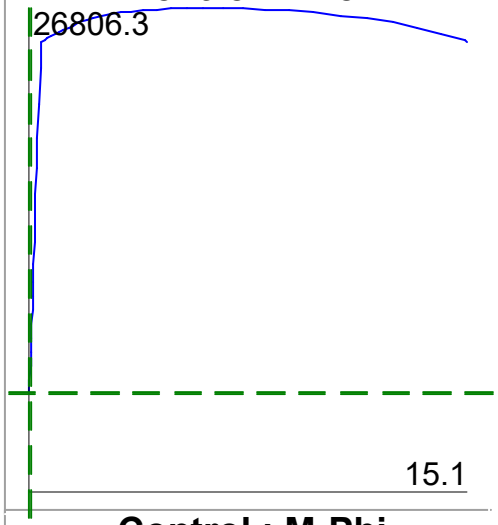
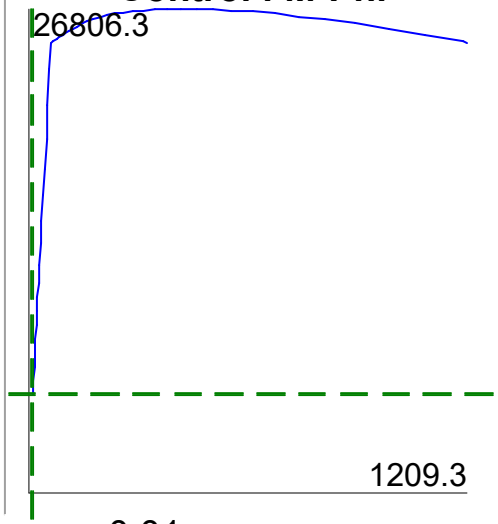
Control : M-ex



Control : M-Phi



ε_{x0} = 0.01 ms

ϕ = 10.84 rad/10^6 in

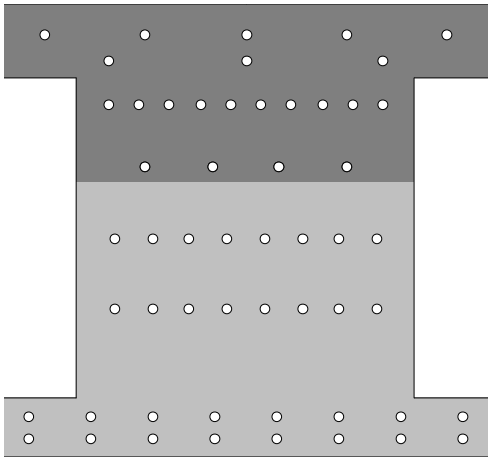
γ_{xy}(avg) = 0.00 ms

Axial Load = -1626.2 kips

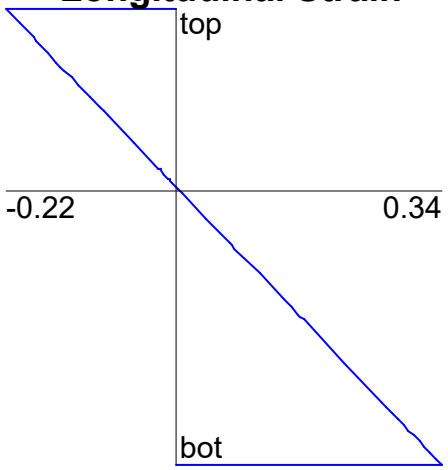
Moment:= 5431.1 ft-kips

Shear = 0.0 kips

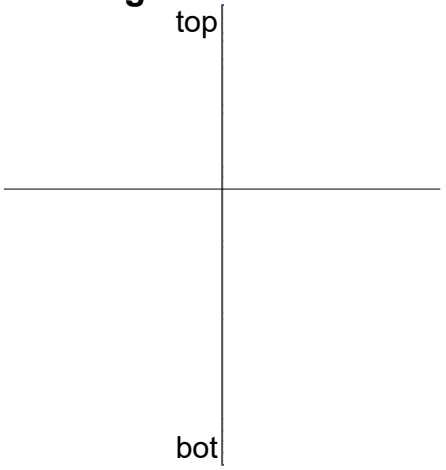
Cross Section



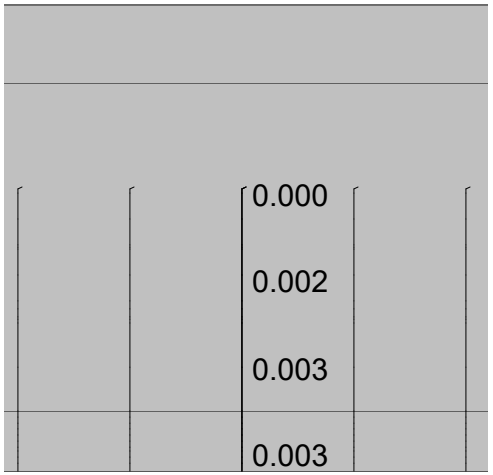
Longitudinal Strain



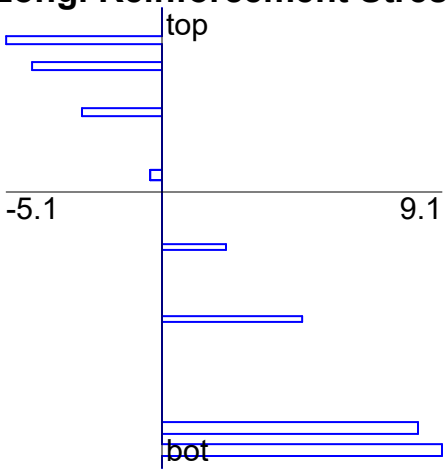
Shrinkage & Thermal Strain



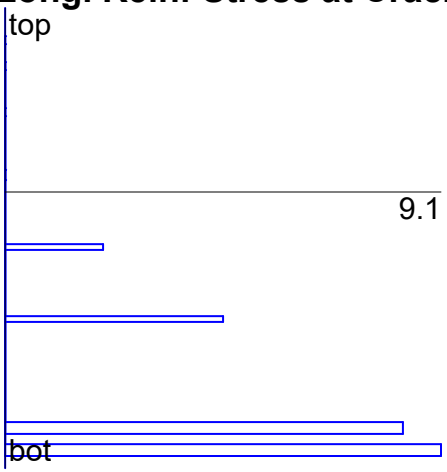
Crack Diagram



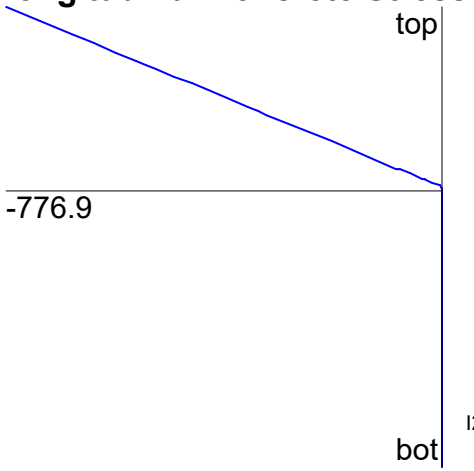
Long. Reinforcement Stress



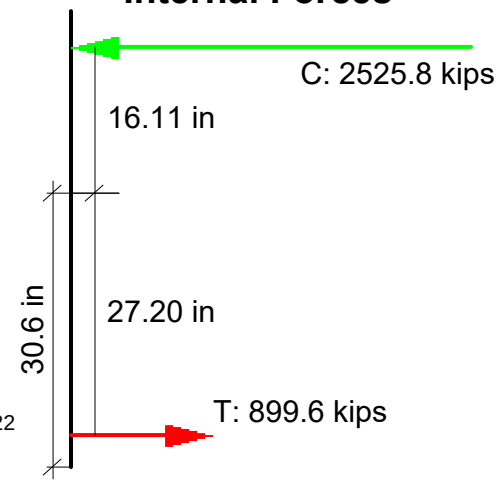
Long. Reinf Stress at Crack



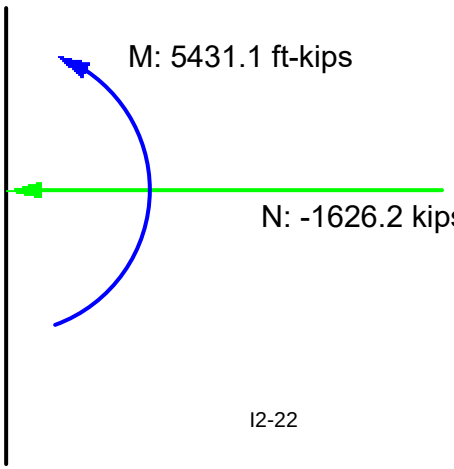
Longitudinal Concrete Stress



Internal Forces

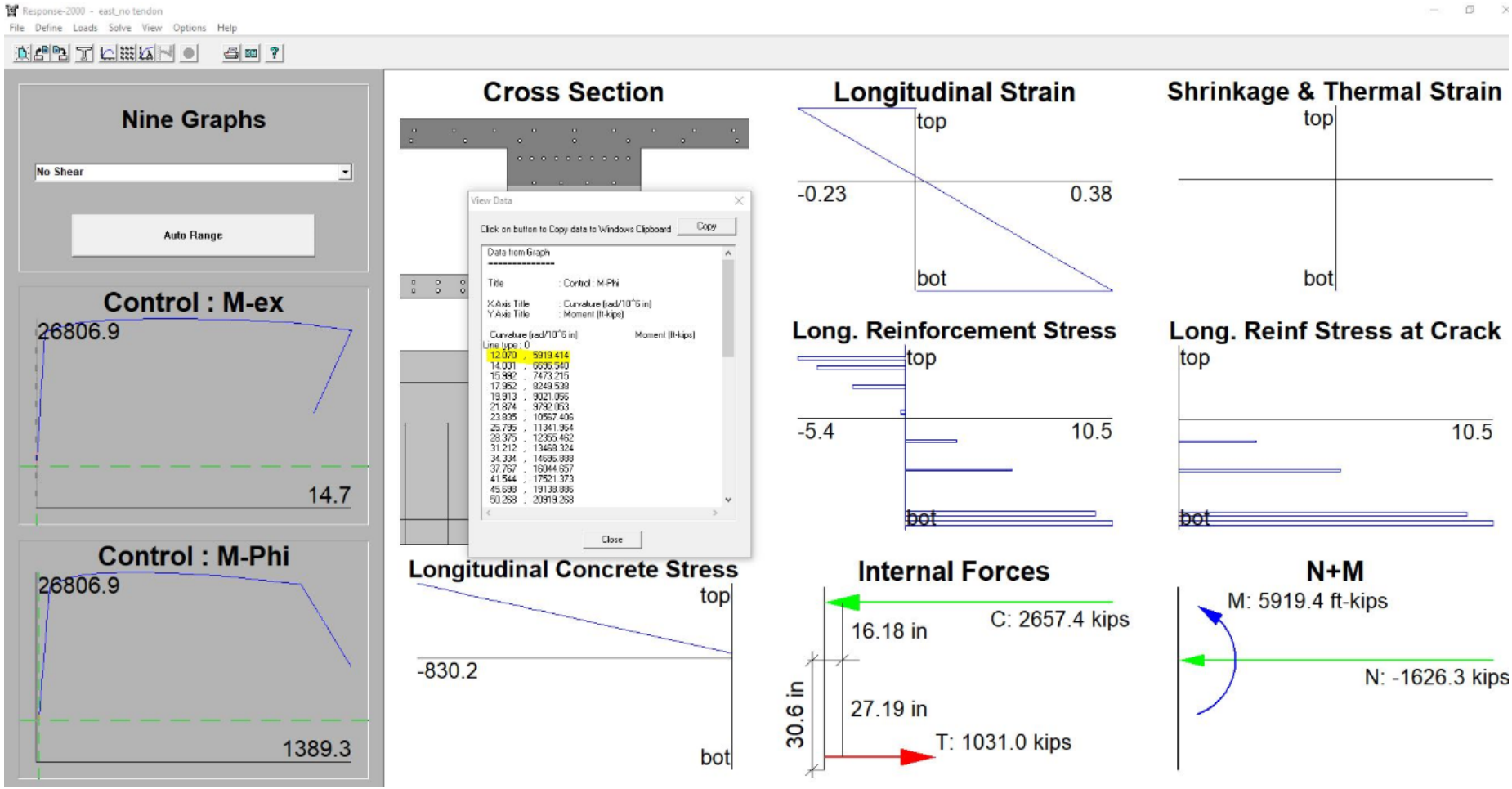


N+M



I2-22

I2-22


Response-2000 - east_no tendon
File Define Loads Solve View Options Help
Nine Graphs
No Shear
Auto Range
Control : M-ex
26806.9
14.7
Control : M-Phi
26806.9
1389.3
Cross Section
View Data
Click on button to Copy data to Windows Clipboard
Copy
Data from Graph
Title : Control : M-Phi
X Axis Title : Curvature (rad/10^5 in)
Y Axis Title : Moment (ft-kips)
Curvature (rad/10^5 in) Moment (ft-kips)
Line type : 0
12.070 , 5919.414
14.001 , 9636.940
15.982 , 7473.215
17.952 , 8249.538
19.913 , 9021.056
21.874 , 9792.053
23.835 , 10567.406
25.795 , 11341.364
28.375 , 12355.462
31.212 , 13468.324
34.334 , 14636.888
37.757 , 16944.857
41.544 , 17521.373
45.698 , 19138.886
50.268 , 20919.268
Close
Longitudinal Concrete Stress
top
-830.2
bot
Longitudinal Strain
top
-0.23
0.38
bot
Long. Reinforcement Stress
top
-5.4
10.5
bot
Internal Forces
C: 2657.4 kips
16.18 in
30.6 in
27.19 in
T: 1031.0 kips
Shrinkage & Thermal Strain
top
bot
Long. Reinf Stress at Crack
top
10.5
bot
N+M
M: 5919.4 ft-kips
N: -1626.3 kips

I2-23

1. Purpose and Assumptions

1. The purpose is to calculate and check the reinforcement distribution for the service limit states at the pier with sectional properties of the east span
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}} \right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi} \right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft = 432.876 \cdot in$
Bottom Width	$b_b := 24.021ft = 288.252 \cdot in$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6.5in$
Total Depth	$h := 4.25ft = 51 \cdot in$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 127.82ft^4$
Vertical Distance from Top of Deck to Centroid	$y := 1.647ft$
Cross-section Area	$A := 52.739ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$
West Span Length	$L_{west} := 76.295$

East Span Length $L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #8 $D_{N8} := 1in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	68	$A_{s.1} := 68 \cdot 0.6in^2 = 40.8 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	56	$A_{s.2} := 56 \cdot 0.6in^2 = 33.6 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	12	$A_{s.3} := 12 \cdot 0.6in^2 = 7.2 \cdot in^2$
Bar in Web (Layer 1)	#8	8	$A_{s.4} := 8 \cdot 0.79in^2 = 6.32 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	30	$A_{s.8} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	30	$A_{s.9} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := \left(h - c_{t.t} - D_{N5}\right) - \frac{D_{N7}}{2} = 47.437 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left[h - \left(h_{ft} - c_b\right) + D_{N5}\right] + \frac{D_{N7}}{2} = 44.563 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \left(h - h_{ft}\right) - \frac{h_{fillet}}{2} = 39.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := 32.5in = 32.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} - s_w = 24.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} - s_w = 16.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} - s_w = 8.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2} = 4.5 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := c_b + D_{N4} + \frac{D_{N8}}{2} = 2 \cdot in$

Maximum Spacing in the Layer Closest to the Tension Face $S_{max} := \frac{(3ft + 6.25in)}{6} = 7.042 \cdot in$

Minimum Spacing in the Layer Closest to the Tension Face $S_{min} := \frac{(2ft + 10.875in)}{6} = 5.813 \cdot in$

3. Reinforcement Distribution

Exposure Condition: Class 2

Exposure Factor (5.6.7) $\gamma_e := 0.75$

Aggregate Size $a_g := 0.75in$ Note: as per AASHTO C5.4.2.1

Negative Moment at SLS $M_{sls} := -7724kip \cdot ft$ Note: maximum moment at pier from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination

Axial Force at SLS $P_{sls} := -1684kip$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_c := c_{t.t} + D_{N4} + \frac{D_{N7}}{2} = 3.438 \cdot in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_s := 1 + \frac{d_c}{0.7 \cdot \left(h - d_c\right)} = 1.103$

Tensile strain in nonprestressed reinforcement at SLS	$\varepsilon_s := 0.58 \cdot 10^{-3}$	Note: From Response 2000 Tensile Stress in Nonprestressed Reinforcement at SLS
Tensile stress in nonprestressed reinforcement at SLS	$f_{ss} := \varepsilon_s \cdot E_s = 16.824 \cdot ksi$	
Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7)	$S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 21.409 \cdot in$	
Minimum Spacing for CIP Concrete (5.10.3.1.1)	$S_{min.limit} := \max(1.5 \cdot D_{N7}, 1.5in, 1.5 \cdot a_g) = 1.5 \cdot in$	

4. Summary

Maximum Spacing in Design	$S_{max} = 7.042 \cdot in$	$\frac{(S_{max})}{(S_{max.limit})} = 0.329$
Maximum Allowable Spacing	$S_{max.limit} = 21.409 \cdot in$	
Maximum Spacing Check	$if(S_{max} \le S_{max.limit}, "ok", "fail") = "ok"$	
Minimum Spacing in Design	$S_{min} = 5.813 \cdot in$	$\frac{(S_{min.limit})}{S_{min}} = 0.258$
Minimum Allowable Spacing	$S_{min.limit} = 1.5 \cdot in$	
Minimum Spacing Check	$if(S_{min} \ge S_{min.limit}, "ok", "fail") = "ok"$	

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	7004.3	7974.6
Inertia (in^4)	2615902.7	3044530.6
y_t (in)	20.5	20.8
y_b (in)	30.7	30.4
S_t (in^3)	127448.7	146270.9
S_b (in^3)	85334.1	100262.3

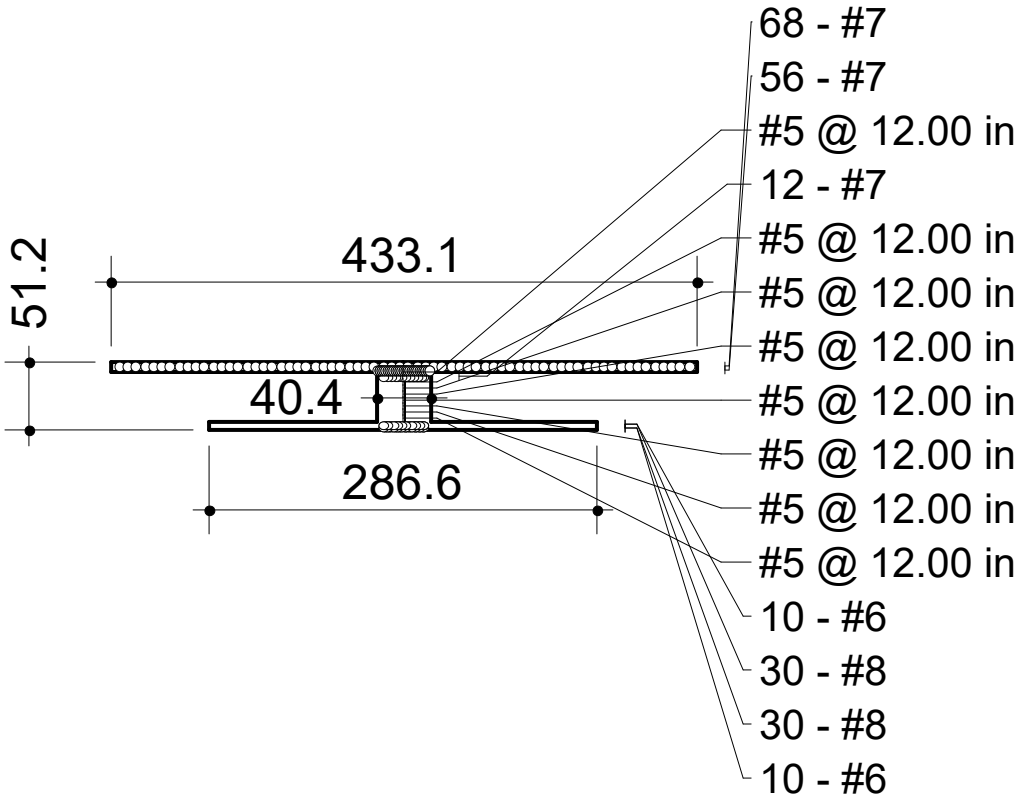
Crack Spacing

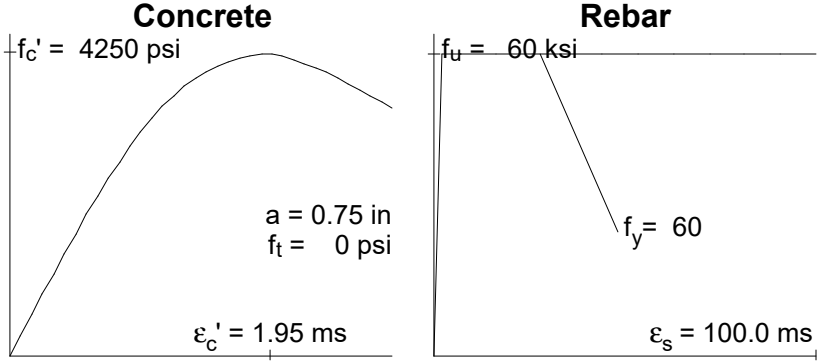
2 x dist + 0.1 d_b /ρ

Loading (N,M,V + dN,dM,dV)

-1684 , -5852 , 0.0 + 0.0 , -1.0 , 0.0

Concrete Shrinkage/Thermal Strain -0.02 ms





All dimensions in inches
Clear cover to transverse reinforcement = 0.69 in

I2-28

Response-2000 v 1.0.5

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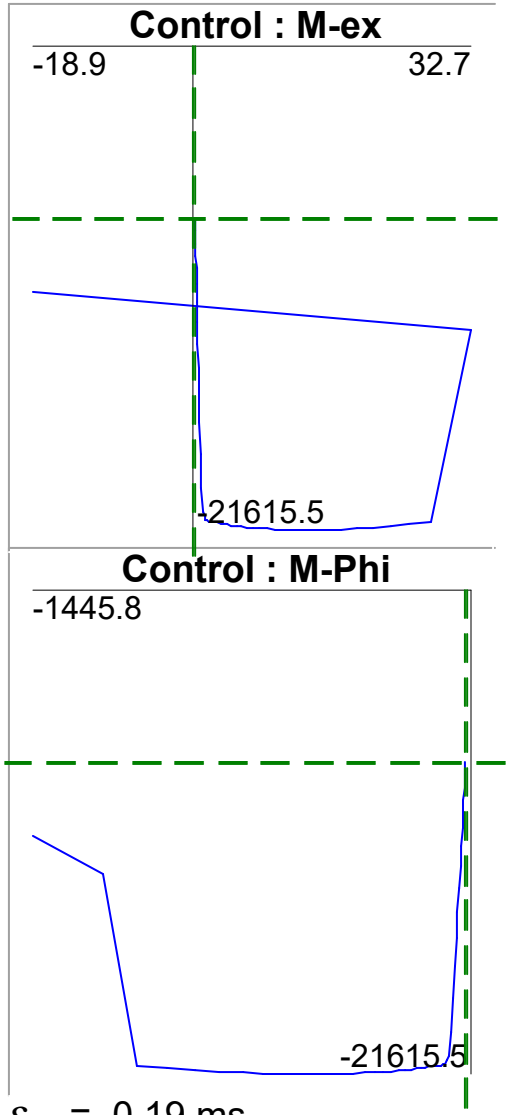


ε_{x0} = 0.19 ms

ϕ = -19.03 rad/10^6 in

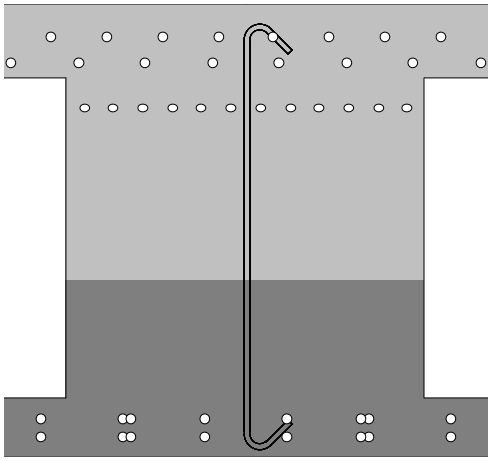
γ_{xy}(avg) = 0.00 ms

Axial Load = -1684.1 kips

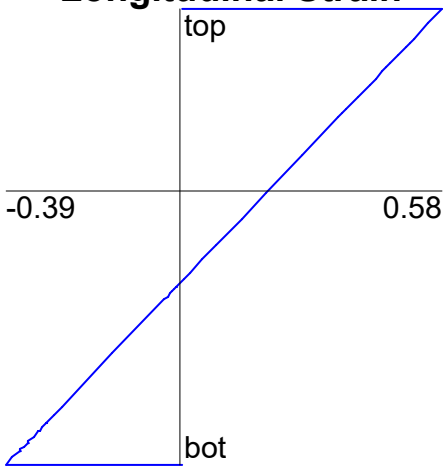
Moment:= -7714.9 ft-kips

Shear = 0.0 kips

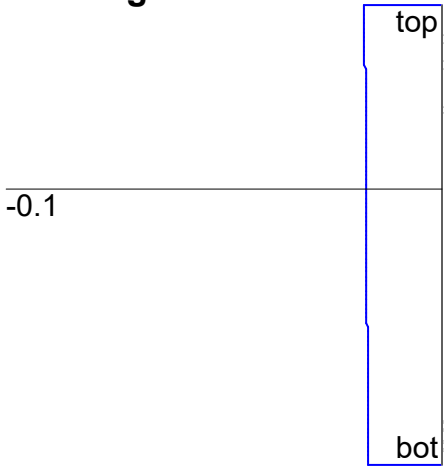
Cross Section



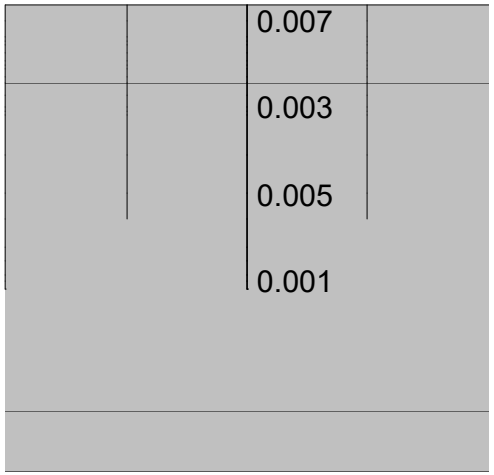
Longitudinal Strain



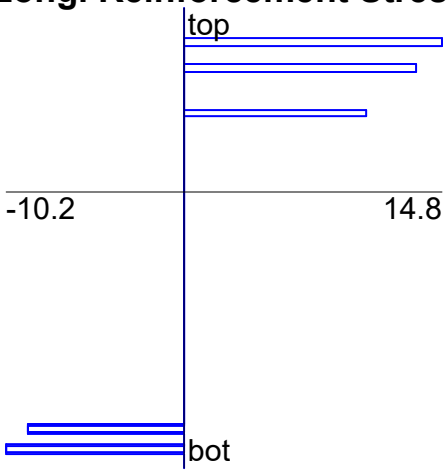
Shrinkage & Thermal Strain



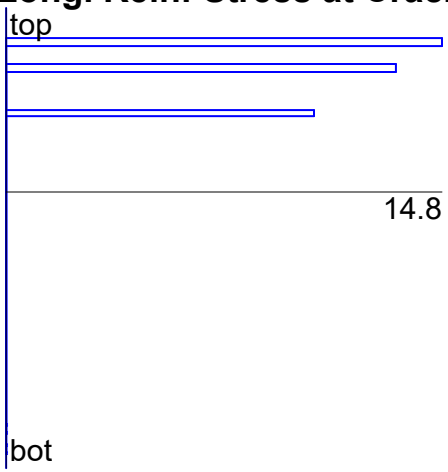
Crack Diagram



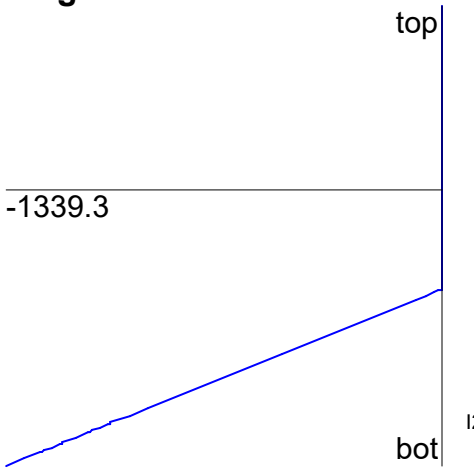
Long. Reinforcement Stress



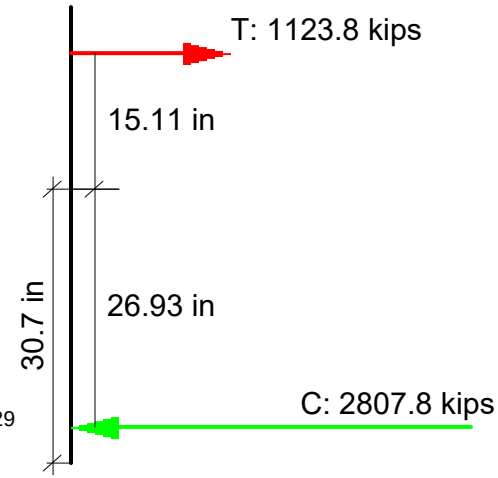
Long. Reinf Stress at Crack



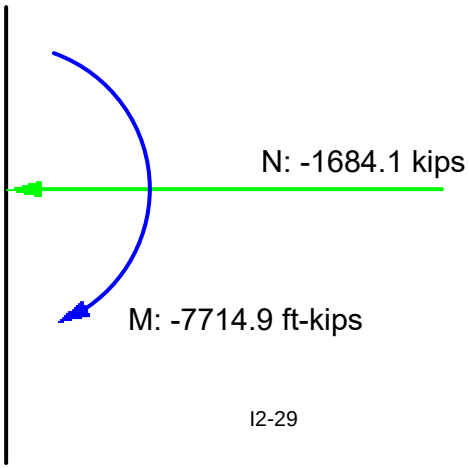
Longitudinal Concrete Stress



Internal Forces



N+M



1. Purpose and Assumptions

1. The purpose is to calculate and check the reinforcement distribution at Pier 2 (west span) for the service limit states
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}} \right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi} \right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft$
Bottom Width	$b_b := 24ft$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6in$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 123.83ft^4$
Vertical Distance from Top of Deck to the Centroid of Cross-section	$y := 1.61ft$
Cross-section Area	$A := 51.842ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$
West Span Length	$L_{west} := 76.295$
East Span Length	$L_{east} := 76.63ft$

Tendon

Strand Area $A_{ps} := 0.217in^2$

Number of Strands per Tendon $N_s := 7$

Number of Tendon $N_t := 4$

Total Tendon Area $A_{ps.total} := A_{ps} \cdot N_s \cdot N_t = 6.076 \cdot in^2$

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #8 $D_{N8} := 1in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	68	$A_{s.1} := 68 \cdot 0.6in^2 = 40.8 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	56	$A_{s.2} := 56 \cdot 0.6in^2 = 33.6 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	12	$A_{s.3} := 12 \cdot 0.6in^2 = 7.2 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2 \cdot in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2 \cdot in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2 \cdot in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	30	$A_{s.8} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	30	$A_{s.9} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := h - c_{t.t} - D_{N5} - \frac{D_{N7}}{2} = 47.437 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := h - h_{ft} + c_b + D_{N5} + \frac{D_{N7}}{2} = 44.563 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := h - h_{ft} - \frac{h_{fillet}}{2} = 39.5 \cdot in$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Web (Layer 1)	$d_{s.4} := 32.5in$
Bar in Web (Layer 2)	$d_{s.5} := d_{s.4} - s_w = 24.5 \cdot in$
Bar in Web (Layer 3)	$d_{s.6} := d_{s.5} - s_w = 16.5 \cdot in$
Bar in Web (Layer 4)	$d_{s.7} := d_{s.6} - s_w = 8.5 \cdot in$
Bar in Bottom Slab (Layer 1)	$d_{s.8} := h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2} = 4 \cdot in$
Bar in Bottom Slab (Layer 2)	$d_{s.9} := c_b + D_{N4} + \frac{D_{N8}}{2} = 2 \cdot in$
Maximum Spacing in the Layer Closest to the Tension Face	$S_{max} := \frac{(3ft + 10.625in)}{7} = 6.661 \cdot in$
Minimum Spacing in the Layer Closest to the Tension Face	$S_{min} := \frac{(3ft + 11in)}{8} = 5.875 \cdot in$

3. Reinforcement Distribution

Exposure Condition	Class 2	
Exposure Factor (5.6.7)	$\gamma_e := 0.75$	
Aggregate Size	$a_g := 0.75in$	Note: as per AASHTO C5.4.2.1
Negative Moment at SLS	$M_{sls} := -7724kip \cdot ft$	Note: maximum moment at pier from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination
Axial Force at SLS	$P_{sls} := -1680kip$	
Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto	$d_c := c_{t.t} + D_{N4} + \frac{D_{N7}}{2} = 3.438 \cdot in$	
Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7)	$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1.103$	
Tensile strain in nonprestressed reinforcement at SLS	$\varepsilon_s := 0.58 \cdot 10^{-3}$	

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss} := \varepsilon_s \cdot E_s = 16.824 \cdot ksi$$

Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7)

$$S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 21.409 \cdot in$$

Minimum Spacing for CIP Concrete (5.10.3.1.1)

$$S_{min.limit} := \max\left(1.5 \cdot D_{N7}, 1.5in, 1.5 \cdot a_g\right) = 1.5 \cdot in$$

4. Summary

Maximum Spacing in Design

$$S_{max} = 6.661 \cdot in$$

$S_{max}/S_{max.limit}=0.31$

Maximum Allowable Spacing

$$S_{max.limit} = 21.409 \cdot in$$

Maximum Spacing Check

$$if\left(S_{max} \le S_{max.limit}, "ok", "fail"\right) = "ok"$$

Minimum Allowable Spacing

$$S_{min.limit} = 1.5 \cdot in$$

$S_{min.limit}/S_{min}=0.25$

Minimum Spacing in Design

$$S_{min} = 5.875 \cdot in$$

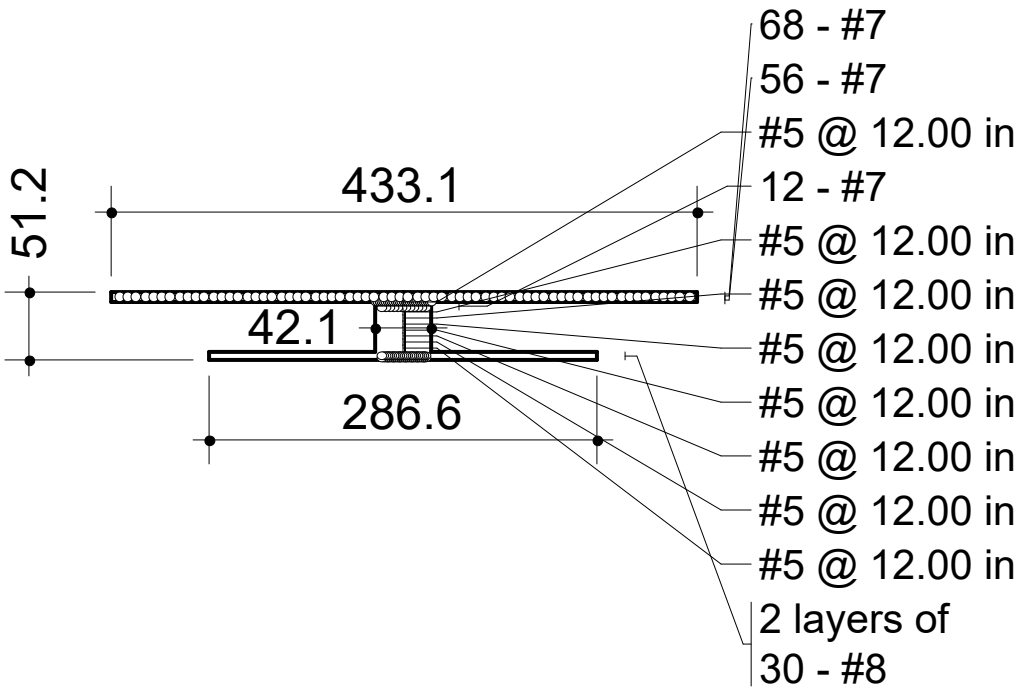
Minimum Spacing Check

$$if\left(S_{min} \ge S_{min.limit}, "ok", "fail"\right) = "ok"$$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6946.1	7854.4
Inertia (in^4)	2551466.0	2938023.0
y_t (in)	20.2	20.3
y_b (in)	31.0	30.9
S_t (in^3)	126613.8	144870.9
S_b (in^3)	82230.0	95082.6

Concrete Shrinkage/Thermal Strain -0.02 ms

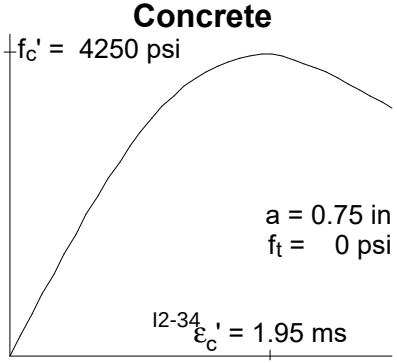


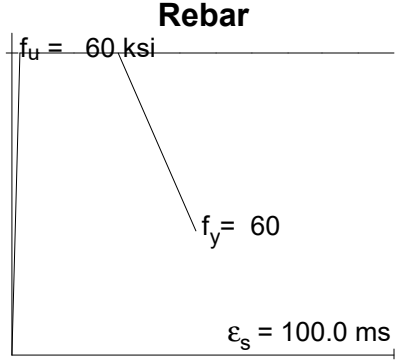
Crack Spacing

2 x dist + 0.1 d_b /ρ

Loading (N,M,V + dN,dM,dV)

-926 , -5679 , 0.0 + 0.0 , -1.0 , 0.0





All dimensions in inches
Clear cover to transverse reinforcement = 0.69 in

Enter Title Here

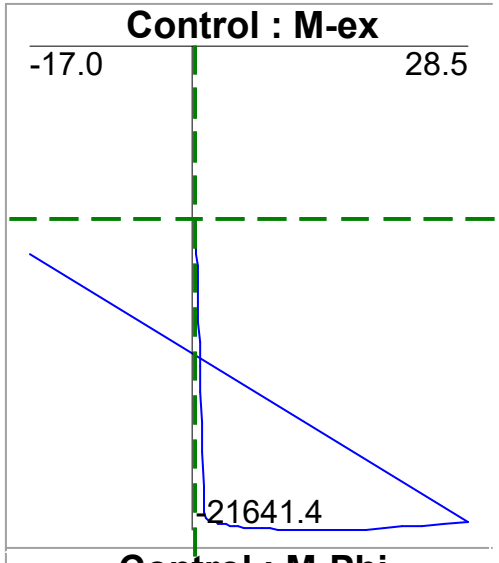
2021/2/10

Response-2000 v 1.0.5

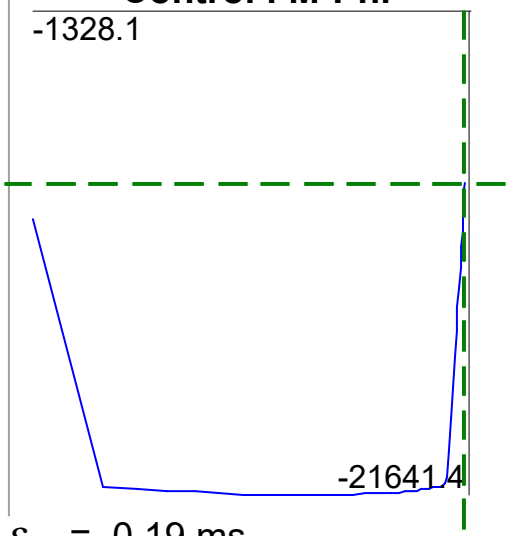
Enter Title Here

2021/10/26 - 2:04 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.19 ms

ϕ = -19.31 rad/10^6 in

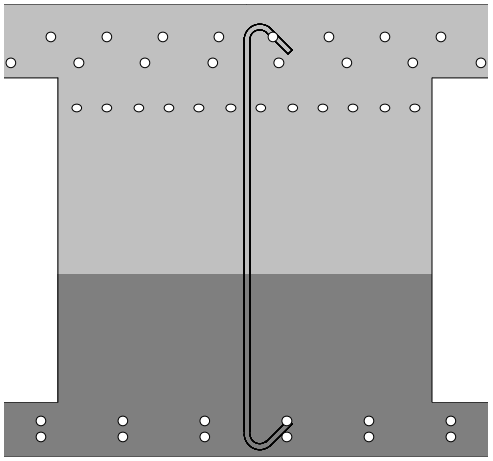
γ_{xy}(avg) = 0.00 ms

Axial Load = -1679.5 kips

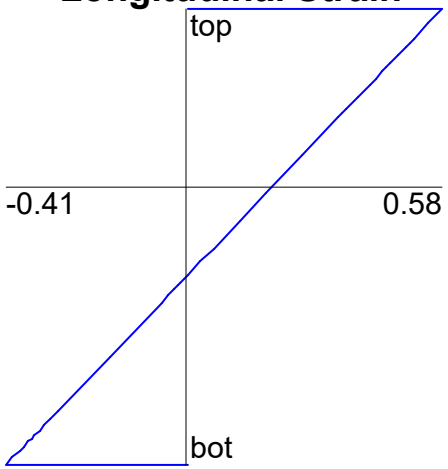
Moment:= -7715.7 ft-kips

Shear = 0.0 kips

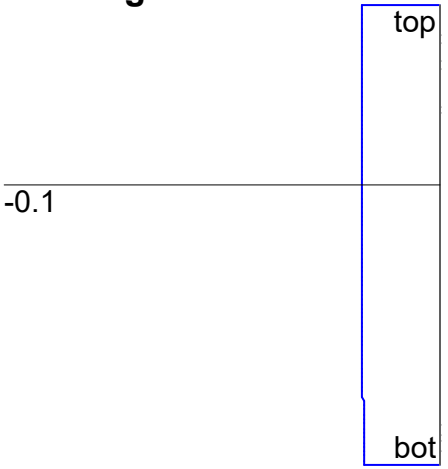
Cross Section



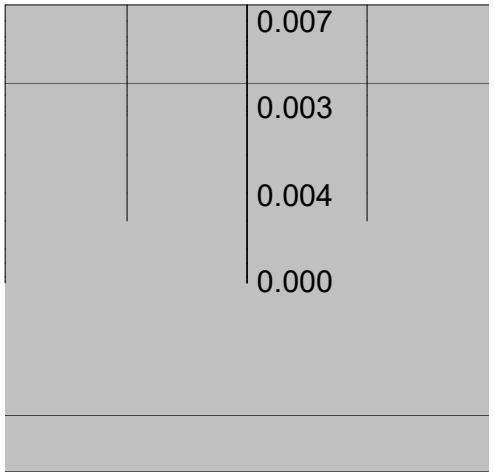
Longitudinal Strain



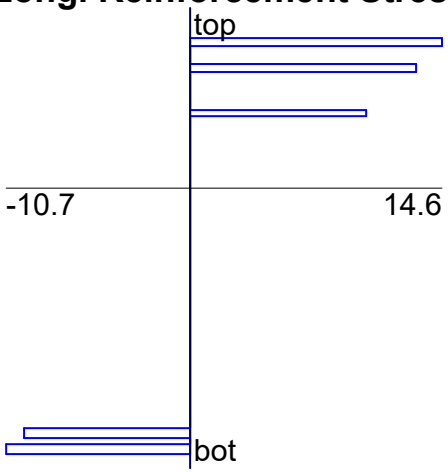
Shrinkage & Thermal Strain



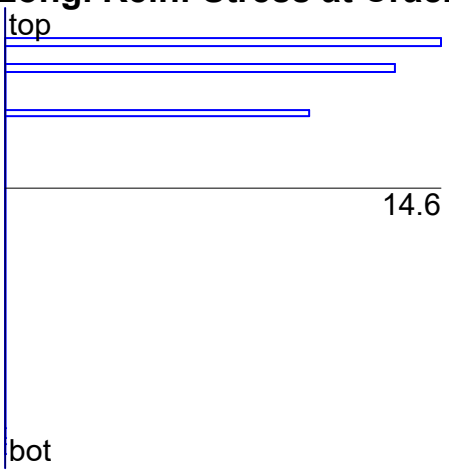
Crack Diagram



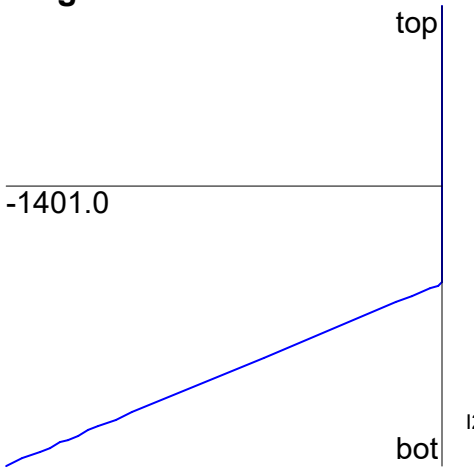
Long. Reinforcement Stress



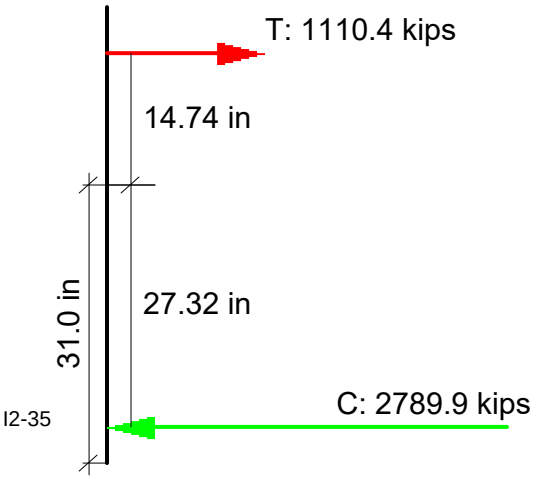
Long. Reinf Stress at Crack



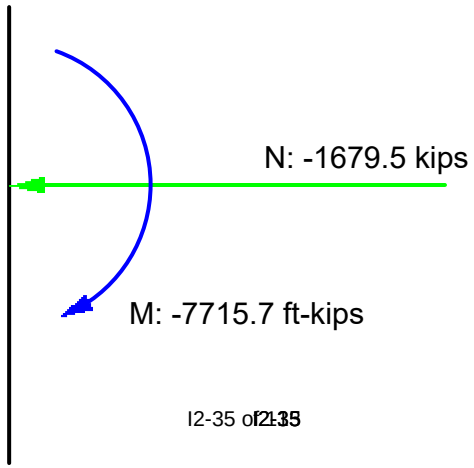
Longitudinal Concrete Stress



Internal Forces



N+M



1. Purpose and Assumptions

1. The purpose is to calculate and check the deformation and the reinforcement distribution for the service limit states in west span
2. The calculation is based on AASHTO 8th Edition, 2017

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}} \right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi} \right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft$
Bottom Width	$b_b := 24ft$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6in$
Web Thickness	$b_w := (10in \cdot 2 + 9in \cdot 2) = 3.167 \cdot ft$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 123.83ft^4$
Vertical Distance from Top of Deck to Centroid of Cross-section	$y := 1.61ft$
Cross-section Area	$A := 51.842ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$

West Span Length $L_{west} := 76.295ft$

East Span Length $L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #10 $D_{N8} := 1in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

Young's Modulus of Reinforcement $E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	40	$A_{s.1} := 40 \cdot 0.6in^2 = 24 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	30	$A_{s.2} := 30 \cdot 0.6in^2 = 18 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	36	$A_{s.8} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	36	$A_{s.9} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2} \right) = 47 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N8}}{2} \right) = 49 \cdot in$

Maximum Spacing in the Layer Closest to the Tension Face $S_{max} := \frac{(3ft + 8.375in)}{4} = 11.094 \cdot in$

Minimum Spacing in the Layer Closest to the Tension Face $S_{min} := \frac{(1ft + 5.375in)}{4} = 4.344 \cdot in$

3. Reinforcement Distribution

Exposure Condition: Class 2

Exposure Factor (5.6.7) $\gamma_e := 0.75$

Aggregate Size $a_g := 0.75in$ Note: as per AASHTO C5.4.2.1

Positive Moment at SLS $M_{sls} := 4157kip \cdot ft$ Note: maximum moment at west span from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination

Axial Force at SLS $P_{sls} := -1686kip$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_c := c_b + D_{N4} + \frac{D_{N8}}{2} = 2 \cdot in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1.058$

Tensile strain in nonprestressed reinforcement at SLS $\varepsilon_s := 0.35 \cdot 10^{-3}$ Note: From Response 2000 Tensile Stress in Nonprestressed Reinforcement at SLS

Tensile stress in nonprestressed reinforcement at SLS $f_{ss} := \varepsilon_s \cdot E_s = 10.153 \cdot ksi$

Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7) $S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 44.862 \cdot in$

Minimum Spacing for CIP Concrete (5.10.3.1.1)

$$S_{min.limit} := \max\left(1.5 \cdot D_{N8}, 1.5in, 1.5 \cdot a_g\right) = 1.5 \cdot in$$

Maximum Allowable Spacing (5.10.3.2)

$$S_{max.limit} := S_{max.limit} = 44.862 \cdot in$$

4. Deformations

Deflection due to Vehicle and Pedestrian Load

$$\Delta := 0.0166ft = 0.199 \cdot in$$

Note: deflection is the maximum absolute deflection from csibridge load combination "LL_Sup_Def"

Cracking Moment (5.6.3.5.2)

$$M_{cr} := f_r \cdot \frac{I_{22}}{h - y} = 3.625 \times 10^3 \cdot kip \cdot ft$$

Maximum Moment in a Component at the Stage for which Deformation is Computed

$$M_a := 4257kip \cdot ft$$

Note: M.sls is from csibridge. It's the maximum moment in mid span in load combintation 'SE1-1 SUP_DEF_ENV'

Axial Force in a Component at the Stage for which Deformation is Computed

$$P_a := -1609kip$$

Curvature at Ma (from Response 2000)

$$\rho := \frac{11.41}{10^6} \frac{rad}{in}$$

Effective Moment of Inertia (5.6.3.5.2)

$$I_{eff} := \frac{M_a}{E_c \cdot \rho} = 9.13 \times 10^5 \cdot in^4$$

Adjustment Factor for Deflection

$$\alpha := \frac{I_{eff}}{I_{22}} = 0.356$$

Adjusted Deflection due to Vehicle and Pedestrian Load

$$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.56 \cdot in$$

Maximum Allowable Deflection (2.5.2.6.2)

$$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$$

5. Summary

Maximum Spacing in Design

$$S_{max} = 11.094 \cdot in$$

Maximum Allowable Spacing

$$S_{max.limit} = 44.862 \cdot in$$

$$\frac{(S_{max})}{S_{max.limit}} = 0.247$$

SLS Check (W) PT removed from

Maximum Spacing Check

$\text{if}\left(S_{max} \le S_{max.limit}, \text{"ok"}, \text{"fail"}\right) = \text{"ok"}$

Minimum Spacing in Design

$S_{min} = 4.344 \cdot \text{in}$

$\frac{\left(S_{min.limit}\right)}{S_{min}} = 0.345$

Minimum Allowable Spacing

$S_{min.limit} = 1.5 \cdot \text{in}$

Minimum Spacing Check

$\text{if}\left(S_{min} \ge S_{min.limit}, \text{"ok"}, \text{"fail"}\right) = \text{"ok"}$

Adjusted Deflection due to Vehicle and Pedestrian Load

$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.56 \cdot \text{in}$

$\frac{\Delta_{adjusted}}{\Delta_{max.limit}} = 0.609$

Maximum Allowable Deflection (2.5.2.6.2)

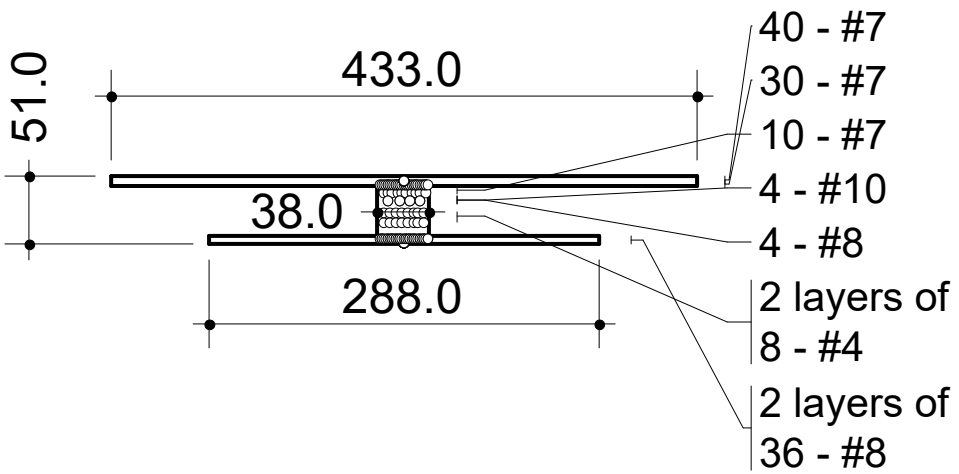
$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot \text{in}$

Deflection Check

$\text{if}\left(\Delta_{adjusted} \le \Delta_{max.limit}, \text{"ok"}, \text{"fail"}\right) = \text{"ok"}$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6795.5	7614.6
Inertia (in^4)	2512262.9	2891210.2
y_t (in)	20.0	20.8
y_b (in)	31.0	30.2
S_t (in^3)	125818.3	138916.1
S_b (in^3)	80955.6	95775.5



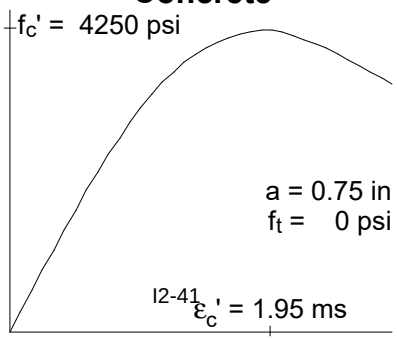
Crack Spacing

2 x dist + 0.1 d_b /ρ

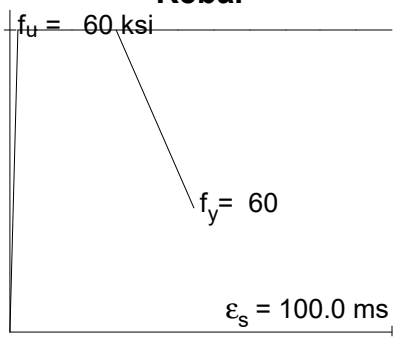
Loading (N,M,V + dN,dM,dV)

-1686 , 4988 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.50 in

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2021/6/16

Response-2000 v 1.0.5

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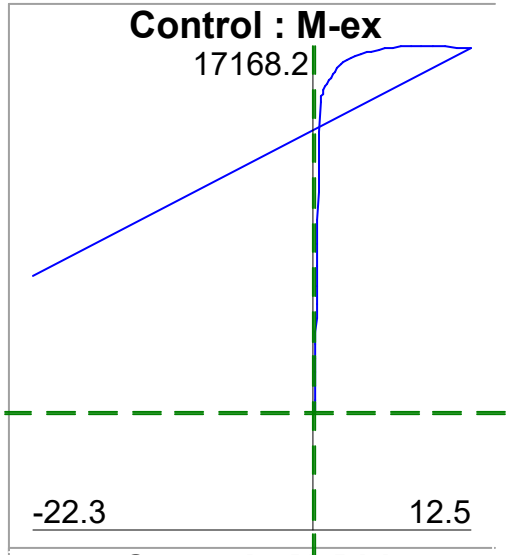
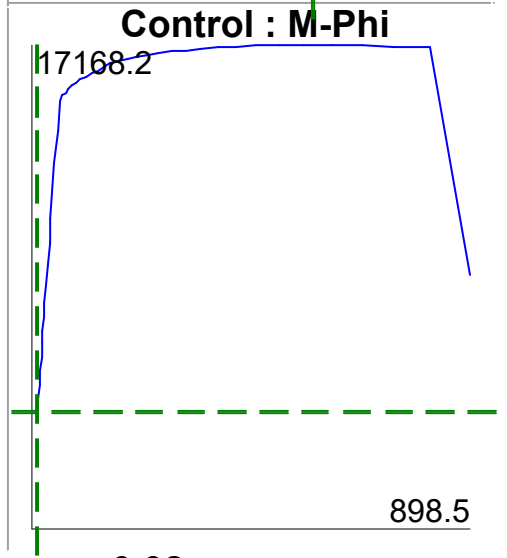
2021/10/26 - 1:21 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.02 ms

ϕ = 10.74 rad/10^6 in

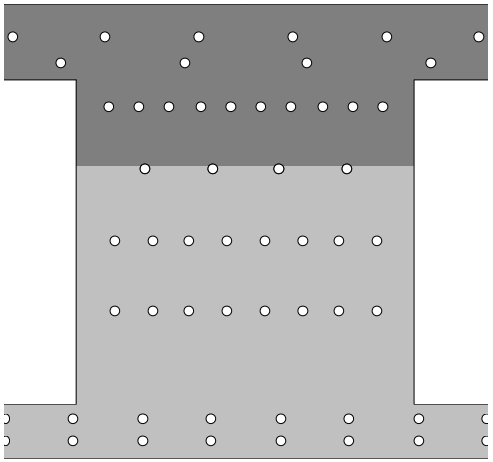
γ_{xy}(avg) = 0.00 ms

Axial Load = -1685.1 kips

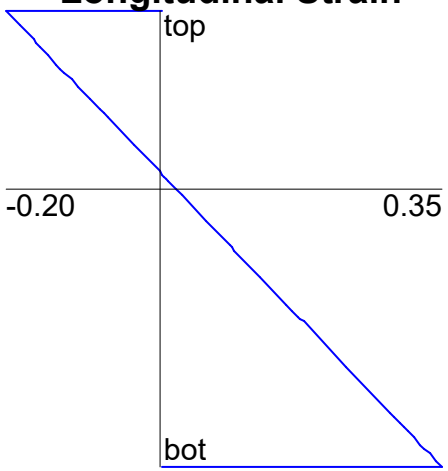
Moment:= 4151.7 ft-kips

Shear = 0.0 kips

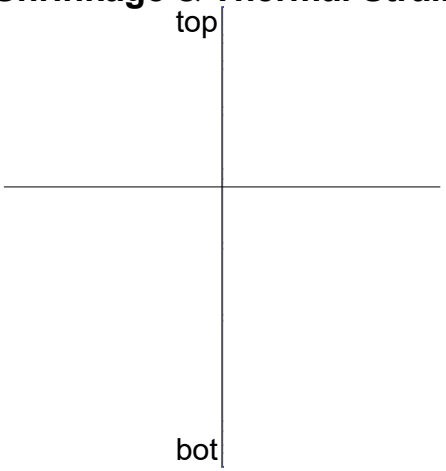
Cross Section



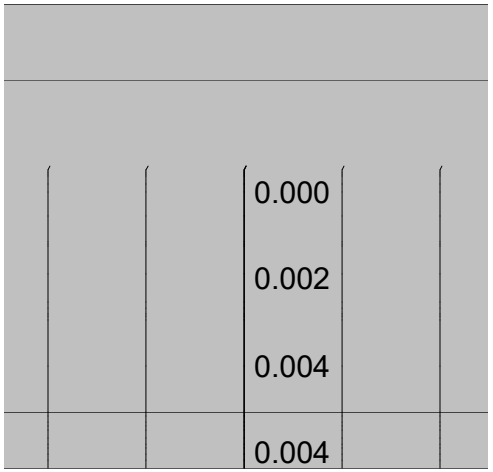
Longitudinal Strain



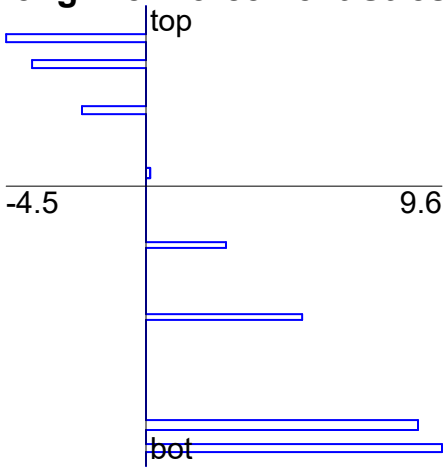
Shrinkage & Thermal Strain



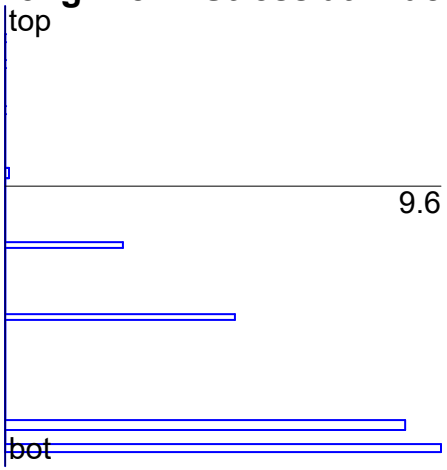
Crack Diagram



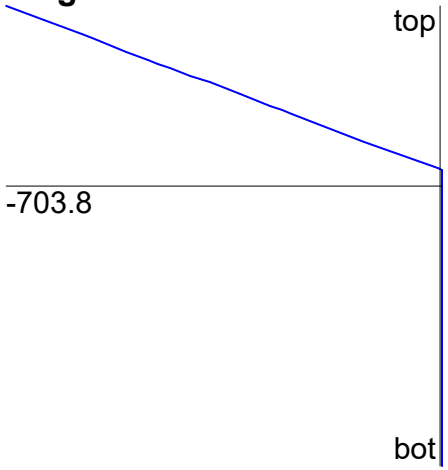
Long. Reinforcement Stress



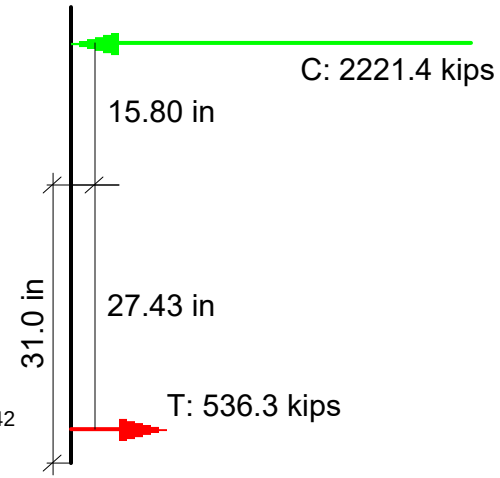
Long. Reinf Stress at Crack



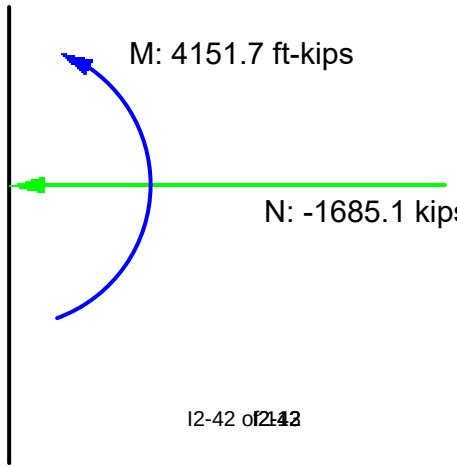
Longitudinal Concrete Stress

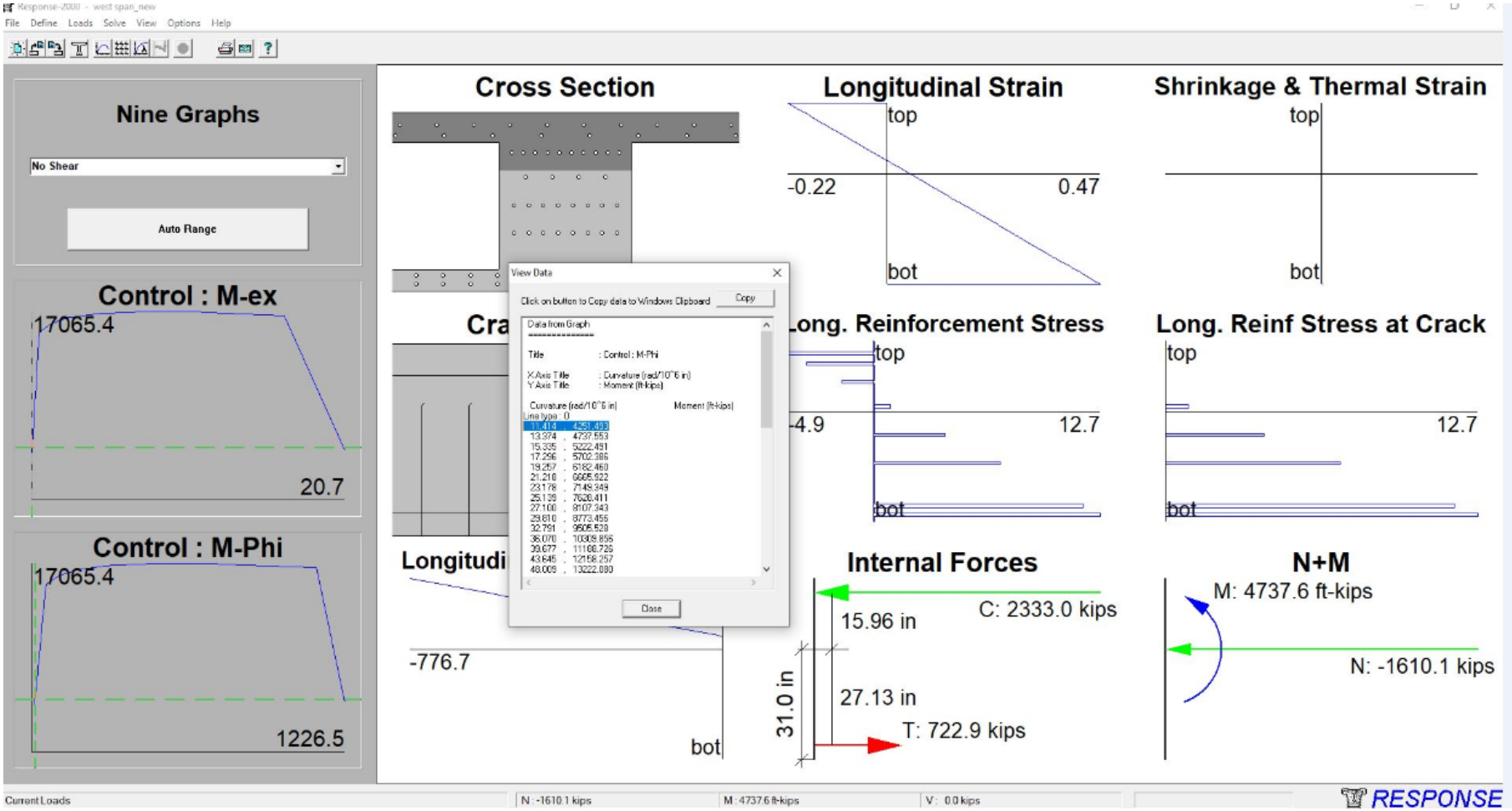


Internal Forces



N+M




Response-2000 - west span_new
File Define Loads Solve View Options Help
Nine Graphs
No Shear
Auto Range
Control : M-ex
17065.4
20.7
Control : M-Phi
17065.4
1226.5
Cross Section
Longitudinal Strain
top
-0.22
0.47
bot
Shrinkage & Thermal Strain
top
bot
View Data
Click on button to Copy data to Windows Clipboard
Copy
Data from Graph
Title : Control : M-Phi
X-Axis Title : Curvature (rad/10^6 in)
Y-Axis Title : Moment (ft-kips)
Curvature (rad/10^6 in) Moment (ft-kips)
Line type : 0
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Long. Reinforcement Stress
top
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bot
Long. Reinf Stress at Crack
top
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bot
Longitudi
-776.7
bot
Internal Forces
C: 2333.0 kips
15.96 in
31.0 in
27.13 in
T: 722.9 kips
N+M
M: 4737.6 ft-kips
N: -1610.1 kips
Current Loads
N : -1610.1 kips
M : 4737.6 ft-kips
V : 0.0 kips
RESPONSE

1. Purpose and Assumptions

1. The purpose is to calculate and check the deformation and the reinforcement distribution for the service limit states at the interface of existing and proposed structure
2. The calculation is based on AASHTO 8th Edition, 2017

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi}\right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft$
Bottom Width	$b_b := 24ft$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6in$
Web Thickness	$b_w := (10in \cdot 2 + 9in \cdot 2) = 3.167 \cdot ft$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 123.83ft^4$
Vertical Distance from Top of Deck to Centroid of Cross-section	$y := 1.61ft$
Cross-section Area	$A := 51.842ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$

West Span Length	$L_{west} := 76.295ft$
East Span Length	$L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab	$c_{t.t} := 2.5in$
Top Cover for Bottom Slab	$c_{t.b} := 1in$
Bottom Cover for Top/Bottom Slab	$c_b := 1in$
Diameter of #10	$D_{N8} := 1in$
Diameter of #7	$D_{N7} := 0.875in$
Diameter of #5	$D_{N5} := 0.625in$
Diameter of #4	$D_{N4} := 0.5in$
Vertical Reinforcement Spacing in Web	$s_w := 8in$
Young's Modulus of Reinforcement	$E_{sv} := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	40	$A_{s.1} := 40 \cdot 0.6in^2 = 24 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	30	$A_{s.2} := 30 \cdot 0.6in^2 = 18 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	30	$A_{s.8} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	30	$A_{s.9} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2}\right) = 47 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N8}}{2}\right) = 49 \cdot in$

Maximum Spacing in the Layer Closest to the Tension Face $S_{max} := \frac{(3ft + 8.375in)}{4} = 11.094 \cdot in$

Minimum Spacing in the Layer Closest to the Tension Face $S_{min} := \frac{(1ft + 5.375in)}{4} = 4.344 \cdot in$

3. Reinforcement Distribution

Note: 5.10.3.1.2 is only for slab on girder bridge, but in this case, we have a box girder birdge with no slabs, so section 5.10.3.1.2 does not apply. Therefore, this calculation only check the maximum spacing based on 5.6.7

Exposure Condition: Class 2

Exposure Factor (5.6.7) $\gamma_e := 0.75$

Aggregate Size $a_g := 0.75in$ Note: as per AASHTO C5.4.2.1

Negative Moment at SLS $M_{sls} := 1231kip \cdot ft$ Note: maximum moment at east span from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination

Axial Force at SLS $P_{sls} := -1555kip$

From Response 2000, the entire cross-section is in compression (results attached in the end) . As per AASHTO 5.6.7, the maximum allowable spacing is infinite large. Therefore, the design passes

Minimum Spacing for CIP Concrete (5.10.3.1.1) $S_{min.limit} := \max\left(1.5 \cdot D_{N8}, 1.5in, 1.5 \cdot a_g\right) = 1.5 \cdot in$

4. Deformations

Deflection due to Vehicle and Pedestrian Load

$\Delta := 0.00845\,ft = 0.101\cdot in$

Note: deflection is the maximum absolute deflection from csibridge load combination "LL_Sup_Def"

Cracking Moment (5.6.3.5.2)

$M_{cr} := f_r \cdot \frac{I_{22}}{h - y} = 3.625 \times 10^3 \cdot kip \cdot ft$

Maximum Moment in a Component at the Stage for which Deformation is Computed

$M_a := 899\,kip \cdot ft$

Note: M.sls is from csibridge. It's the maximum moment in mid span in load combintation 'SE1-1 SUP_DEF_ENV'

Axial Forcein a Component at the Stage for which Deformation is Computed

$P_a := -1571\,kip$

Curvature at Ma (from Response 2000)

$\rho := \frac{1.144}{10^6}\,\frac{rad}{in}$

Effective Moment of Inertia (5.6.3.5.2)

$I_{eff} := \frac{M_a}{E_c \cdot \rho} = 1.923 \times 10^6 \cdot in^4$

Adjustment Factor for Deflection

$\alpha := \frac{I_{eff}}{I_{22}} = 0.749$

Adjusted Deflection due to Vehicle and Pedestrian Load

$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.135 \cdot in$

Maximum Allowable Deflection (2.5.2.6.2)

$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$

5. Summary

Maximum Spacing in Design

$S_{max} = 11.094 \cdot in$

Maximum Allowable Spacing is infinite large based upon AASHTO 5.6.7-> OK

Minimum Spacing in Design

$S_{min} = 4.344 \cdot in$

$\frac{(S_{min.limit})}{S_{min}} = 0.345$

Minimum Allowable Spacing

$S_{min.limit} = 1.5 \cdot in$

Minimum Spacing Check

$if(S_{min} \geq S_{min.limit}, "ok", "fail") = "ok"$

Adjusted Deflection due to Vehicle and Pedestrian Load

$$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.135 \cdot in$$

$$\frac{\Delta_{adjusted}}{\Delta_{max.limit}} = 0.147$$

Maximum Allowable Deflection (2.5.2.6.2)

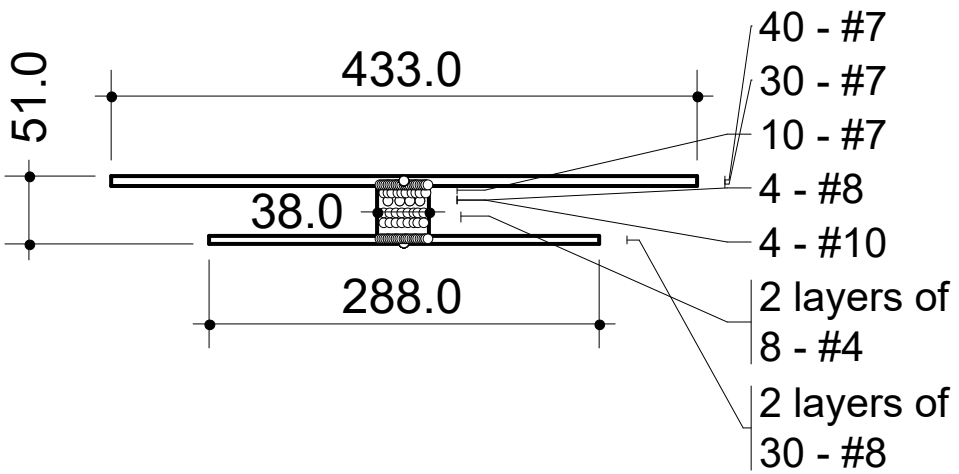
$$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$$

Deflection Check

$$if\left(\Delta_{adjusted} \le \Delta_{max.limit}, "ok", "fail"\right) = "ok"$$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6795.5	7547.8
Inertia (in^4)	2512262.9	2842240.7
y_t (in)	20.0	20.6
y_b (in)	31.0	30.4
S_t (in^3)	125818.3	138144.5
S_b (in^3)	80955.6	93416.1



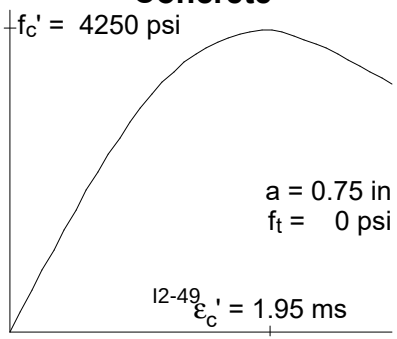
Crack Spacing

2 x dist + 0.1 d_b /ρ

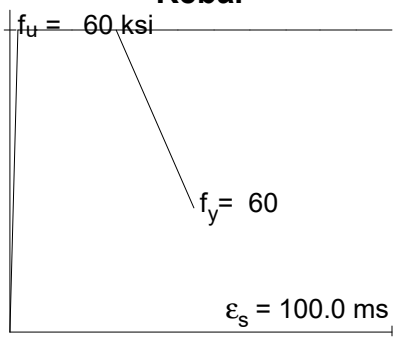
Loading (N,M,V + dN,dM,dV)

-1483 , 1083 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.50 in

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2021/6/16

Response-2000 v 1.0.5

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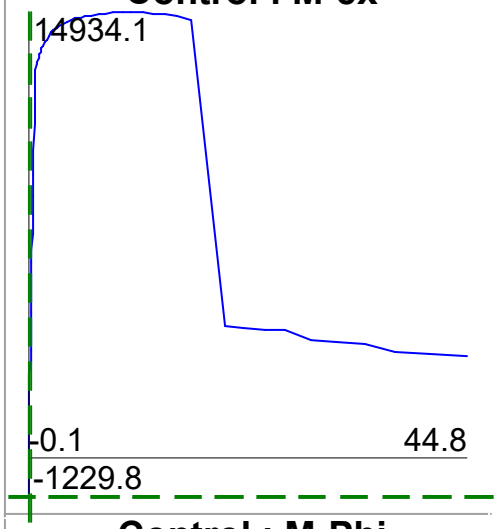
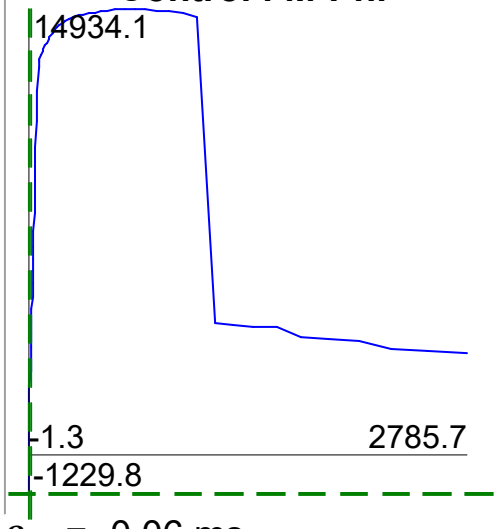
2021/10/26 - 1:35 pm

Control : M-ex



Control : M-Phi



ε_{x0} = -0.06 ms

ϕ = -1.35 rad/10^6 in

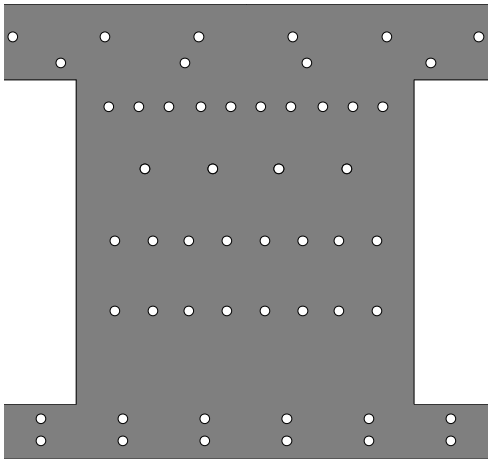
γ_{xy}(avg) = 0.00 ms

Axial Load = -1553.5 kips

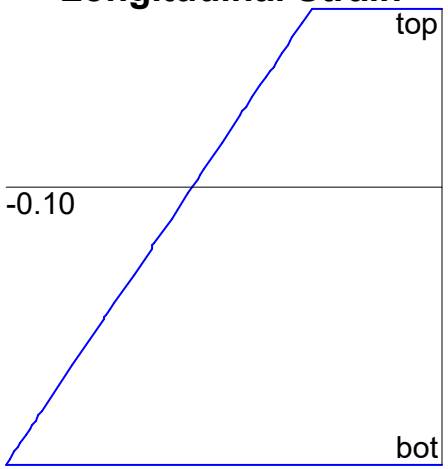
Moment:= -1229.8 ft-kips

Shear = 0.0 kips

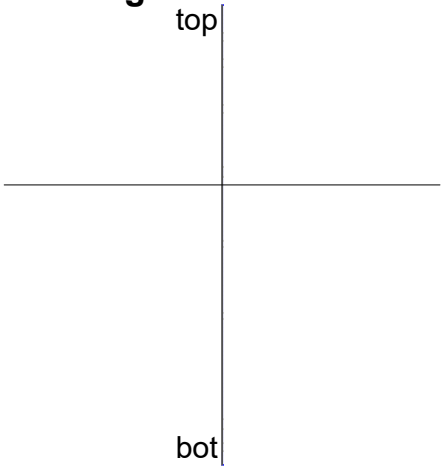
Cross Section



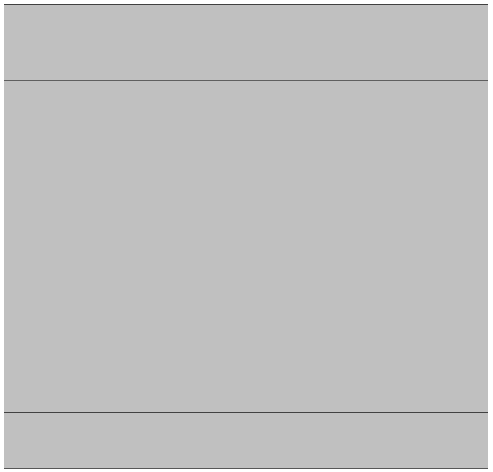
Longitudinal Strain



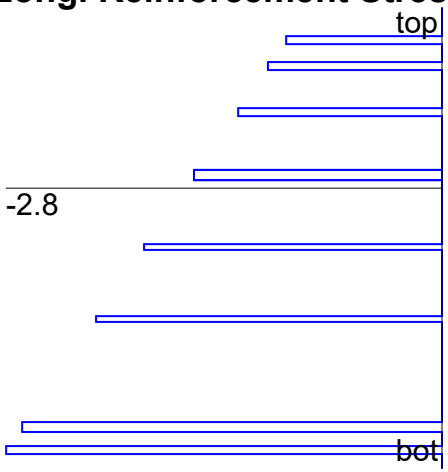
Shrinkage & Thermal Strain



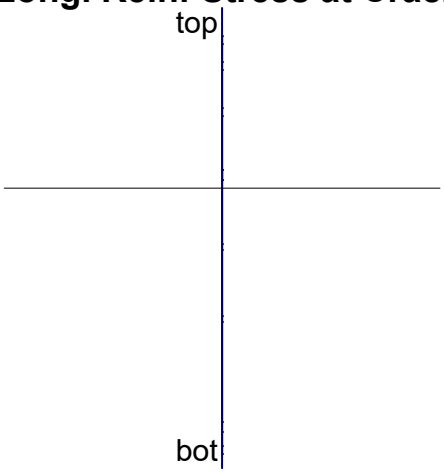
Crack Diagram



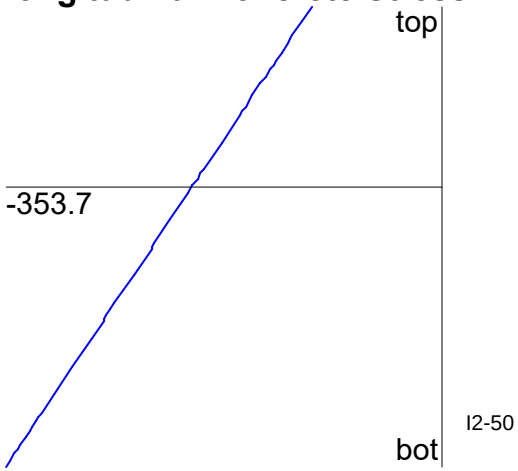
Long. Reinforcement Stress



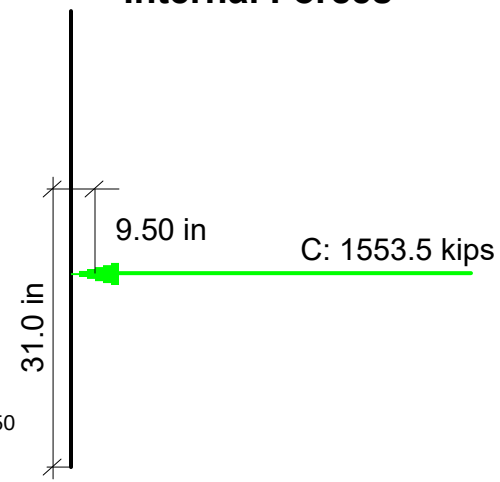
Long. Reinf Stress at Crack



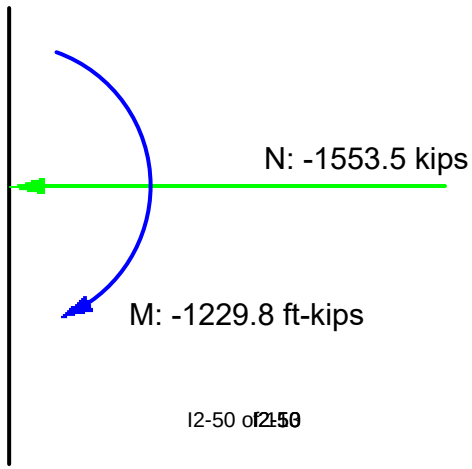
Longitudinal Concrete Stress

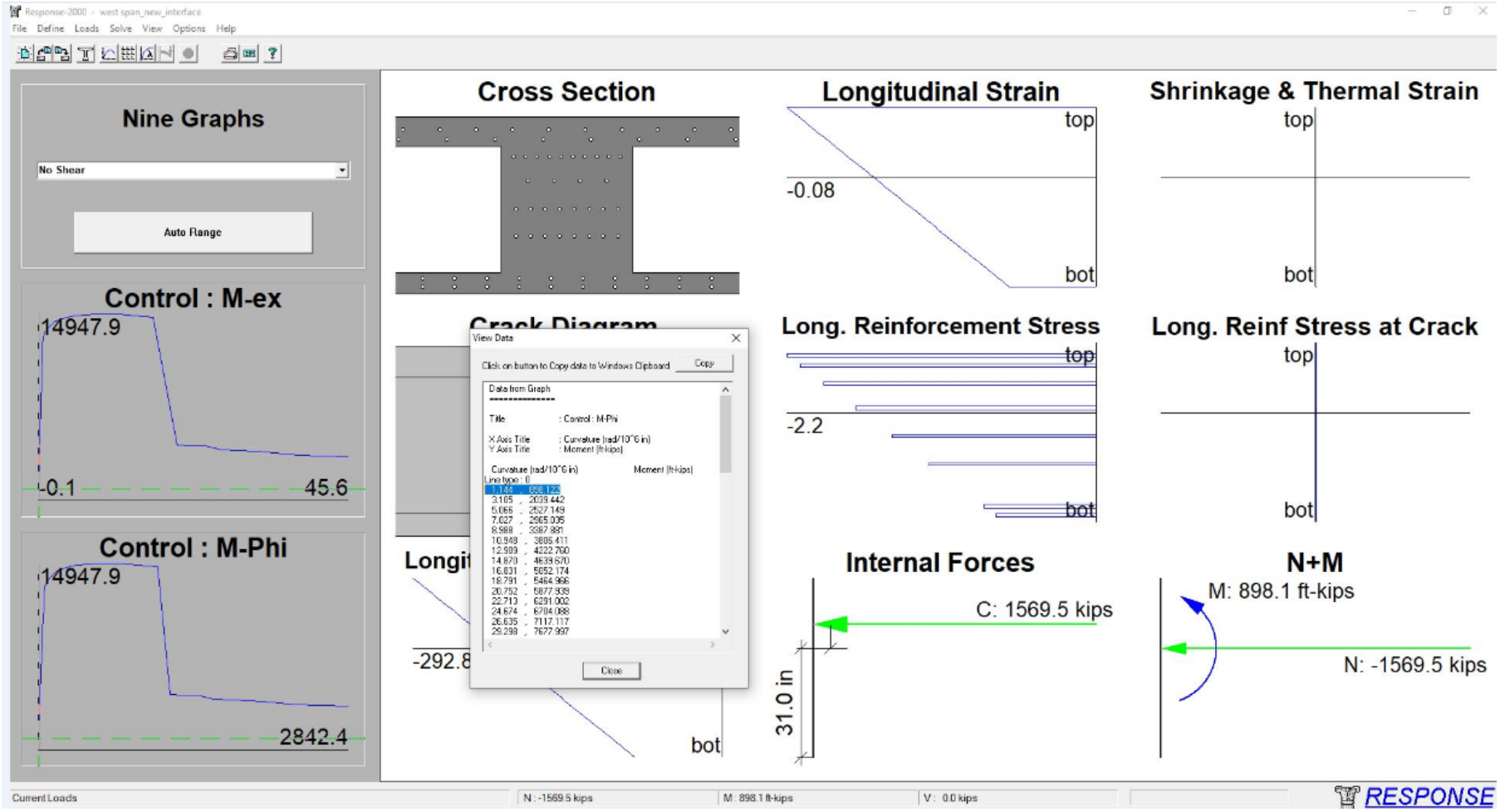


Internal Forces



N+M




Response-2000 - west span_new_interface
File Define Loads Solve View Options Help
Nine Graphs
No Shear
Auto Range
Control : M-ex
14947.9
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Control : M-Phi
14947.9
2842.4
Cross Section
Longitudinal Strain
top
-0.08
bot
Shrinkage & Thermal Strain
top
bot
Crack Diagram
View Data
Click on button to Copy data to Windows Clipboard
Copy
Data from Graph
Title : Control : M-Phi
X Axis Title : Curvature (rad/10^6 in)
Y Axis Title : Moment (ft-kips)
Curvature (rad/10^6 in) Moment (ft-kips)
Line type : 0
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8.988 , 3387.981
10.948 , 3805.411
12.909 , 4222.760
14.870 , 4639.670
16.831 , 5052.174
18.791 , 5464.966
20.752 , 5877.939
22.713 , 6291.002
24.674 , 6704.088
26.635 , 7117.117
28.298 , 7677.997
Close
Longi
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Long. Reinforcement Stress
top
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bot
Internal Forces
C: 1569.5 kips
31.0 in
N+M
M: 898.1 ft-kips
N: -1569.5 kips
Current Loads
N : -1569.5 kips
M : 898.1 ft-kips
V : 0.0 kips
RESPONSE

Interface Shear between Existing and New Structure

Description
The following calculations check the interface capacity between the existing and proposed extension. To be conservative, the typical section of the box girder was used in the calculation of the interface capacity even though the section between the existing and proposed extension is actually the solid diaphragm. The D/C ratio at the interface of the existing and proposed extension is 0.5.

1. Purpose and Assumptions

1. The purpose is to calculate and check the interface shear between the existing and proposed structure
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength $f_c := 5ksi$

Reinforcement Yield Strength $f_y := 60ksi$

3. Structural Properties

Total Web Thickness $b_w := 9in \cdot 2 + 11.18in \cdot 2 = 40.36 \cdot in$

Total Height $h := 4.25ft$

Mild Reinforcement

Diameter of #8 $D_{N8} := 1in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	40	$A_{s.1} := 40 \cdot 0.6in^2 = 24 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	30	$A_{s.2} := 30 \cdot 0.6in^2 = 18 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8	8	$A_{s.4} := 8 \cdot 0.79in^2 = 6.32 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2 \cdot in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2 \cdot in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2 \cdot in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	30	$A_{s.8} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	30	$A_{s.9} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$

$A_{total} := A_{s.1} + A_{s.2} + A_{s.3} + A_{s.4} + A_{s.5} + A_{s.6} + A_{s.7} + A_{s.8} + A_{s.9} = 116 \cdot in^2$

$T := A_{total} \cdot (f_y) = 6.96 \times 10^3 \cdot kip$

4. Forces at Interface

Factored shear force at interface

$V_u := 517kip$

Permanent net compressive force normal to the shear plane

$P_c := 0kip$

Note: Vu is the maximum shear force at interface for strength and extreme load combinations

Note: based on the results from "SUP_STRENGTH_ENVELOPE", the superstructure can be in compression at 996 kips, but to be conservative P.c is taken as zero.

5. Minimum Area of Interface Shear Reinforcement

Area of concrete considered to be engaged in interface shear transfer (in^2)
(5.7.4.2)

$A_{cv} := b_w \cdot h = 14.294 \cdot ft^2$

Minimum area of interface shear reinforcement crossing the shear plane within the area Acv (in^2)
(5.7.4.2)

$$A_{vf.min} := 0.05 \cdot \frac{A_{cv}}{\frac{f_y}{ksi}} = 1.715 \cdot in^2$$

Note: The area of concrete engaged in interface transfer is the total area of the web since the interface shear is mainly carried by the webs. In reality, the shear interface is in the diaphragm and the interface area is significant greater than what is used here, so the conclusion is extremely conservative.

Area of interface shear reinforcement crossing the shear plane within the area Acv (in^2)

$A_{vf} := A_{s.4} + A_{s.5} + A_{s.6} + A_{s.7} = 11.12 \cdot in^2$

Minimum reinforcement check

$\text{if}\left(A_{vf} \le A_{vf.min}, \text{"fail"}, \text{"ok"}\right) = \text{"ok"}$

6. Interface Shear Resistance

Assumed the interface is roughened as per Section 5.3.1 (C) in BDM

Cohesion factor
(5.7.4.3)

$c := 0.24ksi$

Friction factor
(5.7.4.3)

$\mu := 1$

Fraction of concrete strenght available to resist interface shear
(5.7.4.3)

$K_1 := 0.25$

Limiting interface shear resistance
(5.7.4.3)

$K_2 := 1.5ksi$

Nominal shear resistance of interface plane
(5.7.4.3)

$V_{ni} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 1.161 \times 10^3 \cdot kip$

The nominal shear resistance used in the design shall not exceed either of the following:

$$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = 2.573 \times 10^3 \cdot kip$$

$$V_{ni.2} := K_2 \cdot A_{cv} = 3.088 \times 10^3 \cdot kip$$

$$V_{ni.limit} := \min\left(V_{ni.1}, V_{ni.2}\right) = 2.573 \times 10^3 \cdot kip$$

Check for shear force resistance

$$V_{ni} := \begin{cases} V_{ni} & \text{if } V_{ni} \le V_{ni.limit} \\ V_{ni.limit} & \text{if } V_{ni} > V_{ni.limit} \end{cases} = 1.161 \times 10^3 \cdot kip$$

Shear force resistance factor (5.5.4.2)

$$\phi := 0.9$$

7. Summary

Factored Interface shear resistance

$$V_{ri} := V_{ni} \cdot \phi = 1.045 \times 10^3 \cdot kip$$

Total reinforcement area

$$A_{vf} = 11.12 \cdot in^2$$

Minimum reinforcement area

$$A_{vf.min} = 1.715 \cdot in^2$$

Factored interface shear demand

$$V_u = 517 \cdot kip$$

Factored Interface shear resistance

$$V_{ri} := V_{ni} \cdot \phi = 1.045 \times 10^3 \cdot kip$$

D/C ratio

$$\frac{V_u}{V_{ri}} = 0.495$$

Interface Shear between Web and Slabs

Description
The following calculations check the interface capacity between the interface shear capacity between the slabs and the top and bottom of the web. The check was conducted at critical section located at dv away from the east abutment (Pier 3), intermediate pier (Pier 2), and west abutment (Pier 1). The maximum D/C ratio at the interface of the web and the adjacent slab is 0.7 at the dv away from the Intermediate (Pier 2)

1. Purpose and Assumptions

1. The purpose is to calculate and check the interface shear at the top and bottom of the web at d.v away from the edge of the east abutment support
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength $f_c := 5ksi$

Reinforcement Yield Strength $f_y := 60ksi$

3. Structural Properties

Total Web Thickness $b_w := 9in \cdot 2 + 11.18in \cdot 2 = 40.36 \cdot in$

Mild Reinforcement

Diameter of #5 Bars $D_{n5} := 0.625in$

Area of #5 Bar $A_{n5} := 0.31in^2$

Total Number of #5 Transverse Bars $N_{tb} := 8$

Spacing of Transverse Reinforcement in Exteior Web $s_{ext} := 6in$

Spacing of Transverse Reinforcement in Interior Web $s_{int} := 8in$

Interface length considered to be engaged in shear transfer $L_{vi} := 1ft$

Area of Transverse Reinforcement within a Distance L_{vi}

$$A_v := \left[\left(\frac{N_{tb}}{2} \cdot \frac{A_{n5}}{s_{int}}\right) + \frac{\left(\frac{N_{tb}}{2} \cdot A_{n5}\right)}{s_{ext}}\right] \cdot L_{vi} = 4.34 \cdot in^2$$

4. Forces at Interface

Factored shear force $V_u := 688kip$

Permanent net compressive force normal to the shear plane $P_c := 0kip$

Note: Vu is the maximum shear force at ~3.625 ft away from the east abutment support edge. It's obtained from csibridge load combination 'SUP_STRENGTH_ENVELOPE' at station 3.45 ft

5. Minimum Area of Interface Shear Reinforcement

Area of concrete considered to be engaged in interface shear transfer (in^2)
(5.7.4.2)

$A_{cv} := b_w \cdot L_{vi} = 3.363 \cdot ft^2$

Minimum area of interface shear reinforcement crossing the shear plane within the area Acv (in^2) (5.7.4.2)

$$A_{vf.min} := 0.05 \cdot \frac{A_{cv}}{\frac{f_y}{ksi}} = 0.404 \cdot in^2$$

Area of interface shear reinforcement crossing the shear plane within the area Acv (in^2)

$$A_{vf} := A_v = 4.34 \cdot in^2$$

Minimum reinforcement check

$$if\left(A_{vf} \le A_{vf.min}, "fail", "ok"\right) = "ok"$$

6. Interface Shear Resistance

Cohesion factor (5.7.4.3)

$$c := 0.28ksi$$

Friction factor (5.7.4.3)

$$\mu := 1$$

Fraction of concrete strenght available to resist interface shear (5.7.4.3)

$$K_1 := 0.3$$

Limiting interface shear resistance (5.7.4.3)

$$K_2 := 1.8ksi$$

Nominal shear resistance of interface plane (5.7.4.3)

$$V_{ni} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 396.01 \cdot kip$$

The nominal shear resistance used in the design shall not exceed either of the following:

$$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = 726.48 \cdot kip$$

$$V_{ni.2} := K_2 \cdot A_{cv} = 871.776 \cdot kip$$

$$V_{ni.limit} := \min\left(V_{ni.1}, V_{ni.2}\right) = 726.48 \cdot kip$$

Check for shear force resistance

$$V_{ni} := \begin{cases} V_{ni} & \text{if } V_{ni} \le V_{ni.limit} \\ V_{ni.limit} & \text{if } V_{ni} > V_{ni.limit} \end{cases} = 396.01 \cdot kip$$

Shear force resistance factor (5.5.4.2)

$$\phi := 0.9$$

Factored Interface shear resistance

$$V_{ri} := V_{ni} \cdot \phi = 356.409 \cdot kip$$

7. Factored Interface Shear Force Demands

Method 1 based upon AASHTO 5.7.4.5

Vertical distance from the top of the deck to centroid of tensile bars (Layer 1)

$d_{s.8} := 46.635in$

Vertical distance from the the top of the deck to centroid of tensile bars (Layer 2)

$d_{s.9} := 48.865in$

Thickness of the top slab

$h_{ft} := 8.5in$

Distance between the centroid of the tension steel and the mid-thickness of the top slab

$$d_v := \left(\frac{d_{s.8} + d_{s.9}}{2}\right) - \frac{h_{ft}}{2} = 3.625 \cdot ft$$

Factored interface shear stress

$$v_{ui} := \frac{V_u}{b_w \cdot d_v} = 0.392 \cdot ksi$$

note: the demands will be the same between the interface shear at top of the web and bottom of the web

Factored interface shear force in kips/ft

$V_{ui} := v_{ui} \cdot A_{cv} = 189.793 \cdot kip$

Method 2 based on section properties

Calculation of Q for section above interface at top of the web

Centroid from Top

$y_t := 1.65ft$

Width of top flange

$b_t := 45ft$

Thickness of top flange

$t_t := 0.708ft$

First moment of area

$$Q_t := b_t \cdot t_t \cdot \left(y_t - \frac{t_t}{2}\right) = 1.169 \cdot m^3$$

Moment of inertia

$I_{22} := 127.82ft^4$

Factored interface shear stress

$$v_{ui2t} := V_u \cdot \frac{Q_t}{I_{22} \cdot b_w} = 0.459 \cdot ksi$$

note: method 2 governs

Factored interface shear force in kips/ft

$V_{ui2t} := v_{ui2t} \cdot A_{cv} = 222.249 \cdot kip$

Method 2 based on section properties

Calculation of Q for section below interface at bottom of the web

Centroid from Bottom

$y_b := 4.25ft - 1.65ft = 2.6 \cdot ft$

Width of bottom flange

$b_b := 32.6ft$

Thickness of bottom flange

$t_b := 6.5in$

First moment of area

$$Q_b := b_b \cdot t_b \cdot \left(y_b - \frac{t_b}{2}\right) = 41.129 \cdot ft^3$$

Moment of inertia

$I_{22} := 127.82\,ft^4$

Factored interface shear stress

$v_{ui2b} := V_u \cdot \frac{Q_b}{I_{22} \cdot b_w} = 0.457 \cdot ksi$

Factored interface shear force in kips/ft

$V_{ui2b} := v_{ui2b} \cdot A_{cv} = 221.381 \cdot kip$

8. Summary

Total reinforcement area

$A_{vf} = 4.34 \cdot in^2$

Minimum reinforcement area

$A_{vf.min} = 0.404 \cdot in^2$

Does the interface has minimum reinforcement area?

$check_{minrein} := if\left(A_{vf.min} > A_{vf}, "no", "yes"\right) = "yes"$

Factored interface shear demand

$\max\left(V_{ui}, V_{ui2t}, V_{ui2b}\right) = 222.249 \cdot kip$

Factored Interface shear resistance

$V_{ri} := V_{ni} \cdot \phi = 356.409 \cdot kip$

D/C ratio

$\frac{\max\left(V_{ui}, V_{ui2t}, V_{ui2b}\right)}{V_{ri}} = 0.624$

1. Purpose and Assumptions

1. The purpose is to calculate and check the interface shear at the top and bottom of the web at dv away from the edge of the pier support
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength $f_c := 5ksi$

Reinforcement Yield Strength $f_y := 60ksi$

3. Structural Properties

Total Web Thickness $b_w := 9in \cdot 2 + 11.18in \cdot 2 = 40.36 \cdot in$

Mild Reinforcement

Diameter of #5 Bars $D_{n5} := 0.625in$

Area of #5 Bar $A_{n5} := 0.31in^2$

Total Number of #5 Transverse Bars $N_{tb} := 8$

Spacing of Transverse Reinforcement in Exteior Web $s_{ext} := 6in$

Spacing of Transverse Reinforcement in Interior Web $s_{int} := 8in$

Interface length considered to be engaged in shear transfer $L_{vi} := 1ft$

Area of Transverse Reinforcement within a Distance L_{vi}

$$A_v := \left[\left(\frac{N_{tb}}{2} \cdot \frac{A_{n5}}{s_{int}}\right) + \frac{\left(\frac{N_{tb}}{2} \cdot A_{n5}\right)}{s_{ext}}\right] \cdot L_{vi} = 4.34 \cdot in^2$$

4. Forces at Interface

Factored shear force $V_u := 886kip$

Note: Vu is the maximum shear force at ~3.562 ft away from pier. It's obtained from csibridge load combination 'SUP_STRENGTH_ENVELOPE' at station 74.63ft

Permanent net compressive force normal to the shear plane $P_c := 0kip$

5. Minimum Area of Interface Shear Reinforcement

Area of concrete considered to be engaged in interface shear transfer (in^2)
(5.7.4.2)

$$A_{cv} := b_w \cdot L_{vi} = 3.363 \cdot ft^2$$

Minimum area of interface shear reinforcement crossing the shear plane within the area Acv (in^2) (5.7.4.2)

$$A_{vf.min} := 0.05 \cdot \frac{A_{cv}}{\frac{f_y}{ksi}} = 0.404 \cdot in^2$$

Area of interface shear reinforcement crossing the shear plane within the area Acv (in^2)

$$A_{vf} := A_v = 4.34 \cdot in^2$$

Minimum reinforcement check

$$if\left(A_{vf} \le A_{vf.min}, "fail", "ok"\right) = "ok"$$

6. Interface Shear Resistance

Cohesion factor (5.7.4.3)

$$c := 0.28ksi$$

Friction factor (5.7.4.3)

$$\mu := 1$$

Fraction of concrete strenght available to resist interface shear (5.7.4.3)

$$K_1 := 0.3$$

Limiting interface shear resistance (5.7.4.3)

$$K_2 := 1.8ksi$$

Nominal shear resistance of interface plane (5.7.4.3)

$$V_{ni} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 396.01 \cdot kip$$

The nominal shear resistance used in the design shall not exceed either of the following:

$$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = 726.48 \cdot kip$$

$$V_{ni.2} := K_2 \cdot A_{cv} = 871.776 \cdot kip$$

$$V_{ni.limit} := \min\left(V_{ni.1}, V_{ni.2}\right) = 726.48 \cdot kip$$

Check for shear force resistance

$$V_{ni} := \begin{cases} V_{ni} & \text{if } V_{ni} \le V_{ni.limit} \\ V_{ni.limit} & \text{if } V_{ni} > V_{ni.limit} \end{cases} = 396.01 \cdot kip$$

Shear force resistance factor (5.5.4.2)

$$\phi := 0.9$$

Factored Interface shear resistance

$$V_{ri} := V_{ni} \cdot \phi = 356.409 \cdot kip$$

7. Factored Interface Shear Force Demands

Method 1 based upon AASHTO 5.7.4.5

Vertical distance from the extreme compressive fiber to centroid of tensile bars (Layer 1)

$d_{s.1} := 47.44in$

Vertical distance from the extreme compressive fiber to centroid of tensile bars (Layer 2)

$d_{s.2} := 44.56in$

Thickness of the bottom slab

$h_{fb} := 6.5in$

Distance between the centroid of the tension steel and the mid-thickness of the bottom slab

$$d_v := \left(\frac{d_{s.1} + d_{s.2}}{2}\right) - \frac{h_{fb}}{2} = 3.562 \cdot ft$$

Factored interface shear stress

$$v_{ui} := \frac{V_u}{b_w \cdot d_v} = 0.514 \cdot ksi$$

Factored interface shear force in kips/ft

$V_{ui} := v_{ui} \cdot A_{cv} = 248.702 \cdot kip$

note: the demands will be the same between the interface shear at top of the web and bottom of the web

Method 2 based on section properties

Calculation of Q for section above interface at top of the web

Centroid from Top

$y_t := 1.65ft = 1.65 \cdot ft$

Width of top flange

$b_t := 36.067ft$

Thickness of top flange

$t_t := 8.5in$

First moment of area

$$Q_t := b_t \cdot t_t \cdot \left(y_t - \frac{t_t}{2}\right) = 33.105 \cdot ft^3$$

Moment of inertia

$I_{22} := 127.82ft^4$

Factored interface shear stress

$$v_{ui2t} := V_u \cdot \frac{Q_t}{I_{22} \cdot b_w} = 0.474 \cdot ksi$$

note: method 1 governs

Factored interface shear force in kips/ft

$V_{ui2t} := v_{ui2t} \cdot A_{cv} = 229.473 \cdot kip$

Method 2 based on section properties

Calculation of Q for section below interface at bottom of the web

Centroid from Bottom

$y_b := 4.25ft - 1.65ft = 2.6 \cdot ft$

Width of bottom flange

$b_b := 24ft$

Thickness of bottom flange

$t_b := 6.5in$

First moment of area

$$Q_b := b_b \cdot t_b \cdot \left(y_b - \frac{t_b}{2}\right) = 30.279 \cdot ft^3$$

Moment of inertia $I_{22} := 127.82\,ft^4$

Factored interface shear stress $v_{ui2b} := V_u \cdot \frac{Q_b}{I_{22} \cdot b_w} = 0.433 \cdot ksi$ note: method 1 governs

Factored interface shear force in kips/ft $V_{ui2b} := v_{ui2b} \cdot A_{cv} = 209.884 \cdot kip$

8. Summary

Total reinforcement area $A_{vf} = 4.34 \cdot in^2$

Minimum reinforcement area $A_{vf.min} = 0.404 \cdot in^2$

Does the interface has minimum reinforcement area? $check_{minrein} := if\left(A_{vf.min} > A_{vf}, "no", "yes"\right) = "yes"$

Factored interface shear demand $V_{ui} = 248.702 \cdot kip$

Factored Interface shear resistance $V_{ri} := V_{ni} \cdot \phi = 356.409 \cdot kip$

D/C ratio $\frac{V_{ui}}{V_{ri}} = 0.698$

1. Purpose and Assumptions

1. The purpose is to calculate and check the interface shear at the top and bottom of webs at dv away from the edge of the support of pier
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength $f_c := 5ksi$

Reinforcement Yield Strength $f_y := 60ksi$

3. Structural Properties

Total Web Thickness $b_w := 9in \cdot 2 + 11.18in \cdot 2 = 40.36 \cdot in$

Mild Reinforcement

Diameter of #5 Bars $D_{n5} := 0.625in$

Area of #5 Bar $A_{n5} := 0.31in^2$

Total Number of #5 Transverse Bars $N_{tb} := 8$

Spacing of Transverse Reinforcement in Exteior Web $s_{ext} := 6in$

Spacing of Transverse Reinforcement in Interior Web $s_{int} := 8in$

Interface length considered to be engaged in shear transfer $L_{vi} := 1ft$

Area of Transverse Reinforcement within a Distance L_{vi}

$$A_v := \left[\left(\frac{N_{tb}}{2} \cdot \frac{A_{n5}}{s_{int}}\right) + \frac{\left(\frac{N_{tb}}{2} \cdot A_{n5}\right)}{s_{ext}}\right] \cdot L_{vi} = 4.34 \cdot in^2$$

4. Forces at Interface

Factored shear force $V_u := 816kip$

Permanent net compressive force normal to the shear plane $P_c := 0kip$

Note: Vu is the maximum shear force at ~3.583 ft away from pier. It's obtained from csibridge load combination 'SUP_STRENGTH_ENVELOPE' at station 80.54 ft

5. Minimum Area of Interface Shear Reinforcement

Area of concrete considered to be engaged in interface shear transfer (in^2)
(5.7.4.2)

$$A_{cv} := b_w \cdot L_{vi} = 3.363 \cdot ft^2$$

Maximum area of interface shear reinforcement crossing the shear plane within the area Acv (in^2) (5.7.4.2)

$$A_{vf.min} := 0.05 \cdot \frac{A_{cv}}{\frac{f_y}{ksi}} = 0.404 \cdot in^2$$

Area of interface shear reinforcement crossing the shear plane within the area Acv (in^2)

$$A_{vf} := A_v = 4.34 \cdot in^2$$

Minimum reinforcement check

$$if\left(A_{vf} \le A_{vf.min}, "fail", "ok"\right) = "ok"$$

6. Interface Shear Resistance

Assume cast-in-place concrete slab on clean concrete girder surfaces

Cohesion factor (5.7.4.4)

$$c := 0.28ksi$$

Friction factor (5.7.4.4)

$$\mu := 1$$

Fraction of concrete strenght available to resist interface shear (5.7.4.4)

$$K_1 := 0.3$$

Limiting interface shear resistance (5.7.4.4)

$$K_2 := 1.8ksi$$

Nominal shear resistance of interface plane (5.7.4.3)

$$V_{ni} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 396.01 \cdot kip$$

The nominal shear resistance used in the design shall not exceed either of the following:

$$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = 726.48 \cdot kip$$

$$V_{ni.2} := K_2 \cdot A_{cv} = 871.776 \cdot kip$$

$$V_{ni.limit} := \min\left(V_{ni.1}, V_{ni.2}\right) = 726.48 \cdot kip$$

Check for shear force resistance

$$V_{ni} := \begin{cases} V_{ni} & \text{if } V_{ni} \le V_{ni.limit} \\ V_{ni.limit} & \text{if } V_{ni} > V_{ni.limit} \end{cases} = 396.01 \cdot kip$$

Shear force resistance factor (5.5.4.2)

$$\phi := 0.9$$

Factored Interface shear resistance

$$V_{ri} := V_{ni} \cdot \phi = 356.409 \cdot kip$$

7. Factored Interface Shear Force Demands

Method 1 based upon AASHTO 5.7.4.5

Vertical distance from the extreme compressive fiber to centroid of tensile bars (Layer 1)

$d_{s.1} := 47.44in$

Vertical distance from the extreme compressive fiber to centroid of tensile bars (Layer 2)

$d_{s.2} := 44.56in$

Thickness of the bottom slab

$h_{fb} := 6in$

Distance between the centroid of the tension steel and the mid-thickness of the bottom slab

$$d_v := \left(\frac{d_{s.1} + d_{s.2}}{2}\right) - \frac{h_{fb}}{2} = 3.583 \cdot ft$$

Factored interface shear stress

$$v_{ui} := \frac{V_u}{b_w \cdot d_v} = 0.47 \cdot ksi$$

note: the demands will be the same between the interface shear at top of the web and bottom of the web

Factored interface shear force in kips/ft

$V_{ui} := v_{ui} \cdot A_{cv} = 227.721 \cdot kip$

Method 2 based on section properties

Calculation of Q for section above interface at the top of the web

Centroid from Top

$y_t := 1.61ft = 1.61 \cdot ft$

Width of top flange

$b_t := 36.067ft$

Thickness of top flange

$t_t := 8.5in$

First moment of area

$$Q_t := b_t \cdot t_t \cdot \left(y_t - \frac{t_t}{2}\right) = 32.083 \cdot ft^3$$

Moment of inertia

$I_{22} := 123.83ft^4$

Factored interface shear stress

$$v_{ui2t} := V_u \cdot \frac{Q_t}{I_{22} \cdot b_w} = 0.437 \cdot ksi$$

note: method 1 governs

Factored interface shear force in kips/ft

$V_{ui2t} := v_{ui2t} \cdot A_{cv} = 211.419 \cdot kip$

Method 2 based on section properties

Calculation of Q for section below the interface at the bottom of the web

Centroid from Bottom

$y_b := 4.25ft - 1.61ft = 2.64 \cdot ft$

Width of bottom flange

$b_b := 24ft$

Thickness of bottom flange

$t_h := 6in$

First moment of area

$$Q_b := b_b \cdot t_b \cdot \left(y_b - \frac{t_b}{2}\right) = 28.68 \cdot ft^3$$

Moment of inertia

$I_{22} := 123.83ft^4$

Factored interface shear stress	$v_{ui2b} := V_u \cdot \frac{Q_b}{I_{22} \cdot b_w} = 0.39 \cdot ksi$	note: method 1 governs
Factored interface shear force in kips/ft	$V_{ui2b} := v_{ui2b} \cdot A_{cv} = 188.992 \cdot kip$	

8. Summary

Total reinforcement area	$A_{vf} = 4.34 \cdot in^2$
Minimum reinforcement area	$A_{vf.min} = 0.404 \cdot in^2$
Does the interface has minimum reinforcement area?	$check_{minrein} := if(A_{vf.min} > A_{vf}, "no", "yes") = "yes"$
Factored interface shear demand	$V_{ui} = 227.721 \cdot kip$
Factored Interface shear resistance	$V_{ri} := V_{ni} \cdot \phi = 356.409 \cdot kip$
D/C ratio	$\frac{V_{ui}}{V_{ri}} = 0.639$

1. Purpose and Assumptions

1. The purpose is to calculate and check the interface shear at the top and bottom of the web at dv away from the edge of the support at west abutment
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength $f_c := 5ksi$

Reinforcement Yield Strength $f_y := 60ksi$

3. Structural Properties

Total Web Thickness $b_w := 9in \cdot 2 + 11.18in \cdot 2 = 40.36 \cdot in$

Mild Reinforcement

Diameter of #5 Bars $D_{n5} := 0.625in$

Area of #5 Bar $A_{n5} := 0.31in^2$

Total Number of #5 Transverse Bars $N_{tb} := 8$

Spacing of Transverse Reinforcement in Exteior Web $s_{ext} := 8in$

Spacing of Transverse Reinforcement in Interior Web $s_{int} := 13in$

Note: assume the spacing in the new west span is the same as the spacing at the end of the existing west span

Interface length considered to be engaged in shear transfer $L_{vi} := 1ft$

Area of Transverse Reinforcement within a Distance L_{vi}

$$A_v := \left[\left(\frac{N_{tb}}{2} \cdot \frac{A_{n5}}{s_{int}}\right) + \frac{\left(\frac{N_{tb}}{2} \cdot A_{n5}\right)}{s_{ext}}\right] \cdot L_{vi} = 3.005 \cdot in^2$$

4. Forces at Interface

Factored shear force $V_u := 662kip$

Note: Vu is the maximum shear force at ~3.646 ft away from pier. It's obtained from csibridge load combination 'SUP_STRENGTH_ENVELOPE' at station 149.32 ft

Permanent net compressive force normal to the shear plane $P_c := 0kip$

5. Minimum Area of Interface Shear Reinforcement

Area of concrete considered to be engaged in interface shear transfer (in^2)
(5.7.4.2)

$$A_{cv} := b_w \cdot L_{vi} = 3.363 \cdot ft^2$$

Minimum area of interface shear reinforcement crossing the shear plane within the area Acv (in^2) (5.7.4.2)

$$A_{vf.min} := 0.05 \cdot \frac{A_{cv}}{\frac{f_y}{ksi}} = 0.404 \cdot in^2$$

Area of interface shear reinforcement crossing the shear plane within the area Acv (in^2)

$$A_{vf} := A_v = 3.005 \cdot in^2$$

Minimum reinforcement check

$$if\left(A_{vf} \le A_{vf.min}, "fail", "ok"\right) = "ok"$$

6. Interface Shear Resistance

Cohesion factor (5.7.4.3)

$$c := 0.28ksi$$

Friction factor (5.7.4.3)

$$\mu := 1$$

Fraction of concrete strenght available to resist interface shear (5.7.4.3)

$$K_1 := 0.3$$

Limiting interface shear resistance (5.7.4.3)

$$K_2 := 1.8ksi$$

Nominal shear resistance of interface plane (5.7.4.3)

$$V_{ni} := c \cdot A_{cv} + \mu \cdot \left(A_{vf} \cdot f_y + P_c\right) = 315.887 \cdot kip$$

The nominal shear resistance used in the design shall not exceed either of the following:

$$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = 726.48 \cdot kip$$

$$V_{ni.2} := K_2 \cdot A_{cv} = 871.776 \cdot kip$$

$$V_{ni.limit} := \min\left(V_{ni.1}, V_{ni.2}\right) = 726.48 \cdot kip$$

Check for shear force resistance

$$V_{ni} := \begin{cases} V_{ni} & \text{if } V_{ni} \le V_{ni.limit} \\ V_{ni.limit} & \text{if } V_{ni} > V_{ni.limit} \end{cases} = 315.887 \cdot kip$$

Shear force resistance factor (5.5.4.2)

$$\phi := 0.9$$

Factored Interface shear resistance

$$V_{ri} := V_{ni} \cdot \phi = 284.298 \cdot kip$$

7. Factored Interface Shear Force Demands

Method 1 based upon AASHTO 5.7.4.5

Vertical distance from the extreme compressive fiber to centroid of tensile bars (Layer 1)

$d_{s.8} := 47in$

Vertical distance from the extreme compressive fiber to centroid of tensile bars (Layer 2)

$d_{s.9} := 49in$

Thickness of the top slab

$h_{ft} := 8.5in$

Distance between the centroid of the tension steel and the mid-thickness of the top slab

$d_v := \left(\frac{d_{s.8} + d_{s.9}}{2}\right) - \frac{h_{ft}}{2} = 3.646 \cdot ft$

Factored interface shear stress

$v_{ui} := \frac{V_u}{b_w \cdot d_v} = 0.375 \cdot ksi$

Factored interface shear force in kips/ft

$V_{ui} := v_{ui} \cdot A_{cv} = 181.577 \cdot kip$

Method 2 based on section properties

Calculation of Q for section above interface at the top of the web

Centroid from Top

$y_t := 1.61ft = 1.61 \cdot ft$

Width of top flange

$b_t := 36.067ft$

Thickness of top flange

$t_t := 8.5in$

First moment of area

$Q_t := b_t \cdot t_t \cdot \left(y_t - \frac{t_t}{2}\right) = 32.083 \cdot ft^3$

Moment of inertia

$I_{22} := 123.83ft^4$

Factored interface shear stress

$v_{ui2t} := V_u \cdot \frac{Q_t}{I_{22} \cdot b_w} = 0.354 \cdot ksi$

note: method 1 governs

Factored interface shear force in kips/ft

$V_{ui2t} := v_{ui2t} \cdot A_{cv} = 171.519 \cdot kip$

Method 2 based on section properties

Calculation of Q for section below the interface at the bottom of the web

Centroid from Bottom

$y_b := 4.25ft - 1.61ft = 2.64 \cdot ft$

Width of bottom flange

$b_b := 24ft$

Thickness of bottom flange

$t_b := 6in$

First moment of area

$Q_b := b_b \cdot t_b \cdot \left(y_b - \frac{t_b}{2}\right) = 28.68 \cdot ft^3$

Moment of inertia	$I_{22} := 123.83\,ft^4$	
Factored interface shear stress	$v_{ui2b} := V_u \cdot \frac{Q_b}{I_{22} \cdot b_w} = 0.317 \cdot ksi$	note: method 1 governs
Factored interface shear force in kips/ft	$V_{ui2b} := v_{ui2b} \cdot A_{cv} = 153.324 \cdot kip$	

8. Summary

Total reinforcement area	$A_{vf} = 3.005 \cdot in^2$
Minimum reinforcement area	$A_{vf.min} = 0.404 \cdot in^2$
Does the interface has minimum reinforcement area?	$check_{minrein} := if(A_{vf.min} > A_{vf}, "no", "yes") = "yes"$
Factored interface shear demand	$V_{ui} = 181.577 \cdot kip$
Factored Interface shear resistance	$V_{ri} := V_{ni} \cdot \phi = 284.298 \cdot kip$
D/C ratio	$\frac{V_{ui}}{V_{ri}} = 0.639$

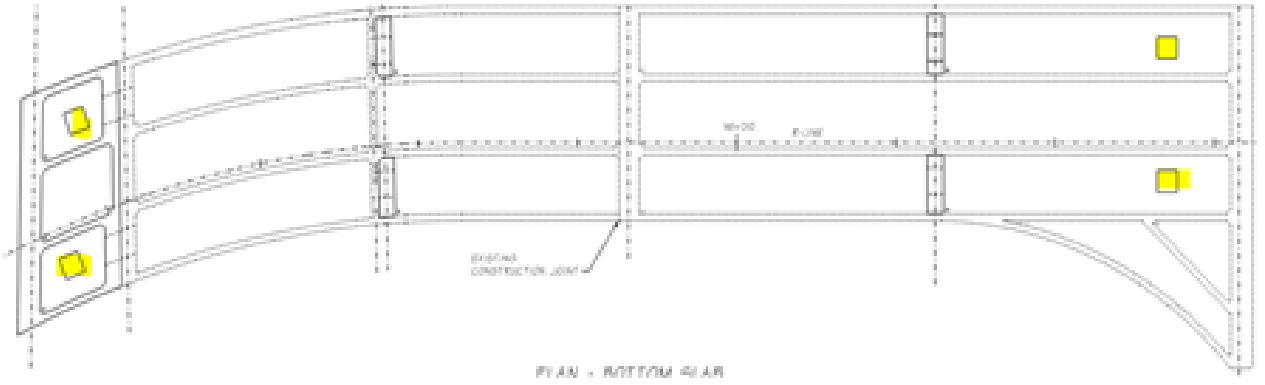
Bottom Slab Access Opening Check

Description
The following calculations calculate and check the flexural capacity at the bottom slab opening during service. The bottom slab accesses are located at 7.5 ft away from Pier 1 and 11.5 ft away

from Pier 3. The width of the opening is assumed to be 2.5 ft.

The following summarizes the opening locations and the corresponding D/C ratios:

Locations of Openings



D/C Ratios

Type	Bottom Slab Opening		
Stage	Service		
		min. reinforcement	ultimate capacity
@7.5ft away from Pier 1		0.41	0.11
@11.5ft away from Pier 3		0.25	0.27

1. Purpose and Assumptions

1. The purpose is to calculate and check the flexural capcity of the superstructure when there are access openings in the bottom slab. The check is at 11.5 ft from east abutment.
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength

$f_c := 5ksi$

Concrete Modulus of Fracture (5.4.2.6)

$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$

Prestressed Tendon Ultimate Tensile Strength

$f_{pu} := 270ksi$

Effective Stress in Prestressing Steel after Losses

$f_{pe} := \frac{254kip}{0.217in^2 \cdot 7} = 167.215 \cdot ksi$

Note: fpe is calculated based on the axial force of the tendon at pier for load case 'DL0_PT_Long_STG-1'

Reinforcement Tensile Yield Strength

$f_y := 60ksi$

Reinforcement Compressive Yield Strength

$f_{y'} := 60ksi$

3. Structural Properties

Girder

Girder Type: Box girder

Number of Cells: 3

Area of bottom slab

$A_{bs} := 14.19ft^2$

note: This area is measured using bluebeam and scaled up based on the slab thickness: 13.1sf/6in*6.5in. Measurements are shown in "typ cross-section" in folder "bottom slab opening".

Width of right opening

$b_{ro} := 2.5ft$

Width of left opening

$b_{lo} := 2.5ft$

note: width of the openings are measured on the marked up dwg using bluebeam Measurements are shown in "opening size and location" in folder "bottom slab opening".

Top Slab Thickness

$h_{ft} := 8.5in$

Bottom Slab Thickness

$h_{fb} := 6.5in$

Area of top slab (excluding fillets and openings)

$A_{bs'} := A_{bs} - (b_{ro} + b_{lo}) \cdot h_{fb} = 11.482 \cdot ft^2$

Top Slab Width

$b_t := 36ft + \frac{7in}{8} = 432.875 \cdot in$

Equivaletn Bottom Slab Width

$b_b := \frac{A_{bs'}}{h_{fb}} = 254.363 \cdot in$

Web Thickness

$b_w := 11.18in \cdot 2 + 9in \cdot 2 = 40.36 \cdot in$

note: moment of inertia and centroid is calculated by assuming the box-girder is equivalent to a wide flange cross-section with top width of b.t and bottom width of b.b, top thickness of h.ft, bottom flange thickness of h.fb, web height of h-h.ft-h.fb, and web thickness of b.w. The assumed cross-section can be found in the corresponding response 2000 file in folder "bottom slab opening".

Total Depth

$h := 4.25ft$

Cross-section Moment of Inertia about Horizontal Axis

$I_{22} := 117.24ft^4$

Vertical Distance from Top of Deck to Centroid

$y := 1.63ft$

Cross-section Area	$A := 52.739ft^2 - \left[\left(b_{ro} + b_{lo}\right)\cdot h_{fb}\right] = 50.031\cdot ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$

note: 52.74sf is obtained from csibridge model for typical superstructure cross-section in east span

Tendon

Strand Area	$A_{ps} := 0.217in^2$
Number of Strands per Tendon	$N_s := 7$
Number of Tendon	$N_t := 4$
Total Tendon Area	$A_{ps.total} := A_{ps}\cdot N_s\cdot N_t = 6.076\cdot in^2$
Longitudinal Distance of the Cross-Section from East Abutment	$x := 11.5ft$
Vertical Distance from Top of Deck at Start of Tendon 1	$y_{tendon.x.1.71} := 1.63ft$
Vertical Distance from Top of Deck at End of Tendon 1	$y_{tendon.x38.09} := 3.313ft$
Length of Tendon 1	$L_{tendon} := (38.09ft + 1.71ft) = 39.8\cdot ft$
Vertical Distance from Top of Deck at Checking Location	$y_{tendon} := \frac{y_{tendon.x38.09} - y_{tendon.x.1.71}}{L_{tendon}}\cdot x + y_{tendon.x.1.71} = 2.116\cdot ft$
Vertical Distance from Top of Deck	$y_{tendon} = 2.116\cdot ft$
Vertical Distance from Extreme Compression Fiber to to Centroid of Prestressing Tendons	$d_p := y_{tendon} = 2.116\cdot ft$

Mild Reinforcement

Top Cover for Top Slab	$c_{t.t} := 2.5in$
Top Cover for Bottom Slab	$c_{t.b} := 1in$
Bottom Cover for Top/Bottom Slab	$c_b := 1in$
Diameter of #10	$D_{N10} := 1.27in$
Diameter of #7	$D_{N7} := 0.875in$
Diameter of #5	$D_{N5} := 0.625in$
Diameter of #4	$D_{N4} := 0.5in$
Vertical Reinforcement Spacing in Web	$s_w := 8in$
Young's Modulus of Reinforcement	$E_s := 200GPa = 29007.548\cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	37	$A_{s.1} := 37\cdot 0.6in^2 = 22.2\cdot in^2$
Bar in Top Slab (Layer 2)	#7	27	$A_{s.2} := 27\cdot 0.6in^2 = 16.2\cdot in^2$

Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#10	40 − 8 = 32	$A_{s.8} := 32 \cdot 1.27in^2 = 40.64 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#10	40 − 8 = 32	$A_{s.9} := 32 \cdot 1.27in^2 = 40.64 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1)	$d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$
Bar in Top Slab (Layer 2)	$d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$
Bar in Top Slab (Layer 3)	$d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$
Bar in Web (Layer 1)	$d_{s.4} := h - 32.5in = 18.5 \cdot in$
Bar in Web (Layer 2)	$d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$
Bar in Web (Layer 3)	$d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$
Bar in Web (Layer 4)	$d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$
Bar in Bottom Slab (Layer 1)	$d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N10}}{2}\right) = 46.635 \cdot in$
Bar in Bottom Slab (Layer 2)	$d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N10}}{2}\right) = 48.865 \cdot in$

4. Flexural Demand

The factored flexural resistance, M_r, must be greater than M_u (ultimate moment), and the minimum reinforcement requirement. The minimum reinforcement requirement states that the flexural resistance must at least equal to $1.33M_u$ or the cracking moment, M_{cr}, whichever is less.

Factored Flexural Resistance

The M_u is:

$$M_{u.east} := 5166kip \cdot ft$$

note: this moment is obtained from csibridge load combination "SUP_STRENGTH_ENVELOPE"

Minimum Reinforcement Requirement

$1.33M_{u.east} = 6870.78 \cdot kip \cdot ft$

Flexural Cracking Variability Factor (5.6.3.3)

$\gamma_1 := 1.6$

Prestress Variability Factor (5.6.3.3)

$\gamma_2 := 1.0$

Ratio of Specified Minimum Yield Strenght to Ultimate Tensile Stregth of the Mild Reinforcement (5.6.3.3)

$\gamma_3 := 0.67$

Section Modulus for the Extreme Fiber

$S_c := \frac{I_{22}}{h - y} = 44.748 \cdot ft^3$

Tendon Vertical Eccentricity

$e := -d_p + y = -0.486 \cdot ft$

Compressive Stress in Concrete due to Effective Prestress Forces

$f_{cpe} := \frac{-(A_{ps.total} \cdot f_{pe})}{A} + \frac{A_{ps.total} \cdot f_{pe} \cdot e}{S_c} = -0.218 \cdot ksi$

Cracking Moment (5.6.3.3)

$M_{cr} := (\gamma_1 \cdot f_r - \gamma_2 \cdot f_{cpe}) \cdot \gamma_3 \cdot S_c = 4646.922 \cdot kip \cdot ft$

Therefore, the demand, D, is

$D := \min[(|M_{cr}|), |1.33M_{u.east}|] = 4646.922 \cdot kip \cdot ft$

5. Factored Flexural Resistance

Stress Block Factor (5.6.2.2)

$\beta_1 := 0.85 - \frac{0.05}{ksi} \cdot (f_c - 4ksi) = 0.8$

Assuming:
1. The neutral axis is within the top slab
2.The compression reinforcements and reinforcements in webs are ignored
3. The tension reinforcements in the slab are assumed to reach the yield strength

Number of Plastic Hinges at Supports (5.6.3.1.2)

$N_{s.ph} := 1$

East Span Length

$l_{east} := 76.63ft$

West Span Length

$l_{west} := 76.30ft$

Length of Tendon between Anchorages

$l_i := l_{east} + l_{west} = 152.93 \cdot ft$

Effective Tendon Length

$l_e := \frac{2 \cdot l_i}{2 + N_{s.ph}} = 1223.44 \cdot in$

from Equations 5.6.3.1.2-1 :

$f_{ps}=f_{pe}+900*(d_p-c/l_e)$ and $c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1 b)$

from 5.6.3.1.2-4:
$c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1 b)$

By substituting 5.6.3.1.2-4 into 5.6.3.1.2-1 we can solve for f_{ps}

$$y_{tendon} = 2.116\cdot ft$$

$$f_{pe} = 167.215\cdot ksi \quad A_{ps.total} = 6.076\cdot in^2 \quad A_{s.8} = 40.64\cdot in^2 \quad A_{s.9} = 40.64\cdot in^2 \quad f_y = 60\cdot ksi$$

$$d_p = 25.396\cdot in \quad f_c = 5\cdot ksi \quad \beta_1 = 0.8 \quad b_b = 254.363\cdot in \quad l_e = 1223.44\cdot in$$

Solve for f_{ps}

$$\left[f_{ps} = 167.215\ ksi + 900\cdot ksi\cdot \left[\frac{25.396\ in - \frac{\left[6.076\ in^2\cdot f_{ps} + \left[\left(40.64\ in^2 + 40.64\ in^2\right)\cdot 60ksi\right]\right]}{[0.85\cdot 5ksi\cdot 0.8\cdot (254.363\ in)]}}{1.223\times 10^3\ in} \right] \right] \text{solve}, f_{ps} \rightarrow 180.82\cdot ksi$$

Average prestressing strength

$$f_{ps} := 180.82ksi$$

Note: f.ps is less than f.py which is assumed to be 0.9*fpu=243ksi

Distance from Compression Extreme Fibre to Neutral Axis

$$c := \frac{A_{ps.total}\cdot f_{ps} + \left[\left(A_{s.8} + A_{s.9}\right)\cdot f_y\right]}{0.85\cdot f_c\cdot \beta_1\cdot b_t} = 4.06\cdot in$$

Depth of Equivalent Rectangular Stress Block

$$a := \beta_1\cdot c = 3.248\cdot in$$

Tension-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$$\varepsilon_{tl} := 0.005$$

Compression-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$$\varepsilon_{cl} := 0.002$$

Net tensile strain in the extreme tension steel

$$\varepsilon_t := 0.0221$$

Note: Net tensile strain in the extreme tension steel are obtained from Response 2000 "Response-2000-east bottom opening results".

Flexural Resistance Factor (5.5.4.2)

$$\phi := \min\left[0.9, 0.75 + \frac{0.15\cdot\left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}}\right] = 0.9$$

Fexural Resistance, ϕM_n, for positive bending moment (5.6.3.2.3)

$$M_n := \left[A_{ps.total} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2} \right) + \left(A_{s.8} \right) \cdot f_y \cdot \left(d_{s.8} - \frac{a}{2} \right) + A_{s.9} \cdot f_y \cdot \left(d_{s.9} - \frac{a}{2} \right) \right] = 20922.006 \cdot kip \cdot ft$$

$$\phi \cdot M_n = 18829.805 \cdot kip \cdot ft$$

7 Summary

	Demand	Resistance	D/C Ratio
Minimum Reinforcement	$M_{cr} = 4646.922 \cdot kip \cdot ft$	$\phi \cdot M_n = 18829.805 \cdot kip \cdot ft$	$\frac{M_{cr}}{(\phi \cdot M_n)} = 0.247$
Ultimate Capacity	$M_{u.east} = 5166 \cdot kip \cdot ft$	$\phi \cdot M_n = 18829.805 \cdot kip \cdot ft$	$\frac{M_{u.east}}{(\phi \cdot M_n)} = 0.274$

1. Purpose and Assumptions

1. The purpose is to calculate and check the flexural capcity of the superstructure when there are openings on the bottom slab. The check is at 7.5 ft away from the centerline of new west abutment. This is approximately at the east edge of the opening.
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$	
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$	
Reinforcement Tensile Yield Strength	$f_y := 60ksi$	
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$	
Prestressed Tendon Ultimate Tensile Strength	$f_{pu} := 270ksi$	
Effective Stress in Prestressing Steel after Losses	$f_{pe} := \frac{250kip}{0.217in^2 \cdot 7} = 164.582 \cdot ksi$	Note: fpe is calculated based on the axial force of the tendon at pier for load case 'DL0_PT_Long_STG-1'

3. Structural Properties

Girder

Girder Type	Box girder	
Number of Cells	3	
Area of top slab (excluding fillets)	$A_{bs} := 13.1ft^2$	note: This area is measured using bluebeam. Measurements are shown in "typ cross-section" in folder "bottom slab opening".
Width of right opening	$b_{ro} := 2.5ft$	note: width of the openings are measured on the marked up dwg using bluebeam Measurements are shown in "opening size and location" in folder "bottom slab opening".
Width of left opening	$b_{lo} := 2.5ft$	
Top Slab Thickness	$h_{ft} := 8.5in$	
Bottom Slab Thickness	$h_{fb} := 6in$	
Area of top slab (excluding fillets and openings)	$A_{bs'} := A_{bs} - (b_{ro} + b_{lo}) \cdot h_{fb} = 10.6 \cdot ft^2$	
Equivalent Bottom Slab Width	$b_b := \frac{A_{bs'}}{h_{fb}} = 254.4 \cdot in$	note: moment of inertia and centroid is calculated by assuming the box-girder is equivalent to a wide flange cross-section with top width of b.t and bottom width of b.b, top thickness of h.ft, bottom flange thickness of h.fb, web height of h-h.ft-h.fb, and web thickness of b.w. The assumed cross-section can be found in the
Top Slab Width	$b_t := 36.073ft = 432.876 \cdot in$	
Web Thickness	$b_w := 11.18in \cdot 2 + 9in \cdot 2 = 40.36 \cdot in$	
Total Depth	$h := 4.25ft$	
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 113.94ft^4$	

Vertical Distance from Top of Deck to Centroid	$y := 1.6ft$	corresponding response 2000 file in folder "bottom slab opening".
Cross-section Area	$A := 51.842\,ft^2 - \left[\left(b_{ro} + b_{lo}\right)\cdot h_{fb}\right] = 49.342\cdot ft^2$	
Height of the Fillet	$h_{fillet} := 0.5ft$	

Tendon

Strand Area	$A_{ps} := 0.217in^2$	
Number of Strands per Tendon	$N_s := 7$	
Number of Tendon	$N_t := 4$	
Total Tendon Area	$A_{ps.total} := A_{ps}\cdot N_s\cdot N_t = 6.076\cdot in^2$	
Vertical Distance from Top of Deck	$y_{tendon} := 2.9ft$	note: knowing the vertical distanace at station 152.93ft is 1.65ft and the vertical distance at 140.425 ft is 3.479 ft, by interpolation, the vertical distance at station 143.68ft is approximately 2.9ft
Vertical Distance from Extreme Compression Fiber to to Centroid of Prestressing Tendons	$d_p := y_{tendon} = 2.9\cdot ft$	

Mild Reinforcement

Top Clearance	$c_t := 2.5in$	
Bottom Clearance	$c_b := 1in$	
Diameter of #8	$D_{N8} := 1in$	
Diameter of #7	$D_{N7} := 0.875in$	
Diameter of #5	$D_{N5} := 0.625in$	
Diameter of #4	$D_{N4} := 0.5in$	
Vertical Reinforcement Spacing in Web	$s_w := 8in$	
Young's Modulus of Reinforcement	$E_s := 200GPa = 2.901\times 10^4\cdot ksi$	

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	40	$A_{s.1} := 40\cdot 0.6in^2 = 24\cdot in^2$
Bar in Top Slab (Layer 2)	#7	30	$A_{s.2} := 30\cdot 0.6in^2 = 18\cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10\cdot 0.6in^2 = 6\cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4\cdot 1.27\cdot in^2 + 4\cdot 0.79in^2 = 8.24\cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 4\cdot 0.2\cdot in^2 + 4\cdot 0.2in^2 = 1.6\cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 4\cdot 0.2\cdot in^2 + 4\cdot 0.2in^2 = 1.6\cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 4\cdot 0.2\cdot in^2 + 4\cdot 0.2in^2 = 1.6\cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	$36 - 8 = 28$	$A_{s.8} := 28\cdot 0.79in^2 = 22.12\cdot in^2$

Bar in Bottom Slab (Layer 2) #8 $36 - 8 = 28$ $A_{s.9} := 28 \cdot 0.79in^2 = 22.12 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_t + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := h_{ft} + \frac{h_{fillet}}{2} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2}\right) = 47 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N8}}{2}\right) = 49 \cdot in$

4. Flexural Demand

The factored flexural resistance, M_r, must be greater than M_u (ultimate moment), and the minimum reinforcementrequirement. The minimum reinforcement requirement states that the flexural resistance must at least equal to $1.33M_u$ or the cracking moment, M_{cr}, whichever is less.

Factored Flexural Resistance

The M_u is:

$M_u := 1379kip \cdot ft$

Note: this is from csibridge "ULS Envelope"

Minimum Reinforcement Requirement

$1.33M_u = 1.834 \times 10^3 \cdot kip \cdot ft$

Flexural Cracking Variability Factor (5.6.3.3) $\gamma_1 := 1.6$

Prestress Variability Factor (5.6.3.3) $\gamma_2 := 1.0$

Ratio of Specified Minimum Yield Strenght to Ultimate Tensile Stregth of the Mild Reinforcement (5.6.3.3)

$$\gamma_3 := 0.67$$

Section Modulus for the Extreme Fiber

$$S_c := \frac{I_{22}}{h - y} = 42.996 \cdot ft^3$$

Tendon Vertical Eccentricity

$$e := -d_p + y = -1.3 \cdot ft$$

Compressive Stress in Concrete due to Effective Prestress Forces

$$f_{cpe} := \frac{-\left(A_{ps.total} \cdot f_{pe}\right)}{A} + \frac{A_{ps.total} \cdot f_{pe} \cdot e}{S_c} = -0.351 \cdot ksi$$

Cracking Moment (5.6.3.3)

$$M_{cr} := \left(\gamma_1 \cdot f_r - \gamma_2 \cdot f_{cpe}\right) \cdot \gamma_3 \cdot S_c = 5.017 \times 10^3 \cdot kip \cdot ft$$

Therefore, the demand, D, is

$$D := \min\left[\left(\left|M_{cr}\right|\right), \left|1.33 M_u\right|\right] = 1.834 \times 10^3 \cdot kip \cdot ft$$

5. Factored Flexural Resistance

Stress Block Factor (5.6.2.2)

$$\beta_1 := 0.85 - \frac{0.05}{ksi} \cdot \left(f_c - 4ksi\right) = 0.8$$

Assuming:
1. The neutral axis is within the top slab
2.The compression reinforcements and reinforcements in webs are ignored
3. The tension reinforcements in the slab are assumed to reach the yield strength

Number of Plastic Hinges at Supports (5.6.3.1.2)

$$N_{s.ph} := 1$$

East Span Length

$$l_{east} := 76.63ft$$

West Span Length

$$l_{west} := 76.30ft$$

Length of Tendon between Anchorages

$$l_i := l_{east} + l_{west} = 152.93 \cdot ft$$

Effective Tendon Length

$$l_e := \frac{2 \cdot l_i}{2 + N_{s.ph}} = 101.953 \cdot ft$$

from Equations 5.6.3.1.2-1 :
$f_{ps}=f_{pe}+900*(d_p-c/l_e)$ and $c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1 b)$

from 5.6.3.1.2-4:
$c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1 b)$

By substituting 5.6.3.1.2-4 into 5.6.3.1.2-1 we can solve for f_{ps}

Solve for Average Stress in Prestressing Steel, f_{ps}

$f_{pe} = 164.582 \cdot ksi$ $A_{ps.total} = 6.076 \cdot in^2$ $A_{s.8} + A_{s.9} = 44.24 \cdot in^2$ $f_y = 60 \cdot ksi$

$d_p = 34.8 \cdot in$ $f_c = 5 \cdot ksi$ $\beta_1 = 0.8$ $b_t = 432.876 \cdot in$ $l_e = 1.223 \times 10^3 \cdot in$

$$\left[f_{ps} = 164.582\ ksi + 900 \cdot ksi \cdot \left[\frac{34.8\ in - \frac{\left(6.076\ in^2 \cdot f_{ps} + 44.24 in^2 \cdot 60 ksi\right)}{[0.85 \cdot 5 ksi \cdot 0.8 \cdot (432.876\ in)]}}{1.223 \times 10^3\ in} \right] \right] \text{solve}, f_{ps} \rightarrow 188.292 \cdot ksi$$

Average prestressing strength	$f_{ps} := 188.292 ksi$	Note: f.ps is less than f.py which is assumed to be 0.9*fpu=243ksi
Distance from Compression Extreme Fibre to Neutral Axis	$c := \frac{A_{ps.total} \cdot f_{ps} + \left[\left(A_{s.8} + A_{s.9}\right) \cdot f_y\right]}{0.85 \cdot f_c \cdot \beta_1 \cdot b_t} = 2.581 \cdot in$	
Depth of Equivalent Rectangular Stress Block	$a := \beta_1 \cdot c = 0.172 \cdot ft$	
Tension-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)	$\varepsilon_{tl} := 0.005$	
Compression-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)	$\varepsilon_{cl} := 0.002$	
Net tensile strain in the extreme tension steel	$\varepsilon_t := 0.0323$	Note: Net tensile strain in the extreme tension steel are obtained from Response 2000 "Response-2000-west abutment bottom opening results".
Flexural Resistance Factor (5.5.4.2)	$\phi := \min\left[0.9, 0.75 + \frac{0.15 \cdot \left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}}\right] = 0.9$	

Fexural Resistance, ϕM_n, for positive bending moment (5.6.3.2.3)

$$M_n := \left[A_{ps.total} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \left(A_{s.8}\right) \cdot f_y \cdot \left(d_{s.8} - \frac{a}{2}\right) + A_{s.9} \cdot f_y \cdot \left(d_{s.9} - \frac{a}{2}\right)\right] = 1.361 \times 10^4 \cdot kip \cdot ft$$

$\phi \cdot M_n = 1.225 \times 10^4 \cdot kip \cdot ft$

6 Summary

	Demand	Resistance	D/C Ratio
Minimum Reinforcement	$M_{cr} = 5.017 \times 10^3 \cdot kip \cdot ft$	$\phi \cdot M_n = 1.225 \times 10^4 \cdot kip \cdot ft$	$\frac{M_{cr}}{(\phi \cdot M_n)} = 0.41$

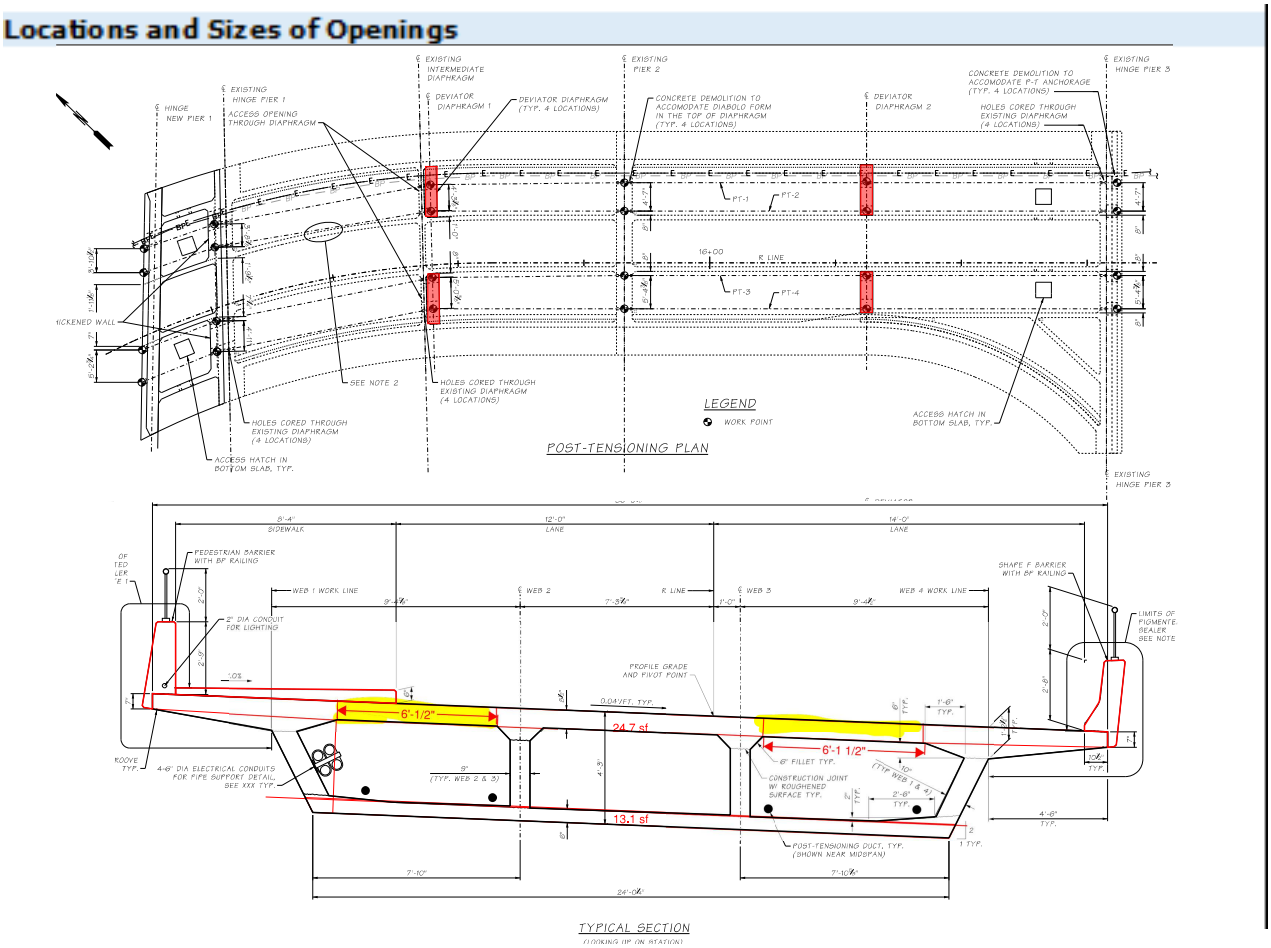
Ultimate Capacity	$M_u = 1.379 \times 10^3 \cdot kip \cdot ft$	$\phi \cdot M_n = 1.225 \times 10^4 \cdot kip \cdot ft$	$\frac{M_u}{(\phi \cdot M_n)} = 0.113$

Temporary Deck Access Opening Check

Description:
The following calculations calculate and check the flexural capacity at the top slab opening during construction and during service.

Notes:
1. For the construction stage, it is assumed that there is no reinforcement nor concrete within the demolition limits. The tendons are not stressed. The bridge is subjected to factored dead load and construction live load. For the service stage, it is assumed that the concrete within the demolition limits is restored but not the reinforcement. The tendons are stressed. The bridge is subjected to all the loadings that are applicable during the service.
2. The top slab accesses are located at 41.63 ft away from Pier 1 and 105.6 ft away from Pier 1. To be conservative, the width of the opening is assumed to be 6'-1.5" as shown below.
3. The governing D/C ratio is 0.41 during service and 0.59 during construction. Both governing D/C ratios are at the west diaphragm, which is located at 105.6 ft from Pier 3.

Locations and Sizes of Openings



D/C Ratios

Type: Top Slab Opening
Stage: Service

	min. reinforcement	ultimate capacity
@105.6ft away from Pier 3	0.37	0.41
@41.63t away from Pier 3	0.24	0.34

Type: Top Slab Opening
Stage: Construction

	min. reinforcement	ultimate capacity
@105.6ft away from Pier 3	0.31	0.35
@41.63t away from Pier 3	0.19	0.31

1. Purpose and Assumptions

1. The purpose is to calculate and check the flexural capcity of the superstructure when there are openings in the top slab. The check is at the centerline of the intermediate diaphragm in east span (41.63 ft from east abutment).This calculation assumes the reinforcement within the demolition limits is not restored after the construction while the bridge is subjected to loadings during service.
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Prestressed Tendon Ultimate Tensile Strength	$f_{pu} := 270ksi$
Effective Stress in Prestressing Steel after Losses	$f_{pe} := \frac{262kip}{0.217in^2 \cdot 7} = 172.482 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$

Note: fpe is calculated based on the axial force of the tendon at pier for load case 'DL0_PT_Long_STG-1'

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft = 432.876 \cdot in$
Bottom Width	$b_b := 24ft + \frac{0.25in \cdot ft}{12in} = 24.021 \cdot ft$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6.5in$
Web Thickness	$b_w := 11.18in \cdot 2 + 9in \cdot 2 = 3.363 \cdot ft$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 127.82ft^4$
Vertical Distance from Top of Deck to Centroid	$y := 1.647ft$
Cross-section Area	$A := 52.739ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$

Tendon

Strand Area	$A_{ps} := 0.217in^2$
Number of Strands per Tendon	$N_s := 7$
Number of Tendon	$N_t := 4$
Total Tendon Area	$A_{ps.total} := A_{ps} \cdot N_s \cdot N_t = 6.076 \cdot in^2$
Vertical Distance from Top of Deck	$y_{tendon} := 3.438ft$
Vertical Distance from Extreme Compression Fiber to to Centroid of Prestressing Tendons	$d_p := y_{tendon} = 3.438 \cdot ft$

Mild Reinforcement

Top Cover for Top Slab	$c_{t.t} := 2.5in$
Top Cover for Bottom Slab	$c_{t.b} := 1in$
Bottom Cover for Top/Bottom Slab	$c_b := 1in$
Diameter of #10	$D_{N10} := 1.27in$
Diameter of #7	$D_{N7} := 0.875in$
Diameter of #5	$D_{N5} := 0.625in$
Diameter of #4	$D_{N4} := 0.5in$
Vertical Reinforcement Spacing in Web	$s_w := 8in$
Young's Modulus of Reinforcement	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	$37 - 12 = 25$	$A_{s.1} := 25 \cdot 0.6in^2 = 15 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	$27 - 12 = 15$	$A_{s.2} := 15 \cdot 0.6in^2 = 9 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#10	40	$A_{s.8} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#10	40	$A_{s.9} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N10}}{2}\right) = 46.635 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N10}}{2}\right) = 48.865 \cdot in$

4. Flexural Demand

The factored flexural resistance, M_r, must be greater than the Strength I load combination, M_u (ultimate moment), and the minimum reinforcement requirement. The minimum reinforcement requirement states that the flexural resistance must at least equal to $1.33M_u$ or the cracking moment, M_{cr}, whichever is less.

Factored Flexural Resistance

The M_u is:

$M_{u.east} := 8210kip \cdot ft$

Note: this moment (Mu) is due to loads at service. The results is obtained from csibrdige "ULS-Envelope"

Minimum Reinforcement Requirement

$1.33M_{u.east} = 1.092 \times 10^4 \cdot kip \cdot ft$

Flexural Cracking Variability Factor (5.6.3.3) $\gamma_1 := 1.6$

Prestress Variability Factor (5.6.3.3) $\gamma_2 := 1.0$

Ratio of Specified Minimum Yield Strenght to Ultimate Tensile Stregth of the Mild Reinforcement (5.6.3.3)

$$\gamma_3 := 0.67$$

Section Modulus for the Extreme Fiber

$$S_c := \frac{I_{22}}{h - y} = 49.105 \cdot ft^3$$

Tendon Vertical Eccentricity

$$e := -d_p + y = -1.791 \cdot ft$$

Compressive Stress in Concrete due to Effective Prestress Forces

$$f_{cpe} := \frac{-\left(A_{ps.total} \cdot f_{pe}\right)}{A} + \frac{A_{ps.total} \cdot f_{pe} \cdot e}{S_c} = -0.403 \cdot ksi$$

Cracking Moment (5.6.3.3)

$$M_{cr} := \left(\gamma_1 \cdot f_r - \gamma_2 \cdot f_{cpe}\right) \cdot \gamma_3 \cdot S_c = 5.979 \times 10^3 \cdot kip \cdot ft$$

Therefore, the demand, D, is

$$D := \min\left[\left(\left|M_{cr}\right|\right), \left|1.33 M_{u.east}\right|\right] = 5.979 \times 10^3 \cdot kip \cdot ft$$

5. Factored Flexural Resistance

Stress Block Factor (5.6.2.2)

$$\beta_1 := 0.85 - \frac{0.05}{ksi} \cdot \left(f_c - 4ksi\right) = 0.8$$

Assuming:
1. The neutral axis is within the top slab
2.The compression reinforcements and reinforcements in webs are ignored
3. The tension reinforcements in the slab are assumed to reach the yield strength

Number of Plastic Hinges at Supports (5.6.3.1.2)

$$N_{s.ph} := 1$$

East Span Length

$$l_{east} := 76.63ft$$

West Span Length

$$l_{west} := 76.30ft$$

Length of Tendon between Anchorages

$$l_i := l_{east} + l_{west} = 152.93 \cdot ft$$

Effective Tendon Length

$$l_e := \frac{2 \cdot l_i}{2 + N_{s.ph}} = 1.223 \times 10^3 \cdot in$$

from Equations 5.6.3.1.2-1 :
$f_{ps}=f_{pe}+900*(d_p-c/l_e)$ and $c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1b)$

from 5.6.3.1.2-4:
$c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1b)$

By substituting 5.6.3.1.2-4 into 5.6.3.1.2-1 we can solve for f_{ps}

$$y_{tendon} = 3.438 \cdot ft$$

$f_{pe} = 172.482 \cdot ksi$ $A_{ps.total} = 6.076 \cdot in^2$ $A_{s.8} = 50.8 \cdot in^2$ $A_{s.9} = 50.8 \cdot in^2$

$d_p = 41.256 \cdot in$ $f_c = 5 \cdot ksi$ $\beta_1 = 0.8$ $b_t = 432.876 \cdot in$ $l_e = 1.223 \times 10^3 \cdot in$

Solve for f_{ps}

$$\left[f_{ps} = 172.482\ ksi + 900 \cdot ksi \cdot \left[\frac{41.256\ in - \frac{\left[6.076\ in^2 \cdot f_{ps} + \left[\left(50.8\ in^2 + 50.8\ in^2\right) \cdot 60ksi\right]\right]}{[0.85 \cdot 5ksi \cdot 0.8 \cdot (432.876\ in)]}}{1.223 \times 10^3\ in} \right] \right] \text{solve}, f_{ps} \rightarrow 199.19 \cdot ksi$$

Average prestressing strength

$f_{ps} := 199.19 \cdot ksi$

Note: f.ps is less than f.py which is assumed to be 0.9*fpu=243ksi

Distance from Compression Extreme Fibre to Neutral Axis

$$c := \frac{A_{ps.total} \cdot f_{ps} + \left[\left(A_{s.8} + A_{s.9}\right) \cdot f_y\right]}{0.85 \cdot f_c \cdot \beta_1 \cdot b_t} = 4.964 \cdot in$$

Depth of Equivalent Rectangular Stress Block

$a := \beta_1 \cdot c = 0.331 \cdot ft$

Tension-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$\varepsilon_{tl} := 0.005$

Compression-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$\varepsilon_{cl} := 0.002$

Net tensile strain in the extreme tension steel

$\varepsilon_t := 0.0145$

Note: Net tensile strain in the extreme tension steel are obtained from Response 2000 "Response-2000-east diaphragm opening outputs".

Flexural Resistance Factor (5.5.4.2)

$$\phi := \min\left[0.9, 0.75 + \frac{0.15 \cdot \left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}}\right] = 0.9$$

Fexural Resistance, ϕM_n, for positive bending moment (5.6.3.2.3)

$$M_n := \left[A_{ps.total} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right) + \left(A_{s.8}\right) \cdot f_y \cdot \left(d_{s.8} - \frac{a}{2}\right) + A_{s.9} \cdot f_y \cdot \left(d_{s.9} - \frac{a}{2}\right)\right] = 2.721 \times 10^4 \cdot kip \cdot ft$$

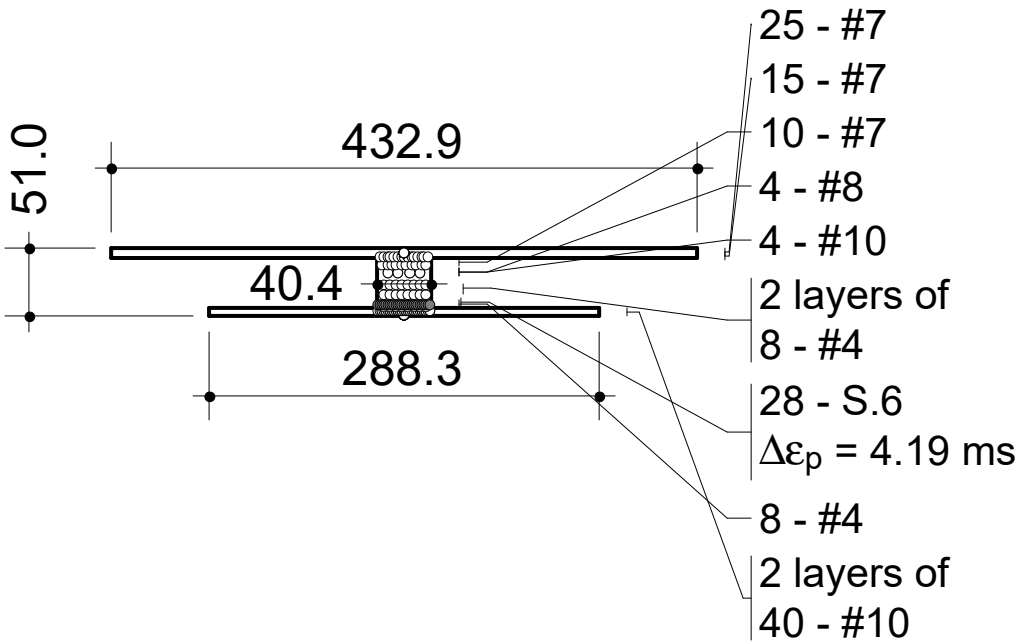
$\phi \cdot M_n = 2.449 \times 10^4 \cdot kip \cdot ft$

6 Summary

	Demand	Resistance	D/C Ratio
Minimum Reinforcement	$M_{cr} = 5.979 \times 10^{3} \cdot kip \cdot ft$	$\phi \cdot M_n = 2.449 \times 10^{4} \cdot kip \cdot ft$	$\frac{M_{cr}}{(\phi \cdot M_n)} = 0.244$
Ultimate Capacity	$M_{u.east} = 8.21 \times 10^{3} \cdot kip \cdot ft$	$\phi \cdot M_n = 2.449 \times 10^{4} \cdot kip \cdot ft$	$\frac{M_{u.east}}{(\phi \cdot M_n)} = 0.335$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6884.0	7945.0
Inertia (in^4)	2527160.2	3106380.4
y_t (in)	20.1	22.3
y_b (in)	30.9	28.7
S_t (in^3)	125961.5	139014.3
S_b (in^3)	81687.2	108409.2



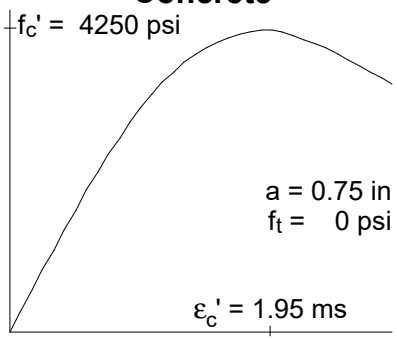
Crack Spacing

2 x dist + 0.1 d_b /ρ

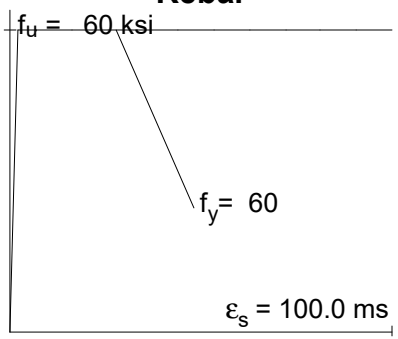
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

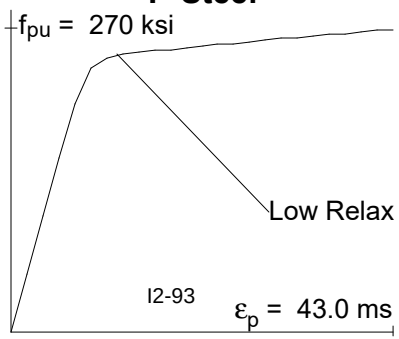
Concrete



Rebar



P-Steel



All dimensions in inches
Clear cover to reinforcement = 1.50 in

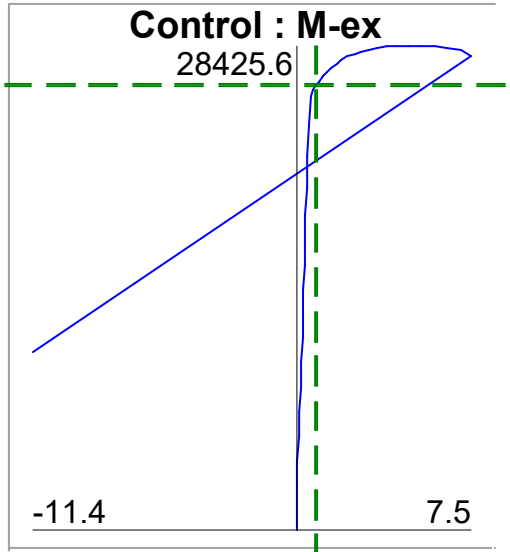
East Diaphragm Top

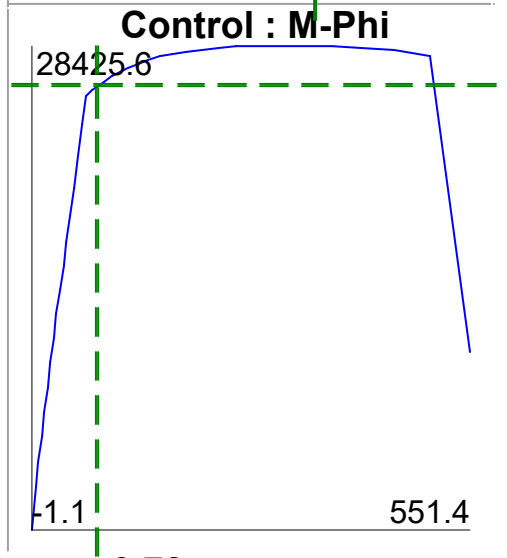
2021/4/6

Response-2000 v 1.0.5

East Diaphragm Top Opening w/ PT

2021/4/5 - 4:37 pm





ε_{x0} = 0.78 ms

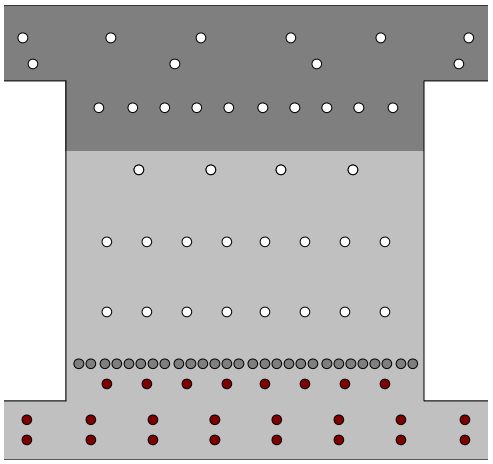
ϕ = 81.96 rad/10^6 in

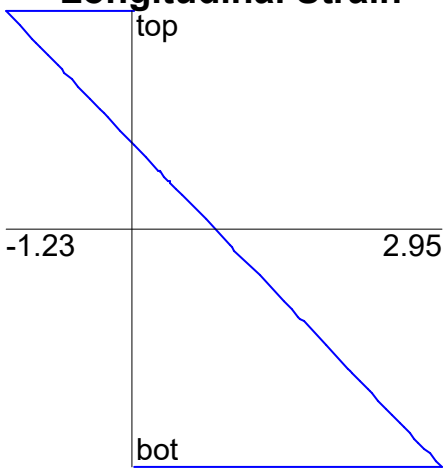
γ_{xy}(avg) = 0.00 ms

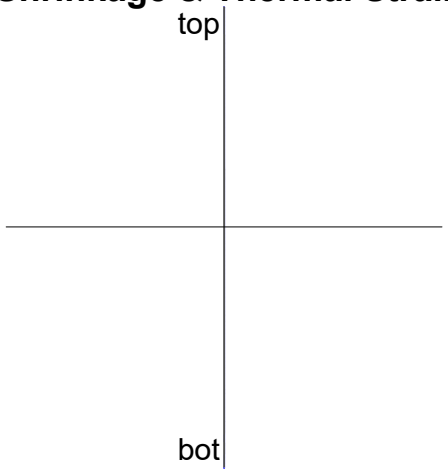
Axial Load = -2.1 kips

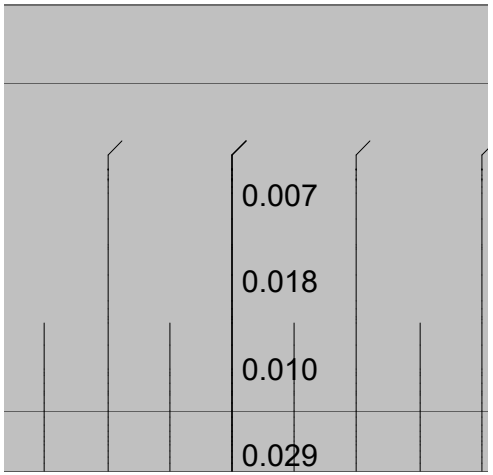
Moment:= 26122.2 ft-kips

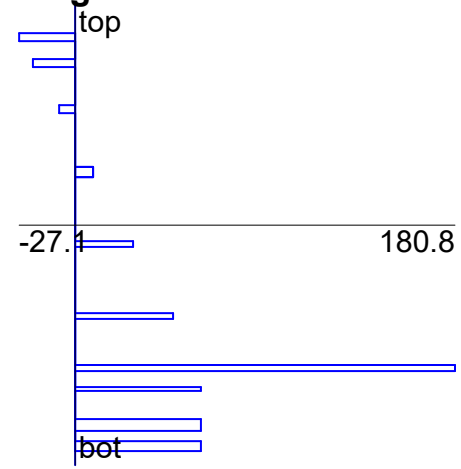
Shear = 0.0 kips

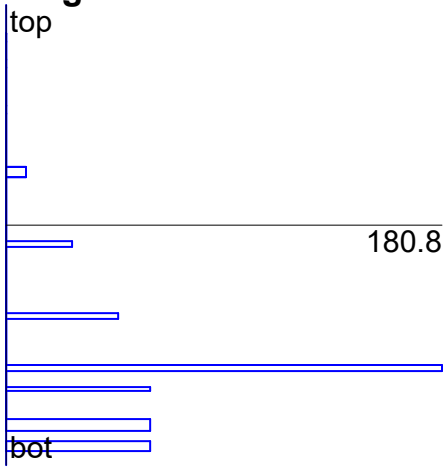


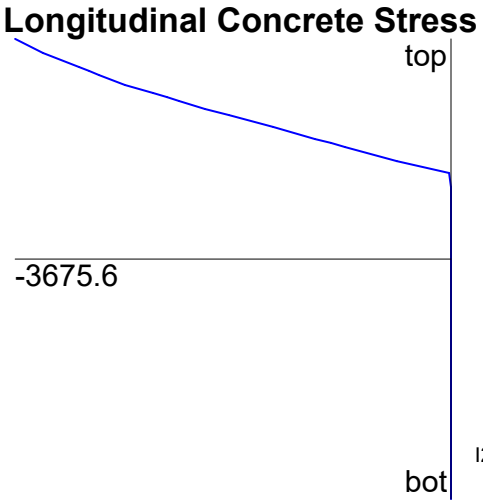


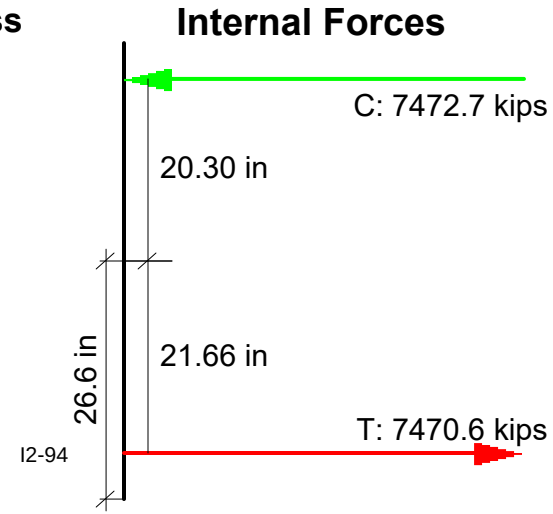


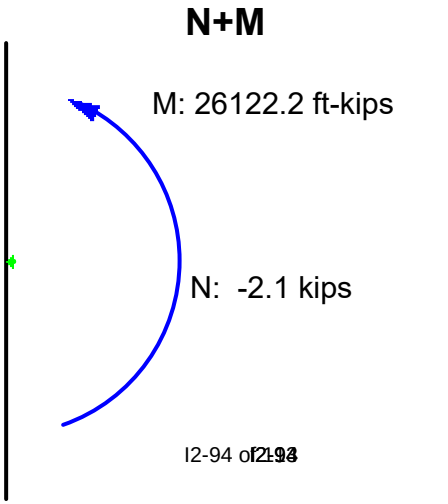












1. Purpose and Assumptions

1. The purpose is to calculate and check the flexural capcity of the superstructure when there are openings on the top slab. The check is for the cross-section at the centerline of the intermediate diaphragm in west span (105.63 ft from east abutment). This calculation assumes the reinforcement within the demolition limits is not restored after the construction while the bridge is subjected to loadings during service.
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Prestressed Tendon Ultimate Tensile Strength	$f_{pu} := 270ksi$
Effective Stress in Prestressing Steel after Losses	$f_{pe} := \frac{273kip}{0.217in^2 \cdot 7} = 179.724 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$

Note: fpe is calculated based on the axial force of the tendon at pier for load case 'DL0_PT_Long_STG-1'

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Area of top slab (excluding fillets)	$A_{ts} := 24.7ft^2$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6in$
Top Width	$b_t := 36.073ft = 432.876 \cdot in$
Bottom Slab Width	$b_b := 24ft + 0.25in = 288.25 \cdot in$
Web Thickness	$b_w := 11.18in \cdot 2 + 9in \cdot 2 = 40.36 \cdot in$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 123.83ft^4$
Vertical Distance from Top of Deck to Centroid	$y := 1.61ft$
Cross-section Area	$A := 51.842ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$

note: measured using bluebeam. Measurements are shown in "typ cross-section" in folder "top slab opening".

Tendon

Strand Area	$A_{ps} := 0.217in^2$
Number of Strands per Tendon	$N_s := 7$
Number of Tendon	$N_t := 4$
Total Tendon Area	$A_{ps.total} := A_{ps} \cdot N_s \cdot N_t = 6.076 \cdot in^2$
Vertical Distance from Top of Deck	$y_{tendon} := 3.479ft$
Vertical Distance from Extreme Compression Fiber to to Centroid of Prestressing Tendons	$d_p := y_{tendon} = 3.479 \cdot ft$

Mild Reinforcement

Top Clearance	$c_t := 2.5in$
Bottom Clearance	$c_b := 1in$
Diameter of #8	$D_{N8} := 1in$
Diameter of #7	$D_{N7} := 0.875in$
Diameter of #5	$D_{N5} := 0.625in$
Diameter of #4	$D_{N4} := 0.5in$
Vertical Reinforcement Spacing in Web	$s_w := 8in$
Young's Modulus of Reinforcement	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	$40 - 14 = 26$	$A_{s.1} := 26 \cdot 0.6in^2 = 15.6 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	$30 - 14 = 16$	$A_{s.2} := 16 \cdot 0.6in^2 = 9.6 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 4 \cdot 0.2 \cdot in^2 + 4 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 4 \cdot 0.2 \cdot in^2 + 4 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 4 \cdot 0.2 \cdot in^2 + 4 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	36	$A_{s.8} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	36	$A_{s.9} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_t + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := h_{ft} + \frac{h_{fillet}}{2} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2}\right) = 47 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N8}}{2}\right) = 49 \cdot in$

4. Flexural Demand

The factored flexural resistance, M_r, must be greater than the Strength I load combination, M_u (ultimate moment), and the minimum reinforcement requirement. The minimum reinforcement requirement states that the flexural resistance must at least equal to $1.33M_u$ or the cracking moment, M_{cr}, whichever is less.

Factored Flexural Resistance

The M_u is: $M_u := 6542kip \cdot ft$

Note: this moment (Mu) is due to loads at service. The results is obtained from csibrdige "ULS-Envelope".

Minimum Reinforcement Requirement

$1.33M_u = 8.701 \times 10^3 \cdot kip \cdot ft$

Flexural Cracking Variability Factor (5.6.3.3) $\gamma_1 := 1.6$

Prestress Variability Factor (5.6.3.3) $\gamma_2 := 1.0$

Ratio of Specified Minimum Yield Strenght to Ultimate Tensile Stregth of the Mild Reinforcement (5.6.3.3) $\gamma_3 := 0.67$

Section Modulus for the Extreme Fiber $S_c := \frac{I_{22}}{h - y} = 46.905 \cdot ft^3$

Tendon Vertical Eccentricity $e := -d_p + y = -1.869 \cdot ft$

Compressive Stress in Concrete due to Effective Prestress Forces $f_{cpe} := \frac{-(A_{ps.total} \cdot f_{pe})}{A} + \frac{A_{ps.total} \cdot f_{pe} \cdot e}{S_c} = -0.448 \cdot ksi$

Cracking Moment (5.6.3.3) $M_{cr} := (\gamma_1 \cdot f_r - \gamma_2 \cdot f_{cpe}) \cdot \gamma_3 \cdot S_c = 5.915 \times 10^3 \cdot kip \cdot ft$

Therefore, the demand, D, is $D := \min[(|M_{cr}|), |1.33 M_u|] = 5.915 \times 10^3 \cdot kip \cdot ft$

5. Factored Flexural Resistance

Stress Block Factor (5.6.2.2) $\beta_1 := 0.85 - \frac{0.05}{ksi} \cdot (f_c - 4ksi) = 0.8$

Assuming:
1. The neutral axis is within the top slab
2. The compression reinforcements and reinforcements in the webs are ignored
3. The tensile reinforcements in the slab are assumed to reach the yield strength

Number of Plastic Hinges at Supports (5.6.3.1.2) $N_{s.ph} := 1$

East Span Length $l_{east} := 76.63ft$

West Span Length $l_{west} := 76.30ft$

Length of Tendon between Anchorages $l_i := l_{east} + l_{west} = 152.93 \cdot ft$

Effective Tendon Length $l_e := \frac{2 \cdot l_i}{2 + N_{s.ph}} = 101.953 \cdot ft$

from Equations 5.6.3.1.2-1 :
$f_{ps}=f_{pe}+900*(d_p-c/l_e)$ and $c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1 b)$

from 5.6.3.1.2-4:
$c=(A_{ps}f_{ps}+A_sf_s)/(0.85f_c\beta_1 b)$

By substituting 5.6.3.1.2-4 into 5.6.3.1.2-1 we can solve for f_{ps}

Solve for Average Stress in Prestressing Steel, f_{ps}

$f_{pe} = 179.724 \cdot ksi$ $A_{ps.total} = 6.076 \cdot in^2$ $A_{s.8} + A_{s.9} = 56.88 \cdot in^2$ $f_y = 60 \cdot ksi$

$d_p = 41.748 \cdot in$ $f_c = 5 \cdot ksi$ $\beta_1 = 0.8$ $b_t = 432.876 \cdot in$ $l_e = 1.223 \times 10^3 \cdot in$

$$\left[f_{ps} = 179.724\ \text{ksi} + 900\cdot\text{ksi}\cdot\left[\frac{41.748\ \text{in} - \frac{\left(6.076\ \text{in}^2\cdot f_{ps} + 56.88\text{in}^2\cdot 60\text{ksi}\right)}{[0.85\cdot 5\text{ksi}\cdot 0.8\cdot(432.876\ \text{in})]}}{1.223\times 10^3\ \text{in}} \right] \right] \text{solve}, f_{ps} \rightarrow 208.108\cdot\text{ksi}$$

Average prestressing strength

$f_{ps} := 208.108\text{ksi}$

Note: f.ps is less than f.py which is assumed to be 0.9*fpu=243ksi

Distance from Compression Extreme Fibre to Neutral Axis

$c := \frac{A_{ps.total}\cdot f_{ps} + \left[\left(A_{s.8} + A_{s.9}\right)\cdot f_y\right]}{0.85\cdot f_c\cdot\beta_1\cdot b_t} = 3.178\cdot\text{in}$

Depth of Equivalent Rectangular Stress Block

$a := \beta_1\cdot c = 0.212\cdot\text{ft}$

Tension-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$\varepsilon_{tl} := 0.005$

Compression-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$\varepsilon_{cl} := 0.002$

Net tensile strain in the extreme tension steel

$\varepsilon_t := 0.0225$

Note: Net tensile strain in the extreme tension steel are obtained from Response 2000 "Response-2000-west diaphragm outputs".

Flexural Resistance Factor (5.5.4.2)

$\phi := \min\left[0.9, 0.75 + \frac{0.15\cdot\left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}}\right] = 0.9$

Fexural Resistance, ϕM_n, for positive bending moment (5.6.3.2.3)

$$M_n := \left[A_{ps.total}\cdot f_{ps}\cdot\left(d_p - \frac{a}{2}\right) + \left(A_{s.8}\right)\cdot f_y\cdot\left(d_{s.8} - \frac{a}{2}\right) + A_{s.9}\cdot f_y\cdot\left(d_{s.9} - \frac{a}{2}\right)\right] = 1.755\times 10^4\cdot\text{kip}\cdot\text{ft}$$

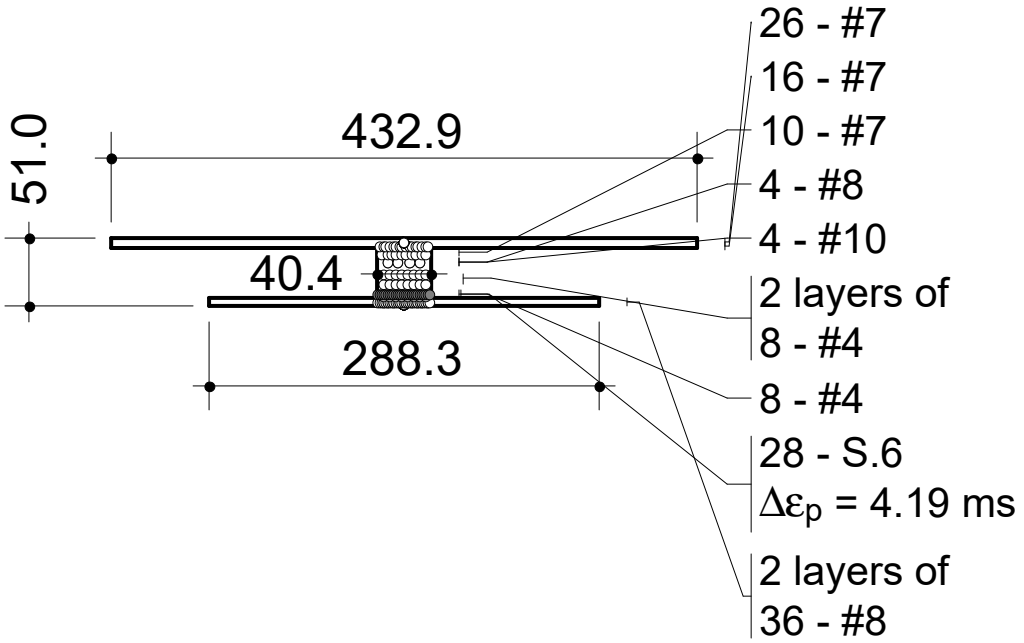
$\phi\cdot M_n = 1.58\times 10^4\cdot\text{kip}\cdot\text{ft}$

6 Summary

	Demand	Resistance	D/C Ratio
Minimum Reinforcement	$M_{cr} = 5.915\times 10^3\cdot\text{kip}\cdot\text{ft}$	$\phi\cdot M_n = 1.58\times 10^4\cdot\text{kip}\cdot\text{ft}$	$\frac{M_{cr}}{(\phi\cdot M_n)} = 0.374$
Ultimate Capacity	$M_u = 6.542\times 10^3\cdot\text{kip}\cdot\text{ft}$	$\phi\cdot M_n = 1.58\times 10^4\cdot\text{kip}\cdot\text{ft}$	$\frac{M_u}{(\phi\cdot M_n)} = 0.414$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6882.1	7636.6
Inertia (in^4)	2526654.2	2902240.0
y_t (in)	20.1	21.3
y_b (in)	30.9	29.7
S_t (in^3)	125950.3	136297.3
S_b (in^3)	81664.9	97697.0



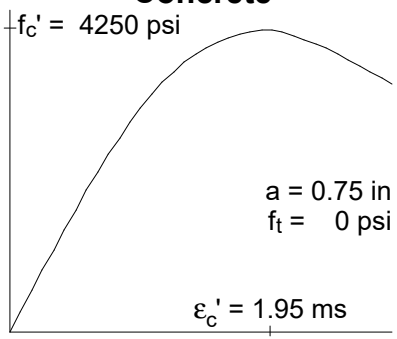
Crack Spacing

2 x dist + 0.1 d_b /ρ

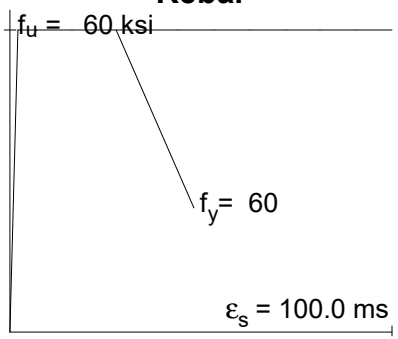
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

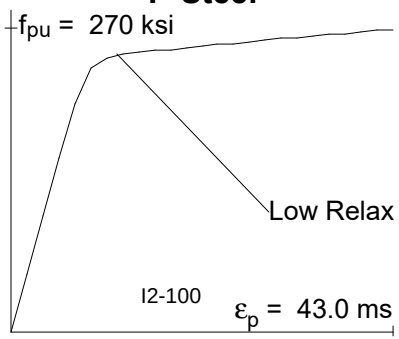
Concrete



Rebar



P-Steel



All dimensions in inches
Clear cover to reinforcement = 1.50 in

West Diaphragm Top Ope

I2-100 I2-100

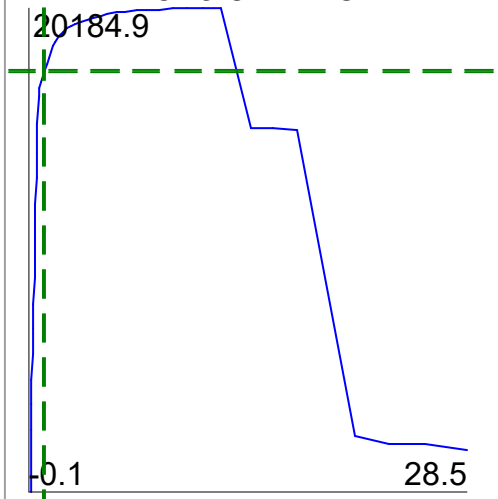
2021/4/5

Response-2000 v 1.0.5

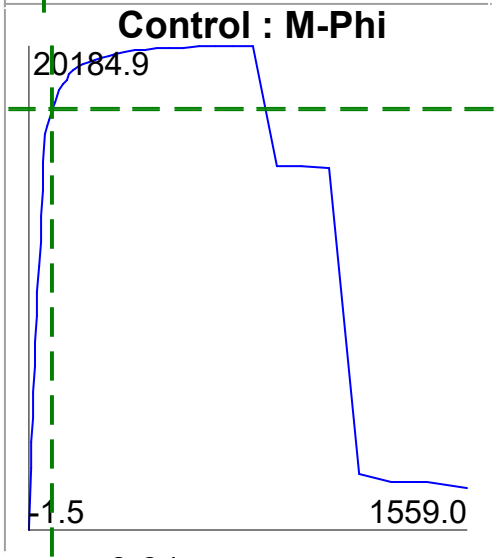
West Diaphragm Top Opening

2021/4/5 - 5:29 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.91 ms

ϕ = 81.22 rad/10^6 in

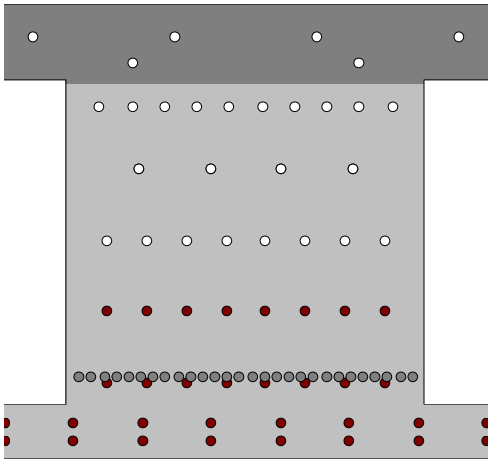
γ_{xy}(avg) = 0.00 ms

Axial Load = -1.4 kips

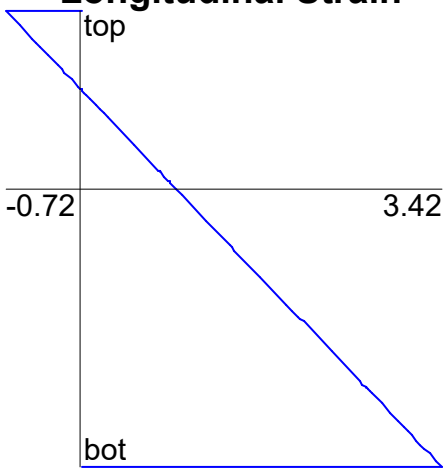
Moment:= 17587.1 ft-kips

Shear = 0.0 kips

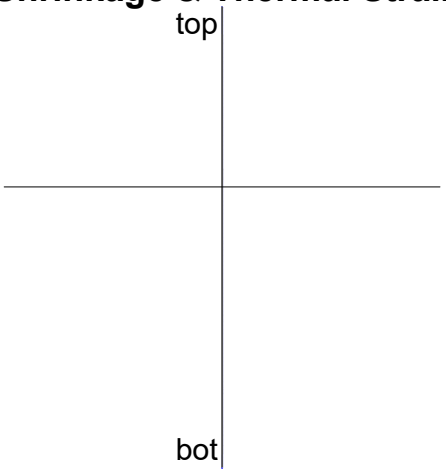
Cross Section



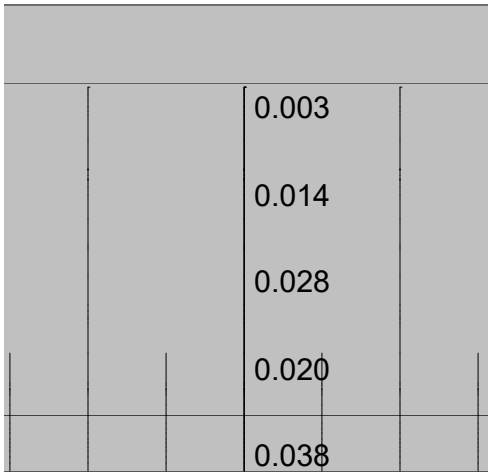
Longitudinal Strain



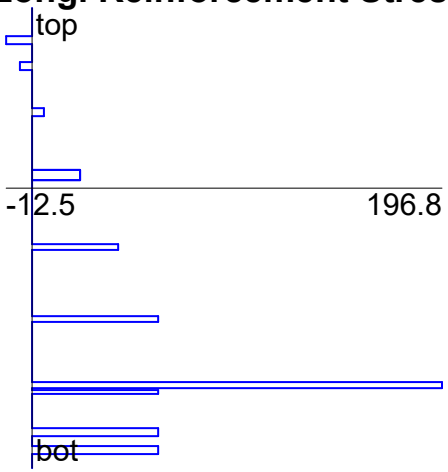
Shrinkage & Thermal Strain



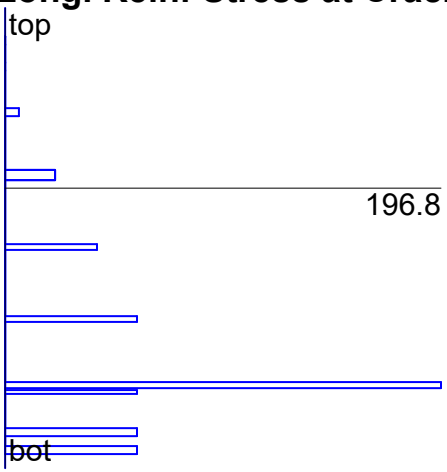
Crack Diagram



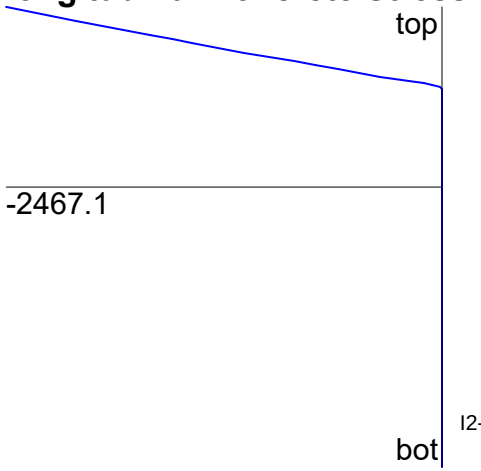
Long. Reinforcement Stress



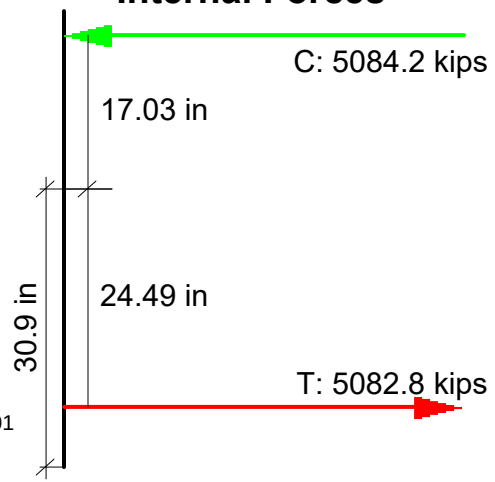
Long. Reinf Stress at Crack



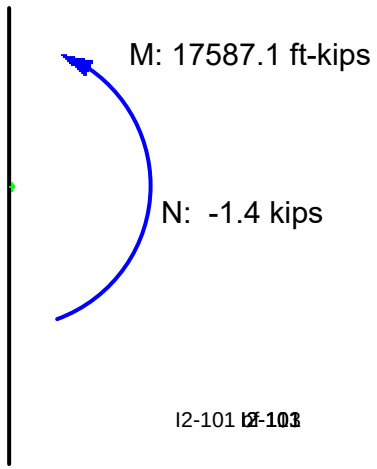
Longitudinal Concrete Stress



Internal Forces



N+M



COWI

SUBJECT: East Diaphragm Top Opening Check (In Construction) PROJECT:_ BR17.7
DESIGNED BY:_LDLI DATE: 10/26/2021
CHECKED BY:CUPG DATE:10/27/2021

1. Purpose and Assumptions

1. The purpose is to calculate and check the flexural capacity of the superstructure when there are openings in the top slab during construction. It is assumed that the tendon are not stressed, no reinforcement or concrete is within the demolition limit, and the bridge is subjected to dead load and construction live load
2. The check is at the centerline of the intermediate diaphragm in east span (41.63 ft from east abutment).
3. The calculation is based on AASHTO 8th Edition, 2017.
4. The deck flare near Pier 3 is ignored to be conservative

2. Material Properties

Concrete Compressive Strength $f_c := 5ksi$

Concrete Modulus of Fracture (5.4.2.6) $f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$

Reinforcement Tensile Yield Strength $f_y := 60ksi$

Reinforcement Compressive Yield Strength $f_{y'} := 60ksi$

3. Structural Properties

Girder

Girder Type: Box girder

Number of Cells: 3

Area of top slab (excluding fillets) $A_{ts} := 24.7ft^2$

note: measured using bluebeam. Measurements are shown in "typ cross-section" in folder "top slab opening".

Width of right opening $b_{ro} := (9ft + 4.5in) - 6in - 4.5in - 1.5ft - 11.18in = 6.068 \cdot ft$

Width of left opening $b_{lo} := (9ft + 4.625in) - 6in - 4.5in - 1.5ft - 11.18in = 6.079 \cdot ft$

note: the the width of the opening is between the edges of fillets of exterior cells as shown in file "typ cross-section".

Top Slab Thickness $h_{ft} := 8.5in$

Bottom Slab Thickness $h_{fb} := 6.5in$

Area of top slab (excluding fillets and openings) $A_{ts'} := A_{ts} - (b_{ro} + b_{lo}) \cdot h_{ft} = 16.096 \cdot ft^2$

Equivalent Top Slab Width $b_t := \frac{A_{ts'}}{h_{ft}} = 272.682 \cdot in$

Bottom Slab Width $b_b := 24ft + 0.25in = 288.25 \cdot in$

Web Thickness $b_w := 11.18in \cdot 2 + 9in \cdot 2 = 40.36 \cdot in$

Total Depth $h := 4.25ft$

Cross-section Moment of Inertia about Horizontal Axis $I_{22} := 103.05ft^4$

Vertical Distance from Top of Deck to Centroid $y := 2.03ft$

Cross-section Area $A := 52.739ft^2 - [(b_{ro} + b_{lo}) \cdot h_{ft}] = 44.135 \cdot ft^2$

Height of the Fillet $h_{fillet} := 0.5ft$

note: moment of inertia and centroid is calculated by assuming the box-girder is equivalent to a wide flange cross-section with top width of b.t and bottom width of b.b, top thickness of h.ft, bottom flange thickness of h.fb, web height of h-h.ft-h.fb, and web thickness of b.w. The assumed cross-section can be found in the corresponding response 2000 file in folder "top slab opening".

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #10 $D_{N10} := 1.27in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

Young's Modulus of Reinforcement $E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	$37 - 12 = 25$	$A_{s.1} := 25 \cdot 0.6in^2 = 15 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	$27 - 12 = 15$	$A_{s.2} := 15 \cdot 0.6in^2 = 9 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#10	40	$A_{s.8} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#10	40	$A_{s.9} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N10}}{2} \right) = 46.635 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N10}}{2} \right) = 48.865 \cdot in$

4. Flexural Demand

The factored flexural resistance, M_r, must be greater than the Strength I load combination, M_u (ultimate moment), and the minimum reinforcement requirement. The minimum reinforcement requirement states that the flexural resistance must at least equal to $1.33M_u$ or the cracking moment, M_{cr}, whichever is less.

Factored Flexural Resistance

The M_u is:

$$M_{u.east} := 6400 kip \cdot ft$$

Note: The results is obtained from csibrdige "Construction in model "20211102 pier 3 vert spring changed stage 2.5 added_withconstruction". This moment (Mu) is due to self weight of the box-girder, sidewalk and barrier , tendon, and construction equipment. As per 3.4.2, the load factor for DC is taken as 1.25 and load factor of construction load is 1.5.
In this model, the density of the superstructure is increased from 0.155ksi to 0.165 ksi to account for the weight of tendons and the weight of the construction equipment (~0.010ksf as per AASHTO5.12.5.3.2) is added as a area load on the bridge deck.

Minimum Reinforcement Requirement

$$1.33M_{u.east} = 8.512 \times 10^3 \cdot kip \cdot ft$$

Flexural Cracking Variability Factor (5.6.3.3) $\gamma_1 := 1.6$

Ratio of Specified Minimum Yield Strenght to Ultimate Tensile Stregth of the Mild Reinforcement (5.6.3.3) $\gamma_3 := 0.67$

Section Modulus for the Extreme Fiber $S_c := \frac{I_{22}}{h - y} = 46.419 \cdot ft^3$

Cracking Moment (5.6.3.3) $M_{cr} := \left(\gamma_1 \cdot f_r\right) \cdot \gamma_3 \cdot S_c = 3.845 \times 10^3 \cdot kip \cdot ft$

Therefore, the demand, D, is $D := \min\left[\left(\left|M_{cr}\right|\right), \left|1.33M_{u.east}\right|\right] = 3.845 \times 10^3 \cdot kip \cdot ft$

5. Factored Flexural Resistance

Stress Block Factor (5.6.2.2) $\beta_1 := 0.85 - \frac{0.05}{ksi} \cdot \left(f_c - 4ksi\right) = 0.8$

Assuming:
1. The neutral axis is within the top slab
2.The compression reinforcements and reinforcements in webs are ignored
3. The tension reinforcements in the slab are assumed to reach the yield strength

Distance from Compression Extreme Fibre to Neutral Axis

$$c := \frac{\left[\left(A_{s.8} + A_{s.9}\right)\cdot f_y\right]}{0.85\cdot f_c\cdot \beta_1\cdot b_t} = 6.575\cdot in$$

Depth of Equivalent Rectangular Stress Block

$$a := \beta_1\cdot c = 0.438\cdot ft$$

Tension-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$$\varepsilon_{tl} := 0.005$$

Compression-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$$\varepsilon_{cl} := 0.002$$

Net tensile strain in the extreme tension steel

$$\varepsilon_t := 0.0145$$

Note: Net tensile strain in the extreme tension steel are obtained from Response 2000 "Response-2000-east diaphragm (results)".

Flexural Resistance Factor (5.5.4.2)

$$\phi := \min\left[0.9, 0.75 + \frac{0.15\cdot\left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}}\right] = 0.9$$

Fexural Resistance, ϕM_n, for positive bending moment (5.6.3.2.3)

$$M_n := \left[\left(A_{s.8}\right)\cdot f_y\cdot\left(d_{s.8} - \frac{a}{2}\right) + A_{s.9}\cdot f_y\cdot\left(d_{s.9} - \frac{a}{2}\right)\right] = 2.292\times 10^4\cdot kip\cdot ft$$

$$\phi\cdot M_n = 2.063\times 10^4\cdot kip\cdot ft$$

6 Check from Response 2000

Response 2000 Resistance

$$M_{n.2000} := 25050 kip\cdot ft$$

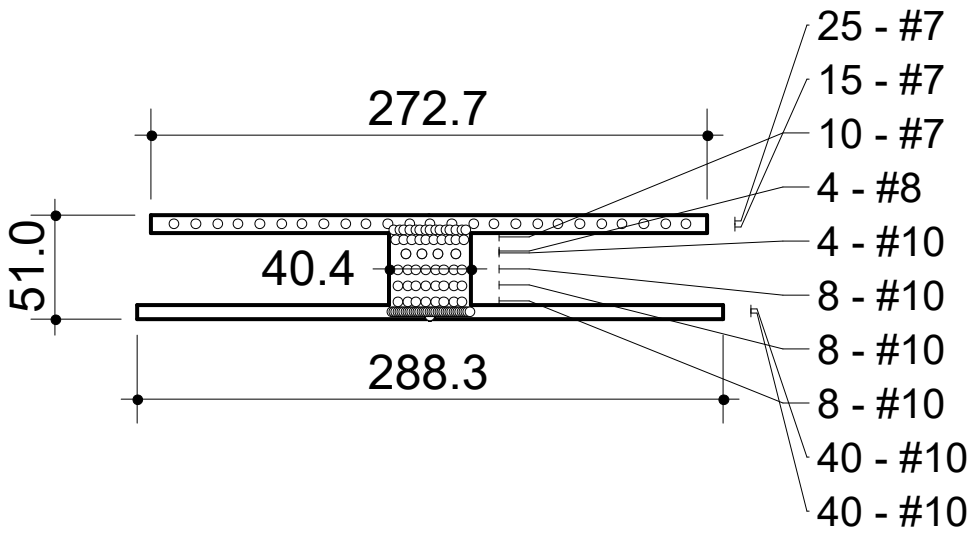
$$difference := \frac{\left(M_{n.2000} - M_n\right)}{M_{n.2000}} = 0.085$$

7 Summary

	Demand	Resistance	D/C Ratio
Minimum Reinforcement	$M_{cr} = 3.845\times 10^3\cdot kip\cdot ft$	$\phi\cdot M_n = 2.063\times 10^4\cdot kip\cdot ft$	$\frac{M_{cr}}{\left(\phi\cdot M_n\right)} = 0.186$
Ultimate Capacity	$M_{u.east} = 6.4\times 10^3\cdot kip\cdot ft$	$\phi\cdot M_n = 2.063\times 10^4\cdot kip\cdot ft$	$\frac{M_{u.east}}{\left(\phi\cdot M_n\right)} = 0.31$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	5644.4	6843.7
Inertia (in^4)	2146496.8	2612090.9
y_t (in)	24.4	26.6
y_b (in)	26.6	24.4
S_t (in^3)	87909.4	98368.3
S_b (in^3)	80747.4	106852.3



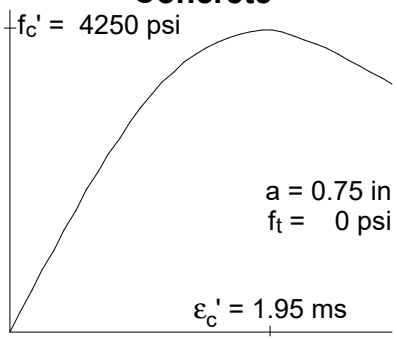
Crack Spacing

2 x dist + 0.1 d_b /ρ

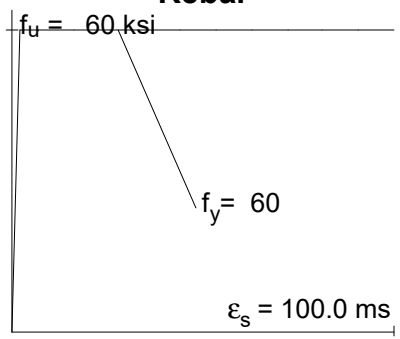
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.50 in

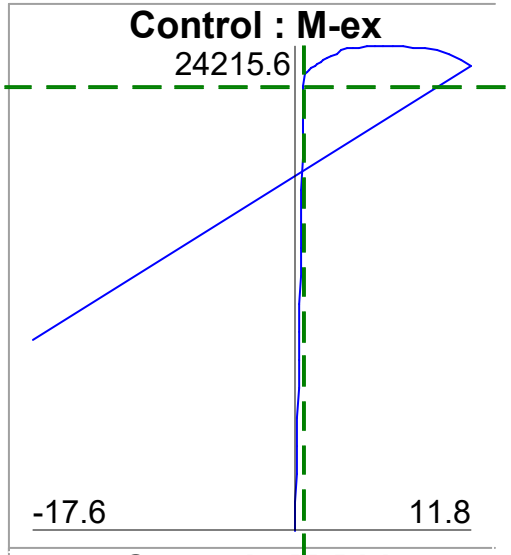
East Diaphragm Top Opening

Response-2000 v 1.0.5

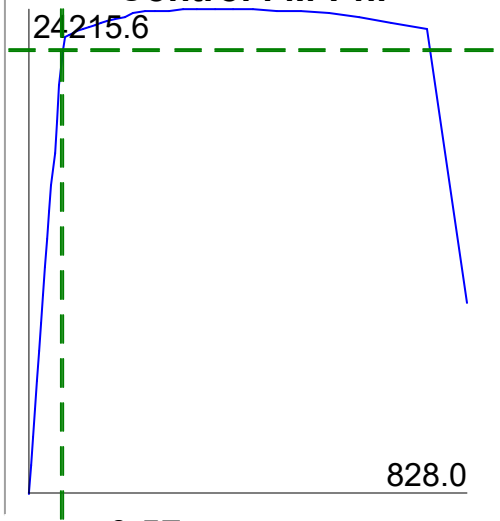
East Diaphragm Top Opening

2021/3/31 - 5:01 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.57 ms

ϕ = 63.16 rad/10^6 in

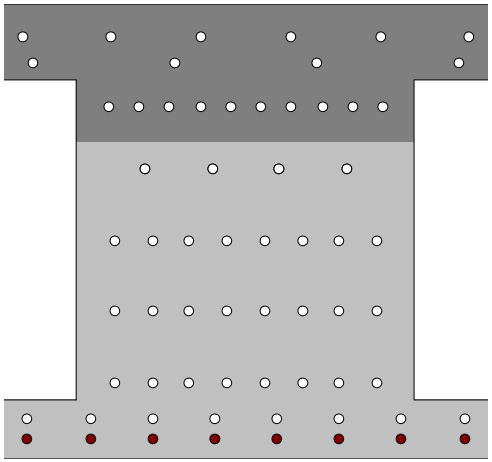
γ_{xy}(avg) = 0.00 ms

Axial Load = 0.0 kips

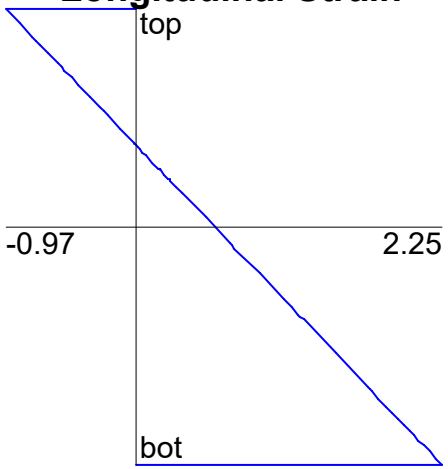
Moment:= 22211.3 ft-kips

Shear = 0.0 kips

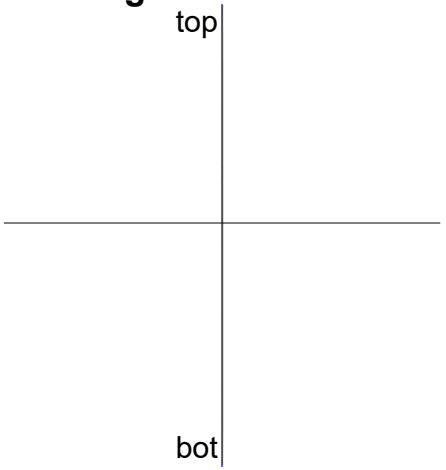
Cross Section



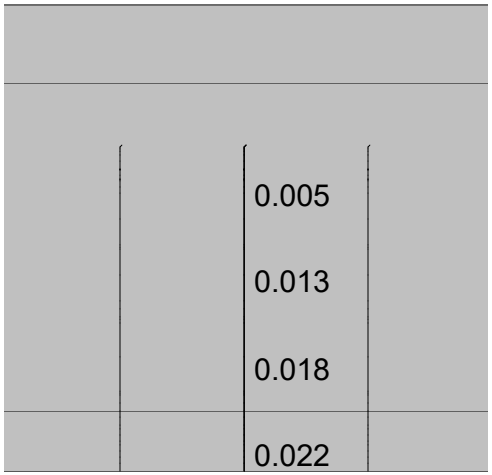
Longitudinal Strain



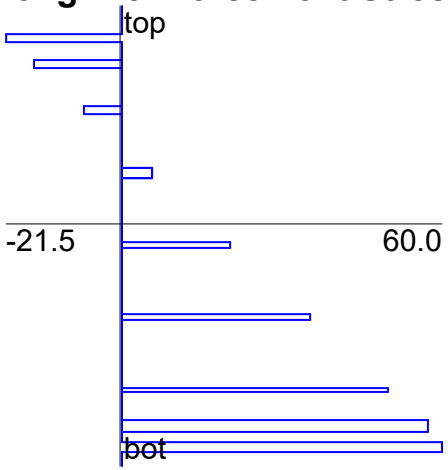
Shrinkage & Thermal Strain



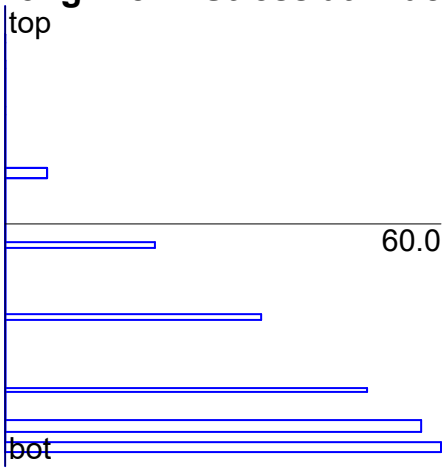
Crack Diagram



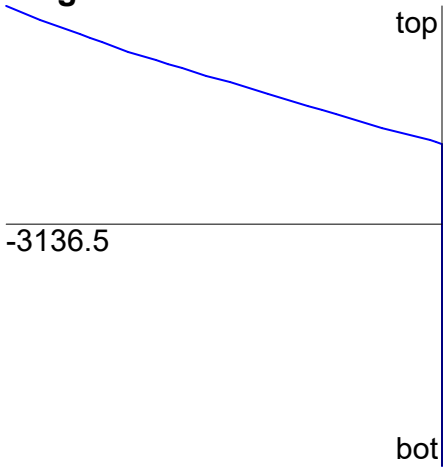
Long. Reinforcement Stress



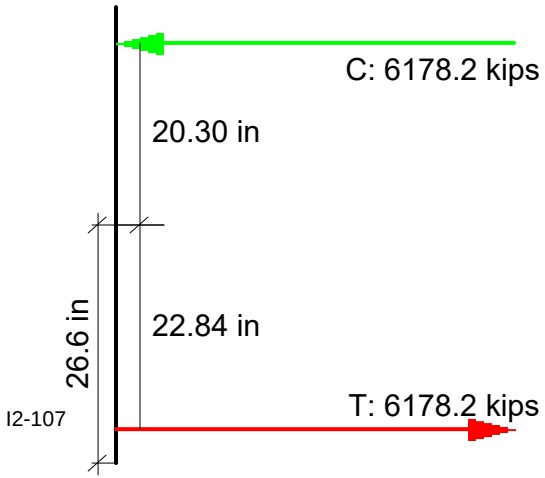
Long. Reinf Stress at Crack



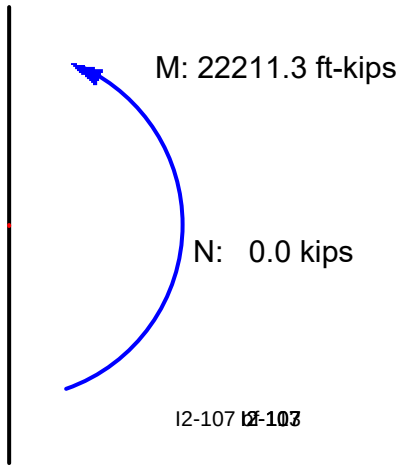
Longitudinal Concrete Stress



Internal Forces



N+M



COWI

SUBJECT: West Diaphragm Top Opening (In Construction) PROJECT:_ BR17.7
DESIGNED BY:_LDLI DATE:_10/27/2021
CHECKED BY:CUPG DATE: 10/27/2021

1. Purpose and Assumptions

1. The purpose is to calculate and check the flexural capacity of the superstructure when there are openings in the top slab during construction. It is assumed that the tendon are not stressed, no reinforcement or concrete is within the demolition limit, and the bridge is subjected to dead load and construction live load
2. The check is at the centerline of the intermediate diaphragm in west span (105.63 ft from east abutment).
3. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$

3. Structural Properties

Girder

Girder Type	Box girder	
Number of Cells	3	
Area of top slab (excluding fillets)	$A_{ts} := 24.7ft^2$	note: measured using bluebeam. Measurements are shown in "typ cross-section" in folder "top slab opening".
Width of right opening	$b_{ro} := (9ft + 4.5in) - 6in - 4.5in - 1.5ft - 11.18in = 6.068 \cdot ft$	note: the the opening is shown as the highlighted region in "typ cross-section" in folder "top slab opening".
Width of left opening	$b_{lo} := (9ft + 4.625in) - 6in - 4.5in - 1.5ft - 11.18in = 6.079 \cdot ft$	
Top Slab Thickness	$h_{ft} := 8.5in$	
Bottom Slab Thickness	$h_{fb} := 6in$	
Area of top slab (excluding fillets and openings)	$A_{ts'} := A_{ts} - (b_{ro} + b_{lo}) \cdot h_{ft} = 16.096 \cdot ft^2$	
Equivalent Top Slab Width	$b_t := \frac{A_{ts'}}{h_{ft}} = 22.724 \cdot ft$	
Bottom Slab Width	$b_b := 24ft + 0.25in = 24.021 \cdot ft$	note: moment of inertia and centroid is calculated by assuming the box-girder is equivalent to a wide flange cross-section with top width of b.t and bottom width of b.b, top thickness of h.ft, bottom flange thickness of h.fb, web height of h-h.ft-h.fb, and web thickness of b.w. The assumed cross-section can be found in the corresponding response 2000 file in folder "top slab opening".
Web Thickness	$b_w := 11.18in \cdot 2 + 9in \cdot 2 = 40.36 \cdot in$	
Total Depth	$h := 4.25ft$	
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 100.495ft^4$	
Vertical Distance from Top of Deck to Centroid	$y := 1.99ft$	
Cross-section Area	$A := 51.842\,ft^2 - [(b_{ro} + b_{lo}) \cdot h_{ft}] = 43.238 \cdot ft^2$	
Height of the Fillet	$h_{fillet} := 0.5ft$	

Mild Reinforcement

Top Clearance	$c_t := 2.5in$
Bottom Clearance	$c_b := 1in$
Diameter of #8	$D_{N8} := 1in$
Diameter of #7	$D_{N7} := 0.875in$
Diameter of #5	$D_{N5} := 0.625in$
Diameter of #4	$D_{N4} := 0.5in$
Vertical Reinforcement Spacing in Web	$s_w := 8in$
Young's Modulus of Reinforcement	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	$40 - 14 = 26$	$A_{s.1} := 26 \cdot 0.6in^2 = 15.6 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	$30 - 14 = 16$	$A_{s.2} := 16 \cdot 0.6in^2 = 9.6 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 4 \cdot 0.2 \cdot in^2 + 4 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 4 \cdot 0.2 \cdot in^2 + 4 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 4 \cdot 0.2 \cdot in^2 + 4 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	36	$A_{s.8} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	36	$A_{s.9} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1)	$d_{s.1} := c_t + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$
Bar in Top Slab (Layer 2)	$d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$
Bar in Top Slab (Layer 3)	$d_{s.3} := h_{ft} + \frac{h_{fillet}}{2} = 11.5 \cdot in$
Bar in Web (Layer 1)	$d_{s.4} := h - 32.5in = 18.5 \cdot in$
Bar in Web (Layer 2)	$d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$
Bar in Web (Layer 3)	$d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2} \right) = 47 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N8}}{2} \right) = 49 \cdot in$

The factored flexural resistance, M_r, must be greater than the Strength I load combination, M_u (ultimate moment), and the minimum reinforcement requirement. The minimum reinforcement requirement states that the flexural resistance must at least equal to $1.33M_u$ or the cracking moment, M_{cr}, whichever is less.

Factored Flexural Resistance

The M_u is:

$$M_u := 4204 kip \cdot ft$$

Note: The results is obtained from csibrdige "Construction in model "20211102 pier 3 vert spring changed stage 2.5 added_with construction". This moment (Mu) is due to self weight of the box-girder, sidewalk and barrier , tendon, and construction equipment. As per 3.4.2, the load factor for DC is taken as 1.25 and load factor of construction load is 1.5. In this model, the density of the superstructure is increased from 0.155ksi to 0.165 ksi to account for the weight of tendons and the weight of the construction equipment (~0.010ksf as per AASHTO5.12.5.3.2) is added as a area load on the bridge deck.

Minimum Reinforcement Requirement

$$1.33M_u = 5.591 \times 10^3 \cdot kip \cdot ft$$

Flexural Cracking Variability Factor (5.6.3.3) $\gamma_1 := 1.6$

Ratio of Specified Minimum Yield Strenght to Ultimate Tensile Stregth of the Mild Reinforcement (5.6.3.3) $\gamma_3 := 0.67$

Section Modulus for the Extreme Fiber $S_c := \frac{I_{22}}{h - y} = 44.467 \cdot ft^3$

Cracking Moment (5.6.3.3) $M_{cr} := \left(\gamma_1 \cdot f_r \right) \cdot \gamma_3 \cdot S_c = 3.684 \times 10^3 \cdot kip \cdot ft$

Therefore, the demand, D, is $D := \min\left[\left(\left| M_{cr} \right| \right), \left| 1.33M_u \right| \right] = 3.684 \times 10^3 \cdot kip \cdot ft$

5. Factored Flexural Resistance

Stress Block Factor (5.6.2.2) $\beta_1 := 0.85 - \frac{0.05}{ksi} \cdot \left(f_c - 4ksi \right) = 0.8$

Assuming:
1. The neutral axis is within the top slab
2.The compression reinforcements and reinforcements in webs are ignored

3. The tension reinforcements in the slab are assumed to reach the yield strength

Distance from Compression Extreme Fibre to Neutral Axis

$$c := \frac{\left[\left(A_{s.8} + A_{s.9}\right)\cdot f_y\right]}{0.85\cdot f_c\cdot \beta_1\cdot b_t} = 3.681\cdot in$$

Depth of Equivalent Rectangular Stress Block

$$a := \beta_1\cdot c = 0.245\cdot ft$$

Tension-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$$\varepsilon_{tl} := 0.005$$

Compression-controlled strain in the extreme tension steel at nominal resistance (Table C5.6.2.1-1)

$$\varepsilon_{cl} := 0.002$$

Net tensile strain in the extreme tension steel

$$\varepsilon_t := 0.0225$$

Note: Net tensile strain in the extreme tension steel are obtained from Response 2000 "Response-2000-west diaphragm (results)".

Flexural Resistance Factor (5.5.4.2)

$$\phi := \min\left[0.9, 0.75 + \frac{0.15\cdot\left(\varepsilon_t - \varepsilon_{cl}\right)}{\varepsilon_{tl} - \varepsilon_{cl}}\right] = 0.9$$

Fexural Resistance, ϕM_n, for positive bending moment (5.6.3.2.3)

$$M_n := \left[\left(A_{s.8}\right)\cdot f_y\cdot\left(d_{s.8} - \frac{a}{2}\right) + A_{s.9}\cdot f_y\cdot\left(d_{s.9} - \frac{a}{2}\right)\right] = 1.323\times 10^4\cdot kip\cdot ft$$

$$\phi\cdot M_n = 1.191\times 10^4\cdot kip\cdot ft$$

6 Check from Response 2000

Response 2000 Resistance

$$M_{n.2000} := 13632.5 kip\cdot ft$$

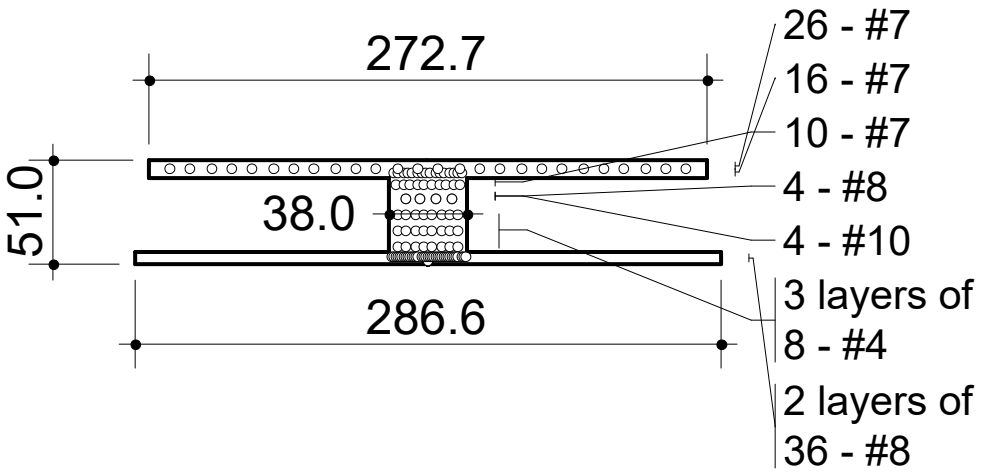
$$difference := \frac{\left(M_{n.2000} - M_n\right)}{M_{n.2000}} = 0.029$$

7 Summary

	Demand	Resistance	D/C Ratio
Minimum Reinforcement	$M_{cr} = 3.684\times 10^3\cdot kip\cdot ft$	$\phi\cdot M_n = 1.191\times 10^4\cdot kip\cdot ft$	$\frac{M_{cr}}{\left(\phi\cdot M_n\right)} = 0.309$
Ultimate Capacity	$M_u = 4.204\times 10^3\cdot kip\cdot ft$	$\phi\cdot M_n = 1.191\times 10^4\cdot kip\cdot ft$	$\frac{M_u}{\left(\phi\cdot M_n\right)} = 0.353$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	5424.5	6136.6
Inertia (in^4)	2078135.3	2385512.7
y_t (in)	23.9	24.8
y_b (in)	27.1	26.2
S_t (in^3)	87053.6	96137.1
S_b (in^3)	76604.6	91097.6



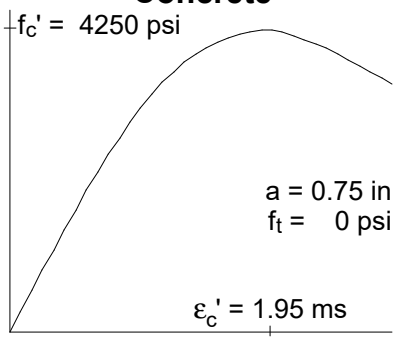
Crack Spacing

2 x dist + 0.1 d_b /ρ

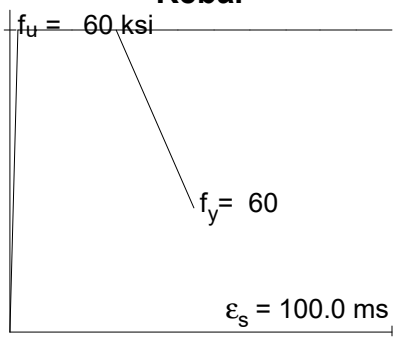
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.50 in

West Span Opening

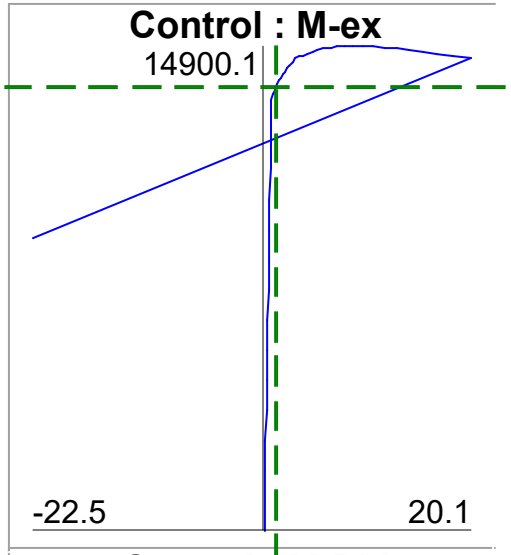
2021/3/31

Response-2000 v 1.0.5

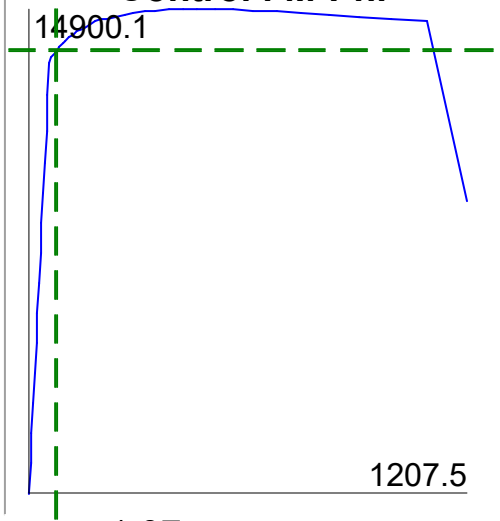
West Span Opening

2021/3/31 - 11:30 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.07 ms

ϕ = 76.12 rad/10^6 in

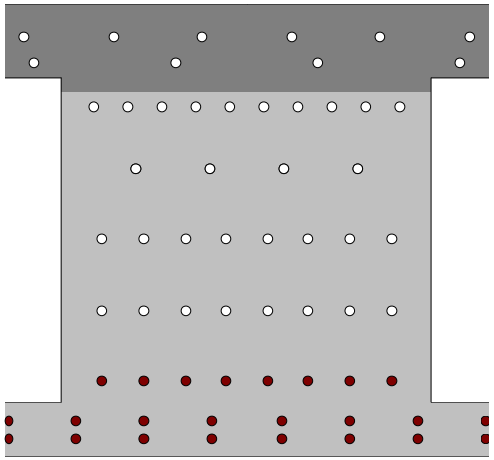
γ_{xy}(avg) = 0.00 ms

Axial Load = -0.1 kips

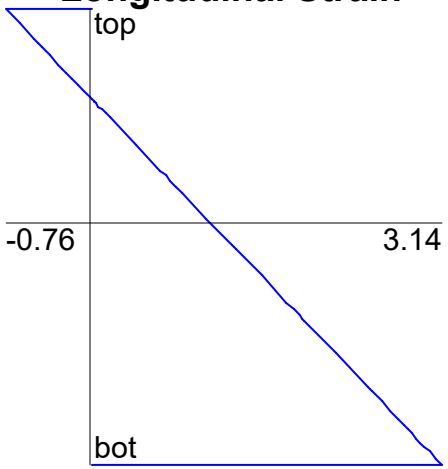
Moment:= 13632.5 ft-kips

Shear = 0.0 kips

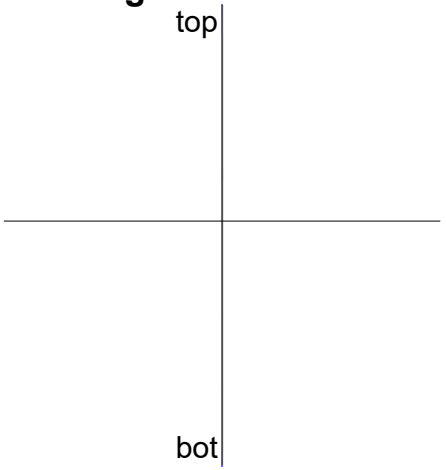
Cross Section



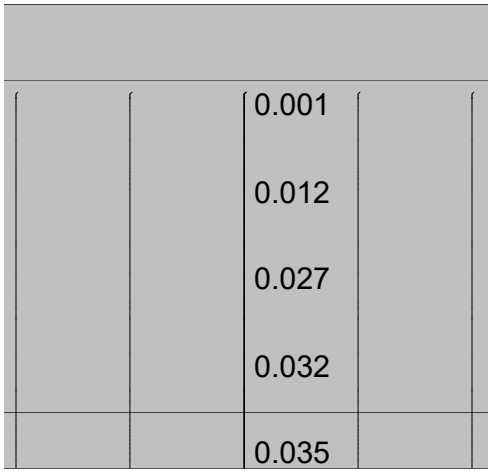
Longitudinal Strain



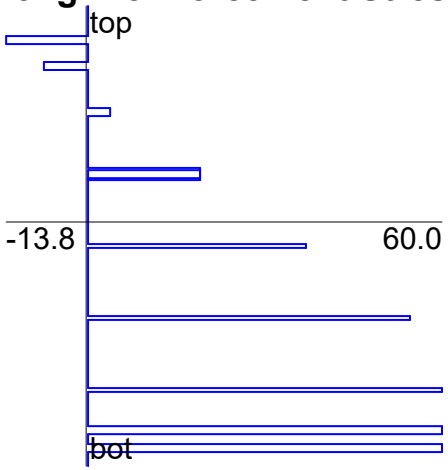
Shrinkage & Thermal Strain



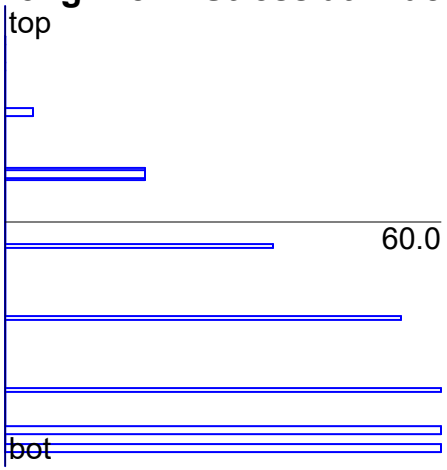
Crack Diagram



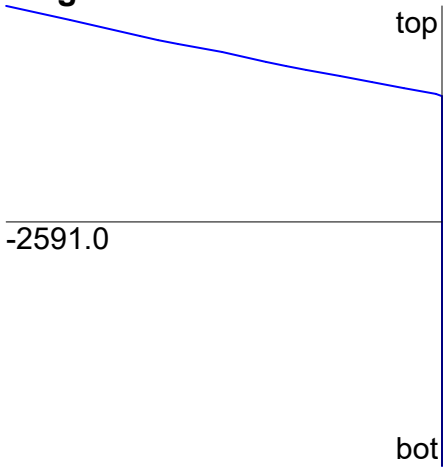
Long. Reinforcement Stress



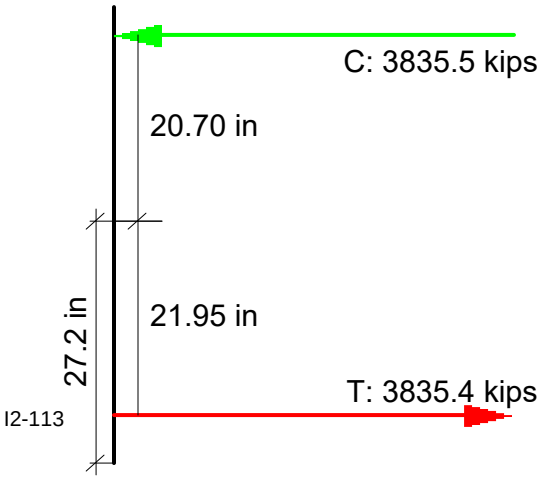
Long. Reinf Stress at Crack



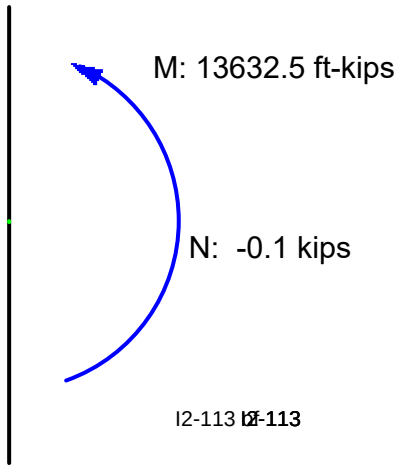
Longitudinal Concrete Stress



Internal Forces



N+M



COWI

PROJECT	I-405 R2B: Bridge 17.7			CONT	A207833
SUBJECT	Superstructure Longitudinal Reinforcement Design -Development Lengths			PAGE	
CALCS BY	LDLI	DATE 2021-10-29	CHECK BY FCCA	DATE	2021-10-29

Calculation Description: The calculation below is done to determine the development lengths and hook development lengths

Summary:

- Calculate development and splice lengths of reinforcing bars in tension
- Based on AASHTO LRFD Bridge Design Specification, 8th Edition, 2017
- Minimum Splice 12.0" length, AASHTO BDS APPENDIX 5.1-A4

Input:

User input=			
f'c =	5.0	ksi	concrete 28-day compressive strength
λ =	1.0		Normal weight concrete AASHTO BDS 5.4.2.8
A_{tr}=	0.0		Transverse reinforcement is ignored as per C5.10.8.2.1.c
n=	2.0		AASHTO BDS Fig.C5.10.8.2.1c-1
K_{tr}=	0.0		AASHTO BDS Eq.5.10.8.2.1c-3. This is conservatively taken as zero
Splice Class=	B		
Splice factor=	1.3		AASHTO BDS Eq.5.10.8.4.3a

DEVELOPMENT AND SPLICE LENGTH FOR FLEXURAL REINFORCEMENT

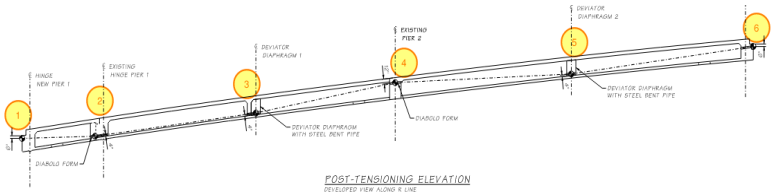
References(AASHTO BDS 2017)								Eq.5.10.8.2.1a-2	5.10.8.2.1b	Fig.C5.10.8.2.1c-1	Eq.5.10.8.2.1c-1 Eq.5.10.8.2.1c-2	5.10.8.2.1b					5.10.8.2.3	5.10.8.2.4b	Eq.5.10.8.2.1a-1
Location		**Size**	**Spacing [in]**	**Epoxy Coated?**	**Clear Cover [in]**	**Yield strength of reinforcement, fy (Ksi)**	**Bar Dia, d_b [in]**	**Basic development Length, l_{db} [in]**	**λ_{rl}**	**c_b [in]**	**λ_{rc}**	**Clear Cover< 3 d_b?**	**Clear Spacing< 6 d_b?**	**λ_{cf}**	**$\lambda_{rl} \times \lambda_{cf} <= 1.7$**	**No.of Bundled Bars**	**Bundled bars Factor**	**λ_{cr}**	**Tension development l_d [in]**
Deck	Longitudinal-Top Rebar	#7	13	YES	3.125	60	0.875	56.3	1	3.56	0.40	NO	NO	1.20	1.20	0	1.0	0.5	14.0
Deck	Longitudinal-Bottom Rebar	#7	13	YES	1.625	60	0.875	56.3	1	2.06	0.42	YES	NO	1.50	1.50	0	1.0	0.5	18.0
Bottom Slab (Typ.)	Longitudinal-Top Rebar	#8	10	NO	1.5	60	1.00	64.4	1	2.00	0.50	YES	NO	1.00	1.00	0	1.0	0.5	17.0
	Longitudinal-Bot Rebar	#8	10	NO	1.5	60	1.00	64.4	1	2.00	0.50	YES	NO	1.00	1.00	0	1.0	0.5	17.0
Ext Web	Longitudinal - Layer 1	#8	5	NO	2.125	60	1.00	64.4	1	2.50	0.40	YES	YES	1.00	1.00	0	1.0	0.5	13.0
	Longitudinal - Layers 2-4	#4	5	NO	2.125	60	0.50	32.2	1	2.38	0.40	NO	NO	1.00	1.00	0	1.0	0.5	12.0
Int Web	Longitudinal - Layer 1	#8	5	NO	2.125	60	1.00	64.4	1	2.50	0.40	YES	YES	1.00	1.00	0	1.0	0.5	13.0
	Longitudinal - Layers 2-4	#4	5	NO	2.125	60	0.50	32.2	1	2.38	0.40	NO	NO	1.00	1.00	0	1.0	0.5	12.0

COWI

Project	I-405 Bridge 17.7	**Project No**	A207833
Title	Tendon 1 Losses	**Date**	11/30/2021
Prepared by	LDLI	**Checked by**	BOHU

This spreadsheet summarizes the tendon profile and calculates the stresses, losses, and elongations for Tendon 1. Note that:
1. The tendon profile used in the model is based on dwg located at:
XL5467_27_DE_BG_BR17.7-PostTensioningLayout.pdf
The geometric calculations for the tendons are included in Calc Book I-11

VERTICAL LAYOUT



Location Ref. #	Length from WP to stressing end (ft)	Offset from deck top (ft)
1	0.0	1.52
2	11.9	3.35
3	47.5	3.35
4	78.4	1.06
5	117.0	3.31
6	157.2	1.52

note: the locations of deviators were taken at the centerline of the deviators

TENDON PROPERTIES

Nominal Area	1.519	in^2
Ep	28500	ksi
fpu	270	ksi
fy	202.5	ksi
Jacking Stress	218.70	ksi
Jacking/Ultimate	0.81	

note: jacking force is is the minimum between the 0.81fpu and 0.74fpu+anchorage loss. Anchorage loss has to be manually adjusted in the formula to match with the value in cell F139

LOSSES

Anchorage Set

Anchorage set	0.375	in
Anchorage set	0.0313	ft

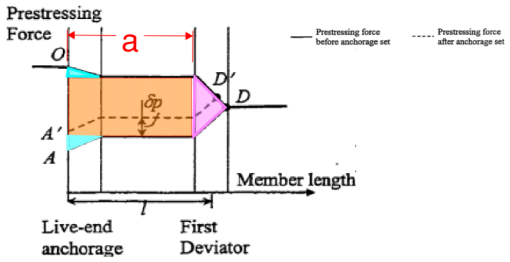
AASHTO C5.9.3.2.1

Tendon Stress before Anchorage Set

Location Ref. #	Diaphragm Thickness (in)	Wobble Coefficient (AASHTO Table 5.9.3.2.2b-1)	Angle change (Rad)	μ (AASHTO Table 5.9.3.2.2b-1)	Friction Loss (ksi)	Tendon Force after Friction AASHTO eq. 5.9.3.2.2b-2 (ksi)
1	35	0.0002			1.5	217.2
2	24	0.0002	0.21	0.23	11.2	206.0
3	36	0.0002	0.18	0.23	10.6	195.5
4	30	0.0002	0.14	0.23	8.4	187.1
5	24	0.0002	0.09	0.23	5.5	181.5
6	33	0.0002			1.4	180.1

Tendon Stress after Anchorage Set

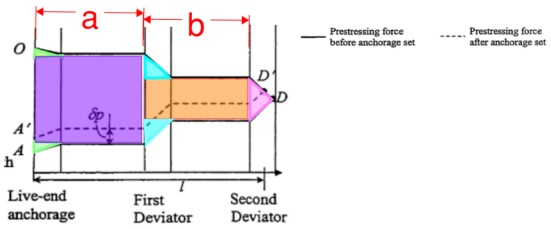
Step 1: Assuming the anchorage set effect ends at the first deviator



Clear Spacing btw Diaphragms	102	in	*note: measured along bridge cl*
a	137	in	
Area 1	3055.00	ksi.in	*note: shaded in orange in sketch above*
Area 2	52.26	ksi.in	*note: shaded in blue in sketch above*
Area 3	268.33	ksi.in	*note: shaded in pink in sketch above*
Area OAD	3375.58	ksi.in	
Δ_{calc}	0.12	in	

is Δcalc>Δset? no

Step 2: Assuming the anchorage set effect ends at the second deviator

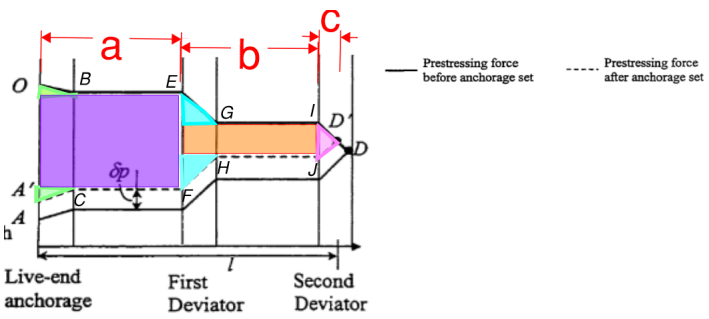


Clear Spacing btw Diaphragms 2 and 3	409.6	in	
b	421.6	in	
a	136.625	in	
Area 1	8903.16	ksi.in	*note: shaded in orange in sketch above*
Area 2	268.33	ksi.in	*note: shaded in blue in sketch above*
Area 3	380.12	ksi.in	*note: shaded in pink in sketch above*
Area 4	5940.2	ksi.in	*note: shaded in purple in sketch above*
Area 5	52.26	ksi.in	*note: shaded in green in sketch above*
Area OAD	15544.12	ksi.in	

Δ_{calc} = A_{OAD}/E_p = 0.55 in

is Δcalc>Δset? yes

Step 3: Find exact location of l (where friction due to anchorage set equals zero)



c		21.7	in	*note: this needs to be manually adjusted so that Δcalc meets the target*
friction loss within c		6.43	ksi	*note: assume friction loss increase linearly within the concrete*
b		421.5905329	in	
a		136.625	in	
Area 1		5419.28	ksi.in	*note: in orange*
Area 2		268.33	ksi.in	*note: in blue*
Area 3		139.47	ksi.in	*note: in pink*
Area 4		4811.2	ksi.in	*note: in purple*
Area 5		52.26	ksi.in	*note: in green*
Area OAD		10690.56	ksi.in	
Δ_{calc} =	A_{OAD}/E_p =	0.375	in	
Target Δ_{calc}		0.375	in	

Therefore, the losses of tendon stress due to anchorage set are:

@ west face of west diaphragm=	OA'=	38.23	ksi
@ east face of end diaphragm=	BC=	35.21	ksi
@ west face of diaphragm #2=	EF=	35.21	ksi
@ east face of diaphragm #2=	GH=	12.85	ksi
@ west face of diaphragm #3=	IJ=	12.85	ksi

Elastic Shortening of Concrete

Tendon longitudinal deformation when stressing

0.094 in — *Note: the deformation is from csibridge model, load case "PT1"*

Strain= 5.12E-05

Loss for 1st stressed tendon	4.38	ksi
Loss for 2nd stressed tendon	2.92	ksi
Loss for 3rd stressed tendon	1.46	ksi
Loss for 4th stressed tendon	0	ksi

Long-term Loss

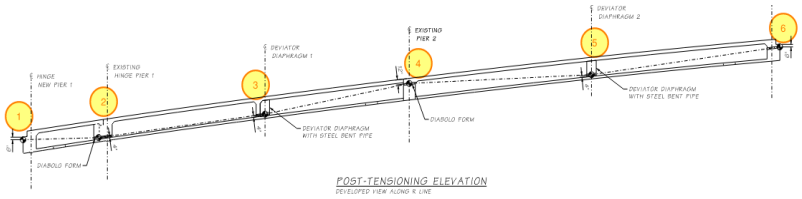
Note: the long term loss is estimated using csibridge by taking the difference between the tendon axial force of a short term case and a long term case. The long term case is "DL3b_Long_STG" and the short term case is "DL3b+PT_AbutRemove_Short_STG". The long term losses are assumed to be constant within each tendon segment. Since the difference of long term losses between the tendons within 5%, the losses are assumed to be consistent among tendons

	Long Term Axial Force	Short Term Axial Force	Loss	
Tendon Segment 1	249	269	20	kip
Tendon Segment 2	265	283	18	kip
Tendon Segment 3	264	282	18	kip
Tendon Segment 4	255	268	13	kip
Tendon Segment 5	247	258	11	kip

COWI	**Project**	I-405 Bridge 17.7	**Project No**	A207833
	Title	Tendon 2 Losses	**Date**	11/30/2021
	Prepared by	LDLI	**Checked by**	BOHU

This spreadsheet summarizes the tendon profile and calculates the stresses, losses, and elongations for Tendon 2. Note that:
1. The tendon profile used in the model is based on preliminary dwg located at:
XL5467_27_DE_BG_BR17.7-PostTensioningLayout.pdf

VERTICAL LAYOUT



Location Ref. #	Length from WP to stressing end (ft)	Offset from deck top (ft)
1	0.0	1.52
2	12.0	3.35
3	47.6	3.35
4	78.4	1.06
5	117.0	3.31
6	157.2	1.52

note: the locations of deviators were taken at the centerline of the deviators

TENDON PROPERTIES

Nominal Area	1.519	in^2
Ep	28500	ksi
fpu	270	ksi
fy	202.5	ksi
Jacking Force	218.70	ksi
Jacking/Ultimate	0.81	

note: jacking force is is the minimum beteween the 0.81fpu and 0.7fpu+anchorage loss.
Anchorage loss has to be manually adjusted in the formula to match with the value in cell F138

LOSSES

Anchorage Set

Anchorage set	0.375	in
Anchorage set	0.0313	ft

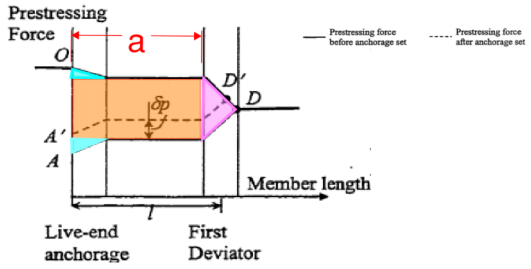
AASHTO C5.9.3.2.1

Tendon Stress before Anchorage Set

Location Ref. #	Diaphragm Thickness (in)	Wobble Coefficient (AASHTO Table 5.9.3.2.2b-1)	Angle change (Rad)	μ (AASHTO Table 5.9.3.2.2b-1)	Friction Loss (ksi)	Tendon Force after Friction AASHTO eq. 5.9.3.2.2b-2 (ksi)
1	35	0.0002			1.5	217.19
2	24	0.0002	0.22	0.23	12.0	205.24
3	36	0.0002	0.18	0.23	10.3	194.96
4	30	0.0002	0.14	0.23	8.4	186.55
5	24	0.0002	0.09	0.23	5.5	181.03
6	33	0.0002			1.4	179.62

Tendon Stress after Anchorage Set

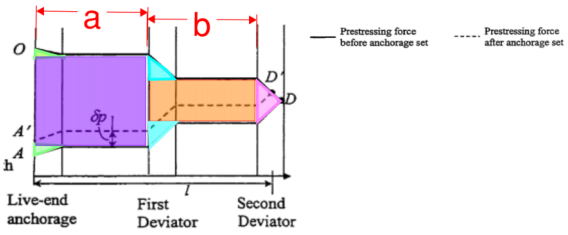
Step 1: Assuming the anchorage set effect ends at the first deviator



Clear Spacing btw Diaphragms		102 in	*note: measured along bridge cl*
a	136.625 in		
Area 1	3265.94 ksi.in		*note: in orange*
Area 2	52.26 ksi.in		*note: in blue*
Area 3	286.85 ksi.in		*note: in pink*
Area OAD	3605.06 ksi.in		
Δ_{calc}	0.13 in		

is Δcalc>Δset? no

Step 2: Assuming the anchorage set effect ends at the second deviator

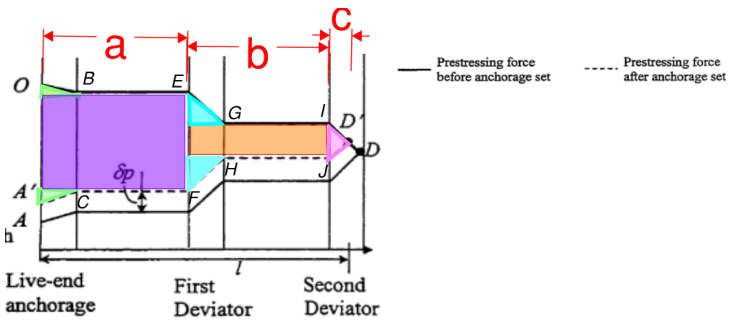


Clear Spacing btw Diaphragms 2 and 3		408.8 in	
b	420.85 in		
a	136.625 in		
Area 1	8651.18 ksi.in		*note: in orange*
Area 2	286.85 ksi.in		*note: in blue*
Area 3	370.02 ksi.in		*note: in pink*
Area 4	6074.5 ksi.in		*note: in purple*
Area 5	52.26 ksi.in		*note: in green*
Area OAD	15434.81 ksi.in		

Δ_{calc} = A_{OAD}/E_p = 0.54 in

is Δcalc>Δset? yes

Step 3: Find exact location of the l_set



c		21.6	in	*note: this needs to be manually adjusted so that Δcalc meets the target*
friction loss within c		6.23	ksi	*note: assume friction loss increase linearly within the concrete*
b		420.8458697	in	
a		136.625	in	
Area 1		5240.60	ksi.in	*note: in orange*
Area 2		286.85	ksi.in	*note: in blue*
Area 3		134.49	ksi.in	*note: in pink*
Area 4		4967.3	ksi.in	*note: in purple*
Area 5		52.26	ksi.in	*note: in green*
Area OAD		10681.47	ksi.in	
Δ_{calc} =	A_{OAD}/E_p =	0.375	in	
Target Δ_{calc}		0.375	in	

Therefore, the losses of tendon stress due to anchorage set are:

@ west face of west diaphragm=	OA'=	39.38	ksi
@ east face of end diaphragm=	BC=	36.36	ksi
@ west face of diaphragm #2=	EF=	36.36	ksi
@ east face of diaphragm #2=	GH=	12.45	ksi
@ west face of diaphragm #3=	IJ=	12.45	ksi

Elastic Shortening of Concrete

Tendon longitudinal deformation when stressing

0.094 in *Note: the deformation is from csibridge model, load case "PT1"*

Strain= 5.12E-05

Loss for 1st stressed tendon	4.38	ksi
Loss for 2nd stressed tendon	2.92	ksi
Loss for 3rd stressed tendon	1.46	ksi
Loss for 4th stressed tendon	0	ksi

Long-term Loss

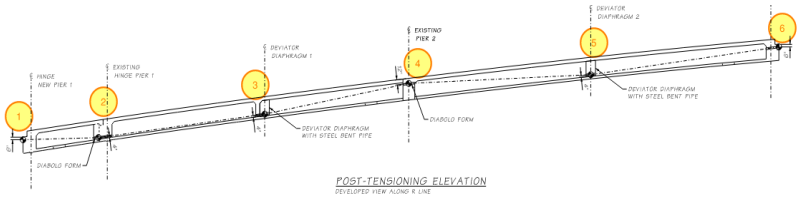
Note: the long term loss is estimated using csibridge by taking the difference between the tendon axial force of a short term case and a long term case. The long term case is "DL3b_Long_STG" and the short term case is "DL3b+PT_AbutRemove_Short_STG". The long term losses are assumed to be constant within each tendon segment. Since the difference of long term losses between the tendons within 5%, the losses are assumed to be consistent among tendons

	Long Term Axial Force	Short Term Axial Force	Loss	
Tendon Segment 1	249	269	20	kip
Tendon Segment 2	265	283	18	kip
Tendon Segment 3	264	282	18	kip
Tendon Segment 4	255	268	13	kip
Tendon Segment 5	247	258	11	kip

COWI	**Project**	I-405 Bridge 17.7	**Project No**	A207833
	Title	Tendon 3 Losses	**Date**	11/30/2021
	Prepared by	LDLI	**Checked by**	BOHU

This spreadsheet summarizes the tendon profile and calculates the stresses, losses, and elongations for Tendon 3. Note that:
1. The tendon profile used in the model is based on preliminary dwg located at:
XL5467_27_DE_BG_BR17.7-PostTensioningLayout.pdf

VERTICAL LAYOUT



Location Ref. #	Length from WP to stressing end (ft)	Offset from deck top (ft)
1	0	1.52
2	12.7	3.35
3	48.5	3.35
4	79.1	1.06
5	117.6	3.31
6	157.8	1.52

note: the locations of deviators were taken at the centerline of the d

TENDON PROPERTIES

Nominal Area	1.519	in^2
Ep	28500	ksi
fpu	270	ksi
fy	202.5	ksi
Jacking Force	218.70	ksi
Jacking/Ultimate	0.81	

note: jacking force is is the minimum beteween the 0.81fpu and 0.7fpu+anchorage loss. Anchorage loss has to be manually adjusted in the formula to match with the value in cell F138

LOSSES

Anchorage Set

Anchorage set	0.375	in
Anchorage set	0.0313	ft

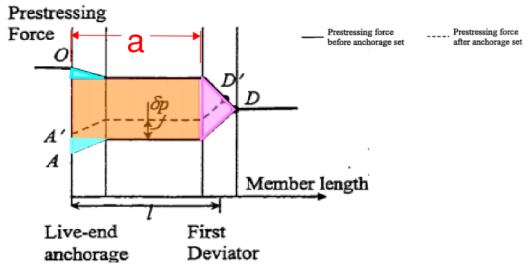
AASHTO C5.9.3.2.1

Tendon Stress before Anchorage Set

Location Ref. #	Diaphragm Thickness (in)	Wobble Coefficient (AASHTO Table 5.9.3.2.2b-1)	Angle change (Rad)	μ (AASHTO Table 5.9.3.2.2b-1)	Friction Loss (ksi)	Tendon Force after Friction AASHTO eq. 5.9.3.2.2b-2 (ksi)
1	35	0.0002			1.5	217.19
2	24	0.0002	0.21	0.23	11.5	205.71
3	36	0.0002	0.20	0.23	11.2	194.55
4	30	0.0002	0.15	0.23	8.6	185.95
5	24	0.0002	0.09	0.23	5.5	180.43
6	33	0.0002			1.4	179.01

Tendon Stress after Anchorage Set

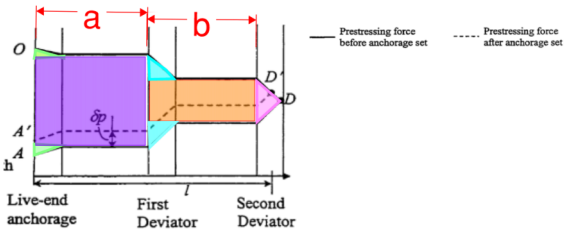
Step 1: Assuming the anchorage set effect ends at the first deviator



Clear Spacing btw Diaphragms		105.25 in	*note: measured along bridge cl*
a	139.9 in		
Area 1	3210.68 ksi.in		*note: in orange*
Area 2	52.26 ksi.in		*note: in blue*
Area 3	275.45 ksi.in		*note: in pink*
Area OAD	3538.38 ksi.in		
Δ_{calc}	0.12 in		

is Δcalc>Δset? no

Step 2: Assuming the anchorage set effect ends at the second deviator

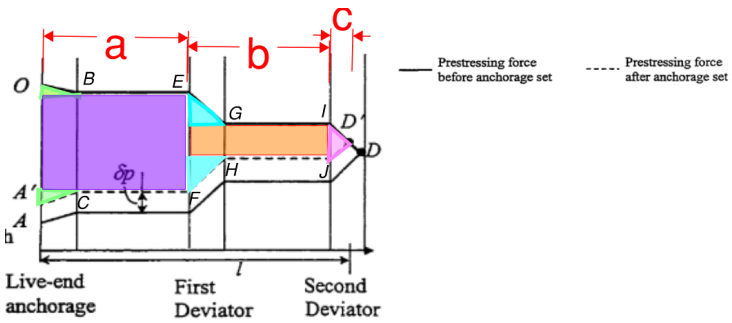


Clear Spacing btw Diaphragms 2 and 3		400.2 in	
b	412.2 in		
a	139.9 in		
Area 1	9201.02 ksi.in		*note: in orange*
Area 2	275.45 ksi.in		*note: in blue*
Area 3	401.80 ksi.in		*note: in pink*
Area 4	6333.0 ksi.in		*note: in purple*
Area 5	52.26 ksi.in		*note: in green*
Area OAD	16263.53 ksi.in		

Δ_{calc} = A_{OAD}/E_p = 0.57 in

is Δcalc>Δset? yes

Step 3: Find exact location of the l_set



c		20.3	in	*note: this needs to be manually adjusted so that Δcalc meets the targ*
friction loss within c		6.37	ksi	*note: assume friction loss increase linearly within the concrete*
b		412.2	in	
a		139.9	in	
Area 1		5247.54	ksi.in	*note: in orange*
Area 2		275.45	ksi.in	*note: in blue*
Area 3		129.22	ksi.in	*note: in pink*
Area 4		4991.4	ksi.in	*note: in purple*
Area 5		52.26	ksi.in	*note: in green*
Area OAD		10695.88	ksi.in	
Δ_{calc} =	A_{OAD}/E_p =	0.375	in	
Target Δ_{calc}		0.375	in	

Therefore, the losses of tendon stress due to anchorage set are:

@ west face of west diaphragm=	OA'=	38.70 ksi
@ east face of end diaphragm=	BC=	35.68 ksi
@ west face of diaphragm #2=	EF=	35.68 ksi
@ east face of diaphragm #2=	GH=	12.73 ksi
@ west face of diaphragm #3=	IJ=	12.73 ksi

Elastic Shortening of Concrete

Tendon longitudinal deformation when stressing

0.094 in *Note: the deformation is from csibridge model, load case "PT1"*

Strain= 5.12E-05

Loss for 1st stressed tendon	4.38 ksi
Loss for 2nd stressed tendon	2.92 ksi
Loss for 3rd stressed tendon	1.46 ksi
Loss for 4th stressed tendon	0 ksi

Long-term Loss

Note: the long term loss is estimated using csibridge by taking the difference between the tendon axial force of a short term case and a long term case. The long term case is "DL3b_Long_STG" and the short term case is "DL3b+PT_AbutRemove_Short_STG". The long term losses are assumed to be constant within each tendon segment. Since the difference of long term losses between the tendons within 5%, the losses are assumed to be consistent among tendons

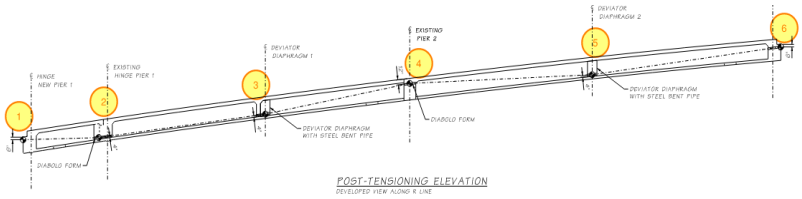
	Long Term Axial Force	Short Term Axial Force	Loss	
Tendon Segment 1	249	269	20	kip
Tendon Segment 2	265	283	18	kip
Tendon Segment 3	264	282	18	kip
Tendon Segment 4	255	268	13	kip
Tendon Segment 5	247	258	11	kip

COWI

Project	I-405 Bridge 17.7	**Project No**	
Title	Tendon 4 Losses	**Date**	11/30/2021
Prepared by	LDLI	**Checked by**	BOHU

This spreadsheet summarizes the tendon profile and calculates the stresses, losses, and elongations for Tendon 4. Note that:
1. The tendon profile used in the model is based on preliminary dwg located at:
XL5467_27_DE_BG_BR17.7-PostTensioningLayout.pdf

VERTICAL LAYOUT



Location Ref. #	Length from WP to stressing end (ft)	Offset from deck top (ft)
1	0.0	1.52
2	12.9	3.35
3	48.8	3.35
4	79.3	1.06
5	117.8	3.31
6	158.0	1.52

note: the locations of deviators were taken at the centerline of the de

TENDON PROPERTIES

Nominal Area	1.519	in^2
Ep	28500	ksi
fpu	270	ksi
fy	202.5	ksi
Jacking Force	218.70	ksi
Jacking/Ultimate	0.81	

note: jacking force is is the minimum beteween the 0.81fpu and 0.7fpu+anchorage loss. Anchorage loss has to be manually adjusted in the formula to match with the value in cell F138

LOSSES

Anchorage Set

Anchorage set	0.375	in
Anchorage set	0.0313	ft

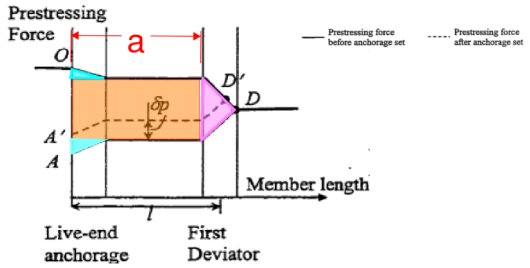
AASHTO C5.9.3.2.1

Tendon Stress before Anchorage Set

Location Ref. #	Diaphragm Thickness (in)	Wobble Coefficient (AASHTO Table 5.9.3.2.2b-1)	Angle change (Rad)	μ (AASHTO Table 5.9.3.2.2b-1)	Friction Loss (ksi)	Tendon Force after Friction AASHTO eq. 5.9.3.2.2b-2 (ksi)
1	35	0.0002			1.5	217.19
2	24	0.0002	0.23	0.23	12.2	205.00
3	36	0.0002	0.20	0.23	11.5	193.54
4	30	0.0002	0.15	0.23	8.6	184.89
5	24	0.0002	0.09	0.23	5.5	179.37
6	33	0.0002			1.4	177.95

Tendon Stress after Anchorage Set

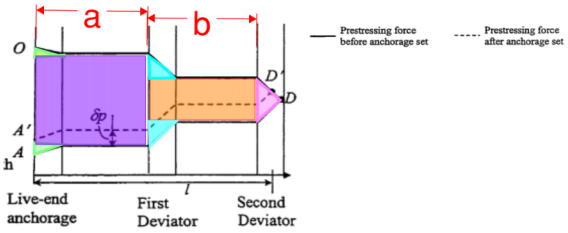
Step 1: Assuming the anchorage set effect ends at the first deviator



Clear Spacing btw Diaphragms			109 in	*note: measured along bridge cl*
a	143.34	in		
Area 1	3494.83	ksi.in		*note: in orange*
Area 2	52.26	ksi.in		*note: in blue*
Area 3	292.58	ksi.in		*note: in pink*
Area OAD	3839.67	ksi.in		
Δ_{calc}	0.13	in		

is Δcalc>Δset? no

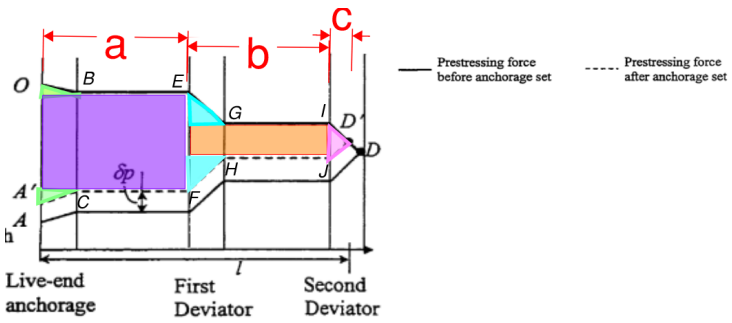
Step 2: Assuming the anchorage set effect ends at the second deviator



Clear Spacing btw Diaphragms 2 and 3			400.5 in	
b	412.5	in		
a	143.3	in		
Area 1	9456.60	ksi.in		*note: in orange*
Area 2	292.58	ksi.in		*note: in blue*
Area 3	412.63	ksi.in		*note: in pink*
Area 4	6780.7	ksi.in		*note: in purple*
Area 5	52.26	ksi.in		*note: in green*
Area OAD	16994.81	ksi.in		
Δ_{calc} =	A_{OAD}/E_p =	0.60	in	

is Δcalc>Δset? yes

Step 3: Find exact location of the l_set



c		18.8	in	*note: this needs to be manually adjusted so that Δcalc meets the targ*
friction loss within c		6.06	ksi	*note: assume friction loss increase linearly within the concrete*
b		413	in	
a		143	in	
Area 1		5001.93	ksi.in	*note: in orange*
Area 2		292.58	ksi.in	*note: in blue*
Area 3		113.98	ksi.in	*note: in pink*
Area 4		5232.9	ksi.in	*note: in purple*
Area 5		52.26	ksi.in	*note: in green*
Area OAD		10693.60	ksi.in	
Δ_{calc} =	A_{OAD}/E_p =	0.375	in	
Target Δ_{calc}		0.375	in	

Therefore, the losses of tendon stress due to anchorage set are:

@ west face of west diaphragm=	OA'=	39.53	ksi
@ east face of end diaphragm=	BC=	36.51	ksi
@ west face of diaphragm #2=	EF=	36.51	ksi
@ east face of diaphragm #2=	GH=	12.13	ksi
@ west face of diaphragm #3=	IJ=	12.13	ksi

Elastic Shortening of Concrete

Tendon longitudinal deformation when stressing

0.094	in	*Note: the deformation is from csibridge model, load case "PT1"*
Strain=	5.12E-05	

Loss for 1st stressed tendon	4.38	ksi
Loss for 2nd stressed tendon	2.92	ksi
Loss for 3rd stressed tendon	1.46	ksi
Loss for 4th stressed tendon	0	ksi

COWI

Project	I-405 Bridge 17.7	**Project No**	
Title	Tendon 4 Losses	**Date**	11/30/2021
Prepared by	LDLI	**Checked by**	BOHU

Long-term Loss

Note: the long term loss is estimated using csibridge by taking the difference between the tendon axial force of a short term case and a long term case. The long term case is "DL3b_Long_STG" and the short term case is "DL3b+PT_AbutRemove_Short_STG". The long term losses are assumed to be constant within each tendon segment. Since the difference of long term losses between the tendons within 5%, the losses are assumed to be consistent among tendons

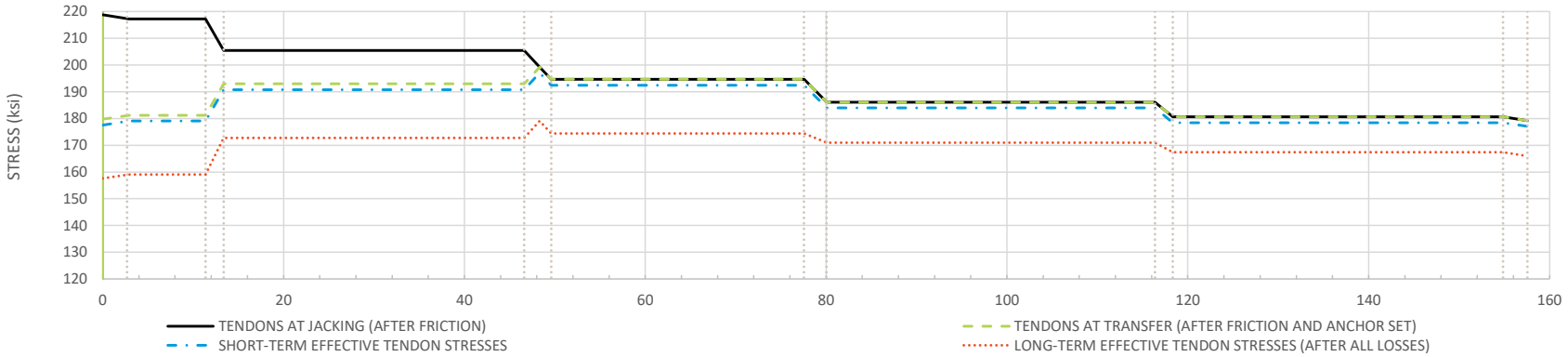
	Long Term Axial Force	Short Term Axial Force	Loss	
Tendon Segment 1	249	269	20	kip
Tendon Segment 2	265	283	18	kip
Tendon Segment 3	264	282	18	kip
Tendon Segment 4	255	268	13	kip
Tendon Segment 5	247	258	11	kip

COWI

Project I-405 Bridge 17.7
Title Tendon Stresses Summary
Prepared by LDLI

Project No
Date 11/30/2021
Checked by BOHU

AVERAGE STRESS IN POST-TENSIONING TENDONS



		PT LOSSES (ksi)							PT EFFECTIVE (ksi)	
DEVIATOR LOCATION	Length from WP to stressing end (ft)	FR	ANC	ES	CR+SH+REL	TOTAL	PT JACKING (ksi)	PT TRANSFER (ksi)	SHORT TERM	LONG TERM
1	0.0	0	39	2	20	61	219	180	178	158
1	2.9	2	36	2	20	60	217	181	179	159
2	11.4	2	36	2	20	60	217	181	179	159
2	13.4	13	13	2	18	46	205	193	191	173
3	46.6	13	13	2	18	46	205	193	191	173
3	48.3	19	0	2	18	40	199	199	197	179
3	49.6	24	0	2	18	44	195	195	192	174
4	77.6	24	0	2	18	44	195	195	192	174
4	80.1	33	0	2	13	48	186	186	184	171
5	116.4	33	0	2	13	48	186	186	184	171
5	118.4	38	0	2	11	51	181	181	178	167
6	154.9	38	0	2	11	51	181	181	178	167
6	157.6	40	0	2	11	53	179	179	177	166

note: distance are measured along cl of bridge deck

TENDON INFORMATION TABLE

		PT LOSSES (ksi)							PT EFFECTIVE (ksi)	
TENDON DESIGNATION	DISTANCE FROM BACK OF PIER 1 (ft)	FR	ANC	ES	CR+SH+REL	TOTAL	PT JACKING (ksi)	PT TRANSFER (ksi)	SHORT TERM	LONG TERM
TENDON 1	0	0	38.2	2	20	61	219	180	178	158
TENDON 2	0	0	39.4	2	20	61	219	179	177	157
TENDON 3	0	0	38.7	2	20	61	219	180	178	158
TENDON 4	0	0	39.5	2	20	61	219	179	177	157

TENDON INFORMATION TABLE

									ELONGATION	
TENDON DESIGNATION	LENGTH (ft)	NUMBER OF STRANDS	TENDON AREA (in^2)	TENDON WEIGHT (lbs)	STRESSING FORCE/TENDON (kip)	STRESSING END	LIVE END FORCE AFTER ANCHOR SET (kip)	DEAD END FORCE AFTER ANCHOR SET (kip)	BEFORE ANCHOR SET (in)	AFTER ANCHOR SET (in)
TENDON 1	157.2	7	1.52	814	332	Pier 1	274	180	10.2	8.5
TENDON 2	157.2	7	1.52	814	332	Pier 1	272	180	10.2	8.5
TENDON 3	157.8	7	1.52	818	332	Pier 1	273	179	10.2	8.5
TENDON 4	158.0	7	1.52	819	332	Pier 1	272	178	10.2	8.5

CHECK JACKING FORCE REQUIREDMENT WITH BDM FIGURE 5.8.3-1

Jacking Stress	218.7	ksi	81% f_{pu}	Governs
Jacking Stress after Anchor Seated	179.7	ksi	67% f_{pu}	

COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I3

CALC BOOK COMPLETION DATE: **2021 Nov 30**

Title: Diaphragms

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I3-0		I3-0	Calc Register (this sheet)	N/A	N/A	
2	I3-1	-	I3-2	1 New Pier 1 Diaphragm	LDLI	BOHU	
3	I3-3	-	I3-10	2 Existing Pier 1 Deviator Diaphragm	LDLI	BOHU	
4	I3-11		I3-13	3 Pier 2 Diaphragm	LDLI	BOHU	
5	I3-14		I3-15	4 Pier 3 Diaphragm	LDLI	BOHU	
6	I3-16		I3-27	5 Deviator Diaphragms 1 and 2	LDLI	BOHU	
7	I3-28		I3-59	6 Deviation Force Check at Deviator Diaphragms 1 and 2 PT-1	LDLI	BOHU	

Pier 1 Diaphragm Check

1. Description

This calculation checks and designs the new pier 1 diaphragm based on AASHTO LRFD 2017. The calculations consist of three parts: 1) design for edge tension resistance ; 2) design for bursting force resistance; 3) check for concrete bearing strength.

2. Tendon Properties

Strand area	$A_{ps} := 0.217\ in^2$
Number of strand per tendon	$N_{ps} := 7$
Tendon area	$A_p := N_{ps} \cdot A_{ps} = 1.52\ in^2$
Tendon ultimate strength	$f_{pu} := 270 \cdot ksi$
Total design PT anchorage force	$F_{ps} := 1.2 \cdot 0.81 \cdot A_p \cdot f_{pu} = 398.65\ kip$

3. Mild Reinforcement Properties

Yield strength $f_y := 60 \cdot ksi$

Bar areas $A_{bar.4} := 0.2 \cdot in^2$ $A_{bar.5} := 0.31 \cdot in^2$ $A_{bar.6} := 0.44 \cdot in^2$ $A_{bar.7} := 0.60 \cdot in^2$ $A_{bar.8} := 0.79 \cdot in^2$

4. Edge Tension

Edge Tension per tendon $T_{edge} := 0.04 \cdot F_{ps} = 15.95\ kip$

Note: assuming 4% of the total PT force to be conservative

Resistance factor $\varphi := 0.9$

Required Steel $A_{s.edge} := \frac{T_{edge}}{\varphi \cdot f_y} = 0.3\ in^2$

AASHTO LRFD 5.5.4.2

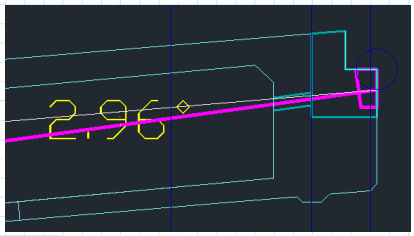
Provide 2-#6 per anchorage, D/C is: $DC_{edge} := \frac{A_{s.edge}}{2 \cdot A_{bar.6}} = 0.34$

5. Bursting Force (LRFD 5.8.4.5.3)

Section height $h := 4 \cdot ft + 3 \cdot in = 51\ in$

PT angle $\alpha := 2.96 \cdot deg$

Note: The angle is measured from the autocad file "09.01.2021_XL5467_27_DE_BG_BR17.7-PTLayout". The angle is assumed to be the same between tendons



Top slab thickness $t_t := 8.5\ in$

Bottom slab thickness $t_b := 6\ in$

Vertical offset of tendon end from top of the box $y := 13\ in + 6.5\ in = 19.5\ in$

Assuming the centroid of the diaphragm cross-section is at the mid-height of the box:

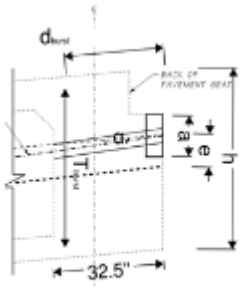
Eccentricity of the anchorage device with respect to the centroid of the cross-section $e := \left| y - \frac{h}{2} \right| = 6\ in$

PT anchor plate dimension (use Dim B in DSI catalog) $a := 170 \cdot mm = 6.69\ in$

Bursting force (AASHTO 5.8.4.5.3) $T_{burst} := 0.25 \cdot F_{ps} \cdot \left(1 - \frac{a}{h}\right) + 0.5 \cdot F_{ps} \cdot \sin(\alpha) = 96.88\ kip$

Required area $A_{s.burst} := \frac{T_{burst}}{\varphi \cdot f_y} = 1.79\ in^2$

Bursting force location $d_{burst} := 0.5 \cdot (h - 2 \cdot e) + 5 \cdot e \cdot \sin(\alpha) = 21.05\ in$



As per AASHTO LRFD 5.9.5.6b, The bursting reinforcement should be distributed between anchor plate and 2.5d.bursting.
Conservatively assuming the PT anchor force will be take by a vertical deep beam

Compression force going to top flange $P_{up} := \frac{F_{ps} \cdot \left(h - y - \frac{t_b}{2}\right)}{h - \frac{t_b}{2} - \frac{t_t}{2}} = 259.69\ kip$

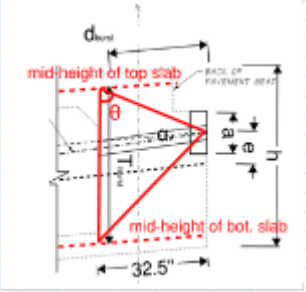
Thickness of diaphragm $t_{dia} := 32.5\ in$

Angle of the strut $\theta := \operatorname{atan}\left(\frac{t_{dia}}{y}\right) = 59.04\ deg$

So the vertical force $T := \frac{P_{up}}{\tan(\theta)} = 155.81\ kip$

Required rebar area $A_{s.f} := \frac{T}{\varphi \cdot f_y} = 2.89\ in^2$

Provide 4-#8 per anchorage, D/C is: $DC_{edge} := \frac{A_{s.f}}{4 \cdot A_{bar.8}} = 0.91$



6. Concrete bearing Strength

Approximately dead load at each web $V_{DL.web} := \frac{434 \cdot kip}{4} \cdot 1.2 = 130.2\ kip$

Note: this is from CSiBridge model load case "*DL3_noPT_AbutRemove". The total shear force is divided by the number of the webs and increased by 20% to account for the uneven distribution of loads between webs

Given 3" on each side and plus the web, the bearing area width

$$B_{bearing} := 2 \cdot 3 \cdot in + 9 \cdot in = 15\ in$$

Allowable bearing strength of concrete $f_{c.b} := 0.7 \cdot 5 \cdot ksi = 3.5\ ksi$

Required length of the bearing area $L_{bearing} := \frac{1.25 \cdot V_{DL.web}}{f_{c.b} \cdot B_{bearing}} = 3.1\ in$

This is less than the thickness of the diaphragm (32.5 in)

By inspection, the check passes.

7. Summary

Required area for edge tension $A_{s.edge} = 0.3\ in^2$

Provide 2-#5 per anchorage, D/C is: $DC_{edge} := \frac{A_{s.edge}}{2 \cdot A_{bar.5}} = 0.48$

Required area for bursting $A_{s.req.burst} := \max\left(A_{s.burst}, A_{s.f}\right) = 2.89\ in^2$

Provide 4-#8 per anchorage, D/C is: $DC_{edge} := \frac{A_{s.req.burst}}{4 \cdot A_{bar.8}} = 0.91$

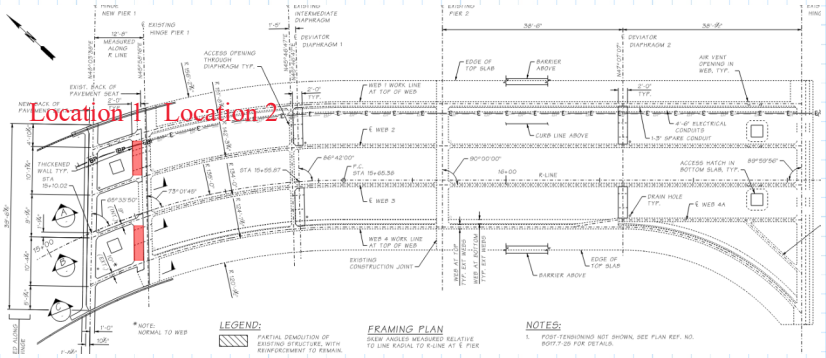
Required length of the bearing area $L_{bearing} = 3.1\ in$

Provide the entire thickness of the diaphragm for bearing length, D/C is: $DC_{edge} := \frac{L_{bearing}}{t_{dia}} = 0.1$

Existing Pier 1 Deviator Design

1. Description

The following items need to be check at the existing Pier 1 deviator design
1. Vertical deviating force to be transferred to bottom flange and then to be transferred to webs or through a frame action to the entire section. For the tendon closed to the utility openings (PT-1), we did a frame analysis to make sure the vertical deviation force will be able to be transferred by a frame action because webs are far away. This sheet will focus on the checks for other three tendons. For a reference, the check for PT-1 is at \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\Design\Diaphragm Design\Utility Blockout Check and Analysis.
2. Longitudinal PT unbalanced longitudinal force needs to be transferred to the bottom flange.
3. Transverse PT force component needs to be transferred to the bottom flange.



2. Material Properties

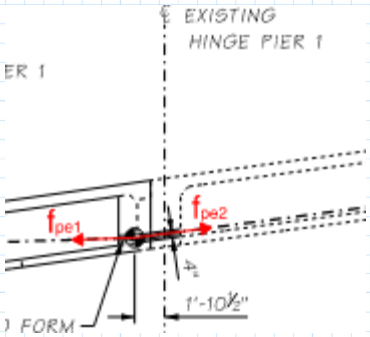
Concrete Compressive Strength	$f'_c := 5 \cdot ksi$
Typical Bar Yield Strength	$f_y := 60 \cdot ksi$

3. Structural Properties

Thickness of bottom slab	$t_{slab} := 6 \cdot in$
Width of deviator (cell)	$b := 7 \cdot ft$
Thicknesses deviators	$t := 24 \cdot in$

4. Tendon Stresses

Tendon ultimate stress	$f_{pu} := 270\ ksi$
Tendon average effective stress at Location 1	$f_{pe1} := 179\ ksi$
Tendon average effective stress at Location 2	$f_{pe2} := 191\ ksi$
Tendon segment 1 ULS loading stress (AASHTO 5.6.3.1)	$f_{ps1} := 15\ ksi + f_{pe1} = 194\ ksi$
Tendon segment 2 ULS loading stress (AASHTO 5.6.3.1)	$f_{ps2} := 15\ ksi + f_{pe2} = 206\ ksi$
Strand area	$A_{strand} := 0.217\ in^2 = 0.217\ in^2$
Tendon area	$A_{tendon} := 0.217\ in^2 \cdot 7 = 1.519\ in^2$



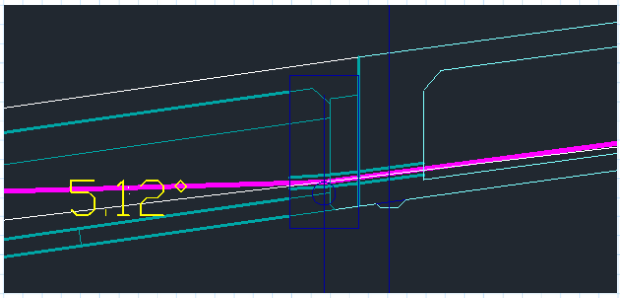
Note: The calculations for the average effective stresses are at: \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\Design\Tendon Design\Tendon Losses, Jacking Force, and Elongations\20211117 Tendon Losses and Stresses

4. Vertical upward force check

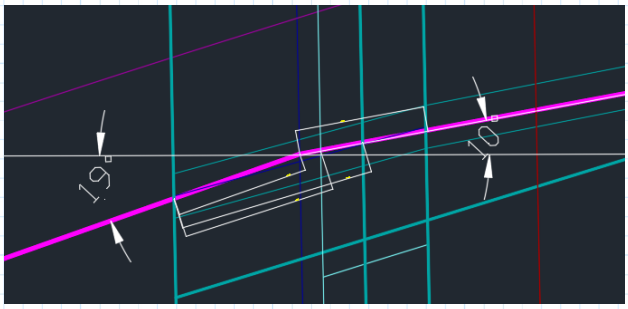
Part 1: The first step is to check the through bolts. One thing to check here is the geometry check to determine how far the bolts need to be from the OD duct.
Part 2: The second step has two cases.
Case 1: the left tendon. We did the frame analysis a while ago to make sure the vertical deviation force will be able to be transferred by a frame action because webs are far away.
Case 2: Other tendons.

Part 1:

Demands



Post-tensioning Elevation View



Post-tensioning Plan View

Tendon angle change in elevation	$\theta_{1.ele} := 5.12$ *deg*	$\theta_{2.ele} := 0$ *deg*
Tendon angle change in plan	$\theta_{1.plan} := -19$ *deg*	$\theta_{2.plan} := 10$ *deg*
Demand factor	$\alpha_p := 1.2$	

Factored tensile demand per tendon

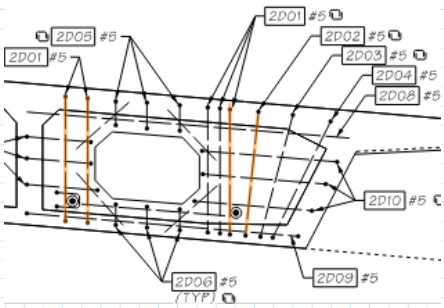
$$T_{dev} := \alpha_p \cdot A_{tendon} \cdot \left(f_{ps1} \cdot \sin\left(\theta_{1.ele}\right) + f_{ps2} \cdot \sin\left(\theta_{2.ele}\right)\right) = 31.56 \ \textit{kip}$$

Steel area required per tendon

Resistance Factor (AASHTO 5.13.2.2) $\phi_{tension} := 0.9$

Area required

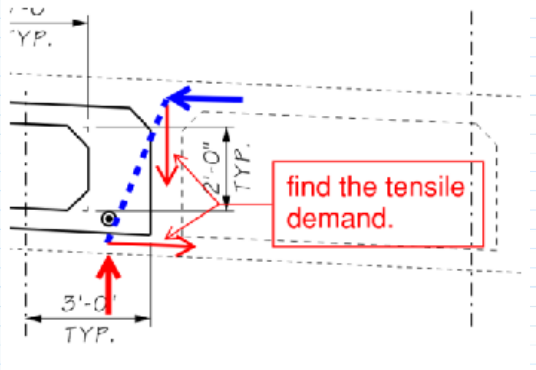
$$A_{s.vert.1} := \frac{T_{dev}}{f_y \cdot \phi_{tension}} = 0.58 \ \textit{in}^2$$

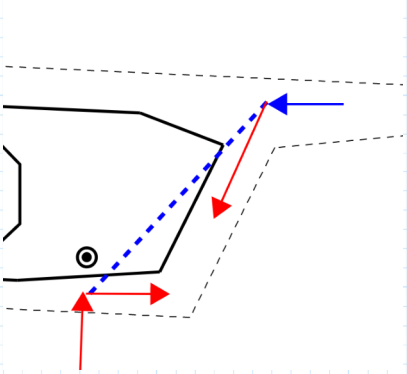


Part 2 (Case 1):

For the demand and capacity for PT-1, the check is at \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\Design\Diaphragm Design\Utility Blockout Check and Analysis.

Part 2 (Case 2):
Tensile demand per tendon within web and bottom flange





Assume the angle changes and tendon stresses are close between tendons. Then the tensile demand within the web and bottom flange will increase when the eccentricity between the tendon and web increases. Therefore, PT-2 and PT-4 are checked.

Interior Cell

Factored tensile demand in web

$$T_{web.dev.int} := T_{dev} = 31.56\ kip$$

Horizontal eccentricity between tendon and cl web

$$e_h := 1\ ft + 9.5\ in + \frac{9}{2}\ in = 2.17\ ft$$

Note: PT-2 is 1'-9.5" away from the face of interior web face. The interior web is 9 in thick

Vertical eccentricity between cl slabs

$$e_v := 4.25\ ft - \frac{(6\ in + 8.5\ in)}{2} = 3.65\ ft$$

Factored tensile demand in bottom slab

$$T_{botslab.dev.int} := T_{dev} \cdot \frac{e_h}{e_v} = 18.75\ kip$$

Exterior Cell

Factored tensile demand in web

$$T_{web.dev.ext} := \frac{T_{dev}}{2} \cdot \sqrt{5} = 35.28\ kip$$

Note: exterior web slope is 2:1

Horizontal eccentricity

$$e_h := 17.5\ in + \frac{10}{2}\ in + \frac{(4.25\ ft - (6\ in + 8.5\ in))}{2} = 3.4\ ft$$

Vertical eccentricity between slabs

$$e_v := 4.25\ ft - \frac{(6\ in + 8.5\ in)}{2} = 3.65\ ft$$

Factored tensile demand in bottom slab

$$T_{botslab.dev.ext} := T_{dev} \cdot \frac{e_h}{e_v} = 29.39\ kip$$

Governing demand in web between interior and exterior cells

$$T_{web.dev} := \max\left(T_{web.dev.int}, T_{web.dev.ext}\right) = 35.28\ kip$$

Governing demand in bottom slab between interior and exterior cells

$$T_{botslab.dev} := 0.25 \cdot \max\left(T_{botslab.dev.int}, T_{botslab.dev.ext}\right) = 7.35\ kip$$

Steel area required per tendon

Resistance Factor (AASHTO 5.13.2.2)

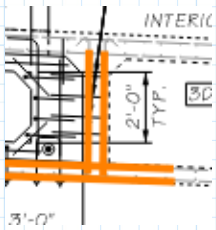
$$\phi_{tension} := 0.9$$

Area required for web stirrup

$$A_{s.vert.2} := \frac{T_{web.dev}}{f_y \cdot \phi_{tension}} = 0.65\ in^2$$

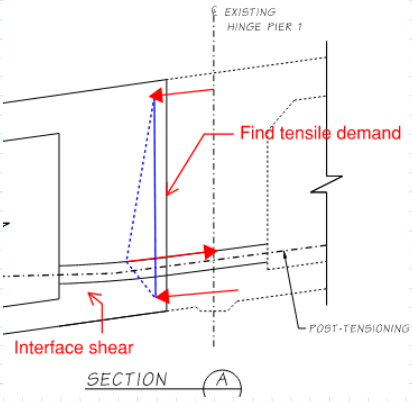
Area required for bottom slab transverse reinforcement

$$A_{s.vert.3} := \frac{T_{botslab.dev}}{f_y \cdot \phi_{tension}} = 0.1\ in^2$$



2. longitudinal check

The longitudinal check consists of two parts:
Part 1: check the vertical rebars resistance; and
Part 2: check interface shear resistance between the bottom slab and deviator diaphragms



Part 1: check the vertical rebars resistance

Longitudinal shear demand per tendon

Stress difference at ultimate flexural limit states:

$$V_{dev.long.1} := \alpha_p \cdot \left(\left(0.9 \cdot f_{pu}\right) \cdot \cos\left(\theta_{1.plan}\right) \cdot \cos\left(\theta_{1.ele}\right) - \left(f_{ps2}\right) \cdot \cos\left(\theta_{2.plan}\right) \cdot \cos\left(\theta_{2.ele}\right) \right) \cdot A_{tendon} = 47.35 \ \textit{kip}$$

Maximum friction force between the tendons and duct:

Friction coefficient (AASHTO 5.9.3.2.2 takes 0.23, but taken as 0.4 to be conservative)

$\mu := 0.4$ $\qquad V_{dev.long.2} := T_{dev} \cdot \mu = 12.62 \ \textit{kip}$

Governing longitudinal factored demands

$$V_{dev.long} := min\left(V_{dev.long.1}, V_{dev.long.2}\right) = 12.62 \ \textit{kip}$$

Vertical offset from top of duct to centroid of tendon (AASHTO Figure C5.9.1.6-1) $\qquad z := 0.5 \ \textit{in}$

Vertical offset from top of duct to centroid of deck $\qquad y_1 := 4.25 \ \textit{ft} - 4 \ \textit{in} - 6 \ \textit{in} - \frac{8.5}{2} \ \textit{in} - z = 3.02 \ \textit{ft}$

Vertical offset from top of duct to centroid of bot slab $\qquad y_2 := z + 4 \ \textit{in} + \frac{6}{2} \ \textit{in} = 0.63 \ \textit{ft}$

Angle β $\qquad \beta := \operatorname{atan}\left(\frac{y_1}{\frac{t}{2}}\right) = 71.7 \ \textit{deg}$

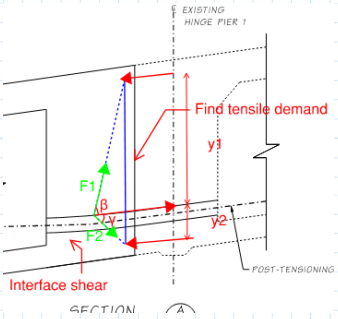
Angle γ $\qquad \gamma := \operatorname{atan}\left(\frac{y_2}{\frac{t}{2}}\right) = 32 \ \textit{deg}$

From free body diagram:

$$F1\cos(\beta) + F2\cos(\gamma) = V_{dev1.long}$$
$$F1\sin(\beta) - F2\sin(\gamma) = 0$$

$$multiplier := \begin{bmatrix} \cos(\beta) & \cos(\gamma) \\ \sin(\beta) & -\sin(\gamma) \end{bmatrix} \qquad v := \begin{bmatrix} V_{dev.long} \\ 0 \end{bmatrix}$$

$$F := \operatorname{lsolve}\left(multiplier, v\right) = \begin{bmatrix} 6.89 \\ 12.33 \end{bmatrix} \ \textit{kip}$$

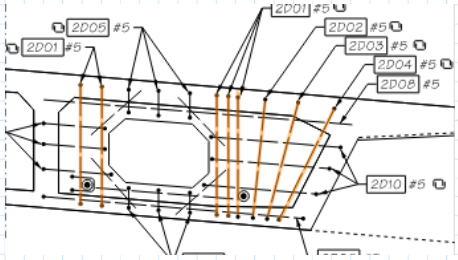


$F_1 := F_0 = 6.89 \ \textit{kip}$ $\qquad F_2 := F_1 = 12.33 \ \textit{kip}$

Factored vertical tensile demand within diaphragm:

$$T_{dia.dev} := \sin(\beta) \cdot F_1 = 6.5 \ \textit{kip}$$

Area of Steel Required



Total tensile resistance for each tendon:

Area required for vertical reinforcement in the diaphragm for each tendon

$$A_{s.long.1} := \frac{T_{dia.dev}}{f_y \cdot \phi_{tension}} = 0.12\ in^2$$

Part 2: Check interface shear resistance between diaphragm and bottom slab (per cell)

Cohesion factor (AASHTO 5.7.4.3)	$c := 0.4 \cdot ksi$	Note: assume normal weight concrete placed monolithically
Friction factor(AASHTO 5.7.4.3)	$\mu := 1.4$	
Fraction of concrete strength available to resist interface shear (AASHTO 5.7.4.3)	$K_1 := 0.25$	
Limiting interface shear resistance (AASHTO 5.7.4.3)	$K_2 := 1.5 \cdot ksi$	
Area of concrete considered to be engaged in interface shear transfer per cell (AASHTO 5.7.4.2)	$A_{cv.dev} := t \cdot \frac{b}{2} = (1.01 \cdot 10^3)\ in^2$	Note: use 50% to be conservative
Shear force resistance factor (AASHTO 5.5.4.2)	$\phi_{interfaceshear} := 0.9$	

Nominal shear resistance from concrete (AASHTO 5.7.4.3)

$$V_{ni.dev} := c \cdot A_{cv.dev} = 403.2\ kip$$

The nominal shear resistance used in the design shall not exceed either of the following:

$$V_{ni.1} := K_1 \cdot f'_c \cdot A_{cv.dev} = 1260\ kip \qquad V_{ni.2} := K_2 \cdot A_{cv.dev} = 1512\ kip$$

Therefore, the factored resistance is:

$$V_{ri.dev1} := V_{ni.dev} \cdot \phi_{interfaceshear} = 362.88\ kip$$

The interface demand is :

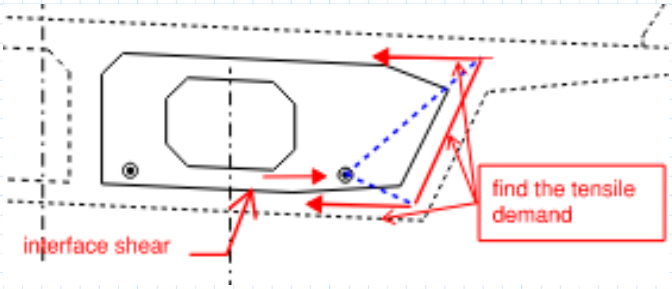
$$V_{dev.long} = 12.62\ kip$$

The concrete resistance is greater than the demand, so only minimum area of interface shear reinforcement is needed as per AASHTO 5.7.4.2:

$$A_{s.long.2} := \frac{0.05 \cdot A_{cv.dev}}{\frac{f_y}{ksi}} = 0.84\ in^2$$

3. Transverse force check

Three things need to be checked.
Part 1- bursting
Part 2- how the force will be transferred to bottom slab
Part 3- Interface shear



Transverse shear demand per tendon

Stress difference at ultimate flexural limit states:

$$V_{dev.trans} := \left| \left(\left\langle f_{ps1} \right\rangle \cdot \sin\left(\theta_{1.plan} \right) \cdot \cos\left(\theta_{1.ele} \right) + \left\langle f_{ps2} \right\rangle \cdot \sin\left(\theta_{2.plan} \right) \cdot \cos\left(\theta_{2.ele} \right) \right) \cdot A_{tendon} \right| = 41.22 \ kip$$

Governing transverse factored demands

$$V_{dev.trans} = 41.22 \ kip$$

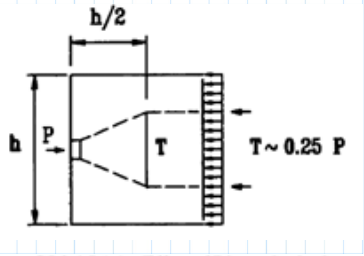


Figure C5.8.4.5.1-1—Effect of Discontinuity in Anchorage Zone

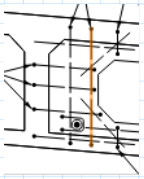
Part 1 - Bursting

Downward force per anchorage $P := V_{dev.trans}$

Horizontal bursting force (AASHTO C5.8.4.5.1-1) $F_{P2.burst} := 0.25 \cdot P = 10.31 \ kip$

Required steel area

$$A_{s.trans.1} := \frac{F_{P2.burst}}{\phi_{tension} \cdot f_y} = 0.19 \ in^2$$

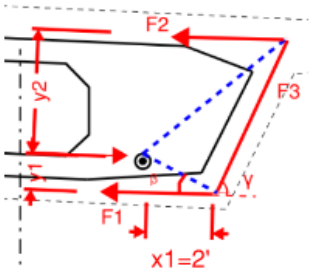


Part 2 - Tension in bottom slab and web

Vertical offset from top of duct to centroid of tendon (AASHTO Figure C5.9.1.6-1) $z := 0.5 \ in$

Vertical offset from top of duct to centroid of bot slab $y_1 := z + 4 \ in + \frac{6}{2} \ in = 7.5 \ in$

Vertical offset from top of duct to centroid of top slab $y_2 := 4.25 \ ft - \frac{8.5 \ in}{2} - y_1 = 3.271 \ ft$



From free body diagram:

$$F_2 := \frac{V_{dev.trans}}{\left(1 + \frac{y_2}{y_1}\right)} = 6.61 \ kip \qquad F_1 := V_{dev.trans} - F_2 = 34.61 \ kip$$

From free body diagram:

$$\frac{F_1 - F_3 \cdot cos(\gamma)}{F_3 \cdot sin(\gamma)} = \frac{1}{tan(\beta)}$$

Horizontal offset from duct to the intersection of centerlines of web and bottom slab (measured using bluebeam) $x_1 := 2 \ ft$

Angle β $\beta := \operatorname{atan}\left(\frac{y_1}{x_1}\right) = 17.4 \ deg$

Angle γ $\gamma := \operatorname{atan}\left(\frac{2}{(1)}\right) = 63.43 \ deg$

$$F_3 := \frac{F_1}{\frac{1}{\tan(\beta)} \cdot \sin(\gamma) + \cos(\gamma)} = 10.46 \ kip$$

Area of steel required in bottom slab for each tendon:

$$A_{s.trans.2} := \frac{F_1}{\phi_{tension} \cdot f_y} = 0.64 \ in^2$$

Area of steel required in web for each tendon:

$$A_{s.trans.3} := \frac{F_3}{\phi_{tension} \cdot f_y} = 0.19 \ in^2$$

Area of steel required in top slab for each tendon:

$$A_{s.trans.4} := \frac{F_2}{\phi_{tension} \cdot f_y} = 0.12 \ in^2$$

Part 3 - Interface shear

Resistance is the same as the interface shear resistance for the longitudinal shear $V_{ni.dev} = 403.2\ kip$

The interface demand is :

$$V_{dev.trans} = 41.22\ kip$$

The concrete resistance is greater than the demand, so only minimum area of interface shear reinforcement is needed as per AASHTO 5.7.4.2. This is covered in the interface shear resistance calculation for the longitudinal force

4. Summary

Required Area of Steel

	vertical tensile force in web caused by transverse force	*vertical tensile force in web caused by vertical force*
For web reinforcement:	$A_{s.trans.3} = 0.19\ in^2$	$A_{s.vert.2} = 0.65\ in^2$

	transverse tensile force in bot slab caused by transverse force	*transverse tensile force in bot slab caused by vertical force*
For bottom slab transverse reinforcement:	$A_{s.trans.2} = 0.64\ in^2$	$A_{s.vert.3} = 0.1\ in^2$

	vertical tensile force near the face of the diaphragm caused by longitudinal force	*vertical tensile force in diaphragm caused by vertical force*	*bursting force near anchorage caused by transverse force*	*minimum reinforcement required to resist interface shear due to longitudinal and transverse forces*
For diaphragm vertical reinforcement:	$A_{s.long.1} = 0.12\ in^2$	$A_{s.vert.1} = 0.58\ in^2$	$A_{s.trans.1} = 0.19\ in^2$	$A_{s.long.2} = 0.84\ in^2$

	transverse tensile force in top slab caused by transverse force
For top slab transverse reinforcement:	$A_{s.trans.4} = 0.12\ in^2$

Provided Area of Steel

$A_{bar.4} := 0.2 \cdot in^2$ $A_{bar.5} := 0.31 \cdot in^2$ $A_{bar.6} := 0.44 \cdot in^2$ $A_{bar.7} := 0.60 \cdot in^2$ $A_{bar.8} := 0.79 \cdot in^2$

For web reinforcement: (6-#5 vertical legs per web)	$A_{s.web} := 6 \cdot A_{bar.5} = 1.86\ in^2$	$DC_{web} := \frac{A_{s.trans.3} + A_{s.vert.2}}{A_{s.web}} = 0.46$	
For bottom slab transverse reinforcement: (6-#4 +1-#8within deviator)	$A_{s.botslab} := 6 \cdot A_{bar.4} + A_{bar.8} = 1.99\ in^2$	$DC_{botslab} := \frac{A_{s.trans.2} + A_{s.vert.3}}{A_{s.botslab}} = 0.39$	
For U-bar around tendon: (2x3-#5 vertical legs per anchor)	$A_{s.ubar} := 6 \cdot A_{bar.5} = 1.86\ in^2$	$DC_{ubar} := \frac{A_{s.vert.1}}{A_{s.ubar}} = 0.31$	extend into bottom slab
For diaphragm vertical reinforcement to transfer long. force to slabs (2-#5)	$A_{s.diaphrgm.1} := 2 \cdot A_{bar.5} = 0.62\ in^2$	$DC_{diaphragm.1} := \frac{A_{s.long.1}}{A_{s.diaphrgm.1}} = 0.2$	extend into bottom slab
For diaphragm vertical reinforcement to resist bursting force (1-#5)	$A_{s.diaphrgm.2} := A_{bar.5} = 0.31\ in^2$	$DC_{diaphragm.2} := \frac{A_{s.trans.1}}{A_{s.diaphrgm.2}} = 0.62$	extend into bottom slab

For diaphragm vertical reinforcement to provide min reinf. area for interface shear (1-#5)

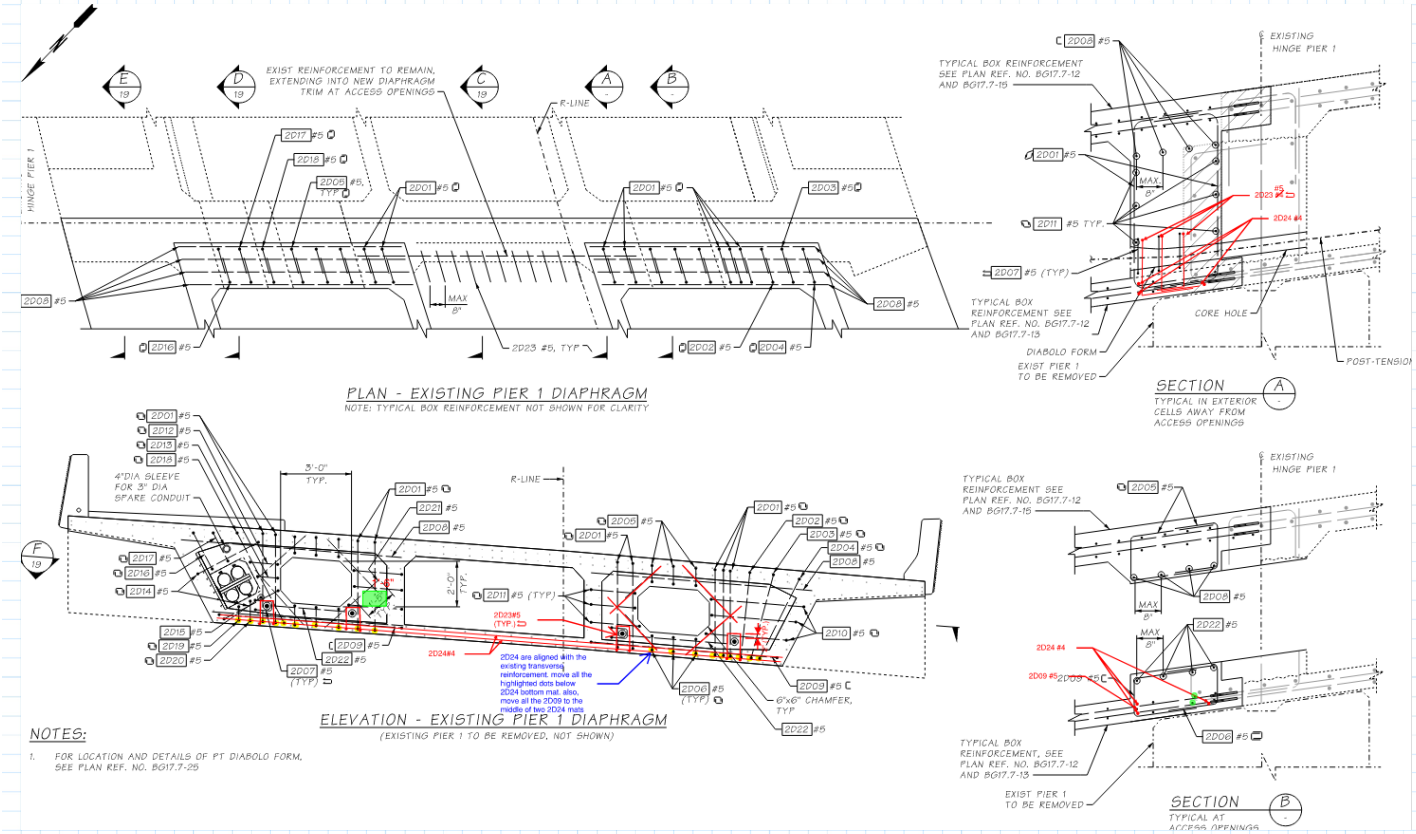
$$DC_{diaphragm.e} := \frac{A_{s.long.2}}{A_{s.ubar} + A_{s.diaphrgm.1} + A_{s.diaphrgm.2}} = 0.3$$

no need to provide resistance. only to satisfy min area requirement in aashto

For top slab transverse reinforcement:
(6-#5 within deviator)

$$A_{s.topslab} := 6 \cdot A_{bar.5} = 1.86\ in^2$$

$$DC_{topslab} := \frac{A_{s.trans.4}}{A_{s.topslab}} = 0.07$$



Pier 2 Diaphragm Check

The following items need to be check at the existing pier diaphragm for PT.

1. longitudinal PT unbalanced longitudinal force needs to be transferred to top flange or web;
2. vertical downward PT force component will cause horizontal bursting forces in both transverse and longitudinal direction; and

Checks for transverse PT force component is not necessary because the tendon is straight in the plan view, meaning no PT force component in the transverse direction.

Reinforcement Properties

Bar Size $A_{bar.4} := 0.2 \cdot in^2$ $A_{bar.5} := 0.31\ in^2$ $A_{bar.6} := 0.44\ in^2$ $A_{bar.8} := 0.79 \cdot in^2$

steel yield strength $f_y := 60\ ksi$

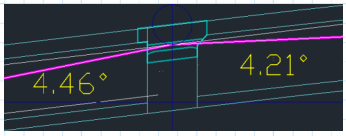
1. Longitudinal check

Strand area $A_{ps} := 0.217\ in^2$

Number of strand per tendon $N_{ps} := 7$

Tendon area $A_p := N_{ps} \cdot A_{ps} = 1.519\ in^2$

Tendon ultimate strength $f_{pu} := 270 \cdot ksi$



Tendon angle change in elevation $\theta_{1.ele} := 4.46\ deg$ $\theta_{2.ele} := 4.21\ deg$

Tendon angle change in plan $\theta_{1.plan} := 0\ deg$ $\theta_{2.plan} := 0\ deg$

Demand factor $\alpha_p := 1.2$

Tendon average effective stress at Location 1 $f_{pe1} := 201\ ksi$

Tendon average effective stress at Location 2 $f_{pe2} := 109\ ksi$

Tendon segment 1 ULS loading stress (AASHTO 5.6.3.1) $f_{ps1} := 15\ ksi + f_{pe1} = 216\ ksi$

Tendon segment 2 ULS loading stress (AASHTO 5.6.3.1) $f_{ps2} := 15\ ksi + f_{pe2} = 124\ ksi$

Tensile demand:

$$T_{vert} := \alpha_p \cdot A_p \cdot \left(f_{ps1} \cdot \sin\left(\theta_{1.ele}\right) + f_{ps2} \cdot \sin\left(\theta_{2.ele}\right)\right) = 47.21\ kip$$

Longitudinal:

Stress difference at ultimate flexural limit states:

$$V_{long.1} := \alpha_p \cdot \left(\left(0.9 \cdot f_{pu}\right) \cdot \cos\left(\theta_{1.plan}\right) \cdot \cos\left(\theta_{1.ele}\right) - \left(f_{ps2}\right) \cdot \cos\left(\theta_{2.plan}\right) \cdot \cos\left(\theta_{2.ele}\right)\right) \cdot A_p = 216.18\ kip$$

Maximum friction force between the tendons and duct:

$\mu := 0.4$ $V_{long.2} := T_{vert} \cdot \mu = 18.88\ kip$

Governing longitudinal factored demands

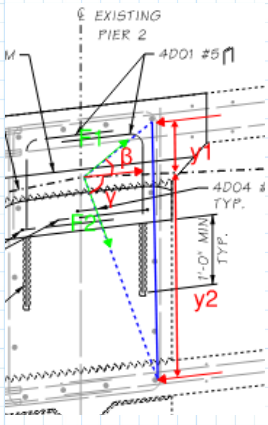
$$V_{long} := min\left(V_{long.1}, V_{long.2}\right) = 18.88\ kip$$

Tensile demand per tendon within diaphragm

Thickness of diaphragm $t := 2.5\ ft$

Vertical offset from top of duct to centroid of deck $y_1 := 12\ in - 2\ in - \frac{8.5}{2}\ in = 5.75\ in$

Vertical offset from top of duct to centroid of bot slab $y_2 := 4.25\ ft - y_1 - \frac{6}{2}\ in = 3.52\ ft$



Angle β $\beta := \operatorname{atan}\left(\frac{y_1}{\frac{t}{2}}\right) = 21\ deg$

Angle γ $\gamma := \operatorname{atan}\left(\frac{y_2}{\frac{t}{2}}\right) = 70.5\ deg$

From free body diagram:

$$\begin{aligned} &F1\cos(\beta)+F2\cos(\gamma)=V_{long}\\ &F1\sin(\beta)-F2\sin(\gamma)=0 \end{aligned}$$

$$multiplier:=\begin{bmatrix}\cos(\beta) & \cos(\gamma)\\ \sin(\beta) & -\sin(\gamma)\end{bmatrix} \qquad v:=\begin{bmatrix}V_{long}\\ 0\end{bmatrix}$$

$$F:=\text{lsolve}(multiplier,v)=\begin{bmatrix}17.8\\ 6.76\end{bmatrix}\ kip \qquad F_1:=F_0=17.8\ kip \qquad F_2:=F_1=6.76\ kip$$

Factored vertical tensile demand within diaphragm:

$$T_{dia.}:=\sin(\beta)\cdot F_1=6.4\ kip$$

Total tensile resistance for each tendon:

Resistance factor $\varphi:=0.9$

Assume the existing #4 are fully developed:

$$T_{dia.vertbar}:=\varphi\cdot f_y\cdot A_{bar.4}\cdot 2=21.6\ kip$$

D/C ratio $$DC_{longcheck.dia.1}:=\frac{T_{dia.}}{T_{dia.vertbar}}=0.29$$

2. Vertical downward force check

Vertical component $T_{vert}=47.2\ kip$

Number of #6 required $$N_{s6.req}:=\frac{T_{vert}}{\varphi\cdot A_{bar.6}\cdot f_y}=1.987$$

Number of #6 legs available for PT-1 $N_{s6.available}:=2\cdot 3=6$

D/C $$\frac{N_{s6.req}}{N_{s6.available}}=0.33$$

Downward force per anchorage $T_{vert}=47.21\ kip$

Horizontal bursting force $F_{P2.burst}:=0.25\cdot T_{vert}=11.8\ kip$

Required steel area $$A_{s.p2.burst}:=\frac{F_{P2.burst}}{\varphi\cdot f_y}=0.22\ in^2$$

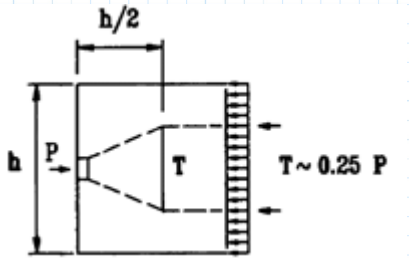


Figure C5.8.4.5.1-1—Effect of Discontinuity in Anchorage Zone

In the longitudinal and transverse direction, we have at least one #5 U-bar:

$A_{s5.available}:=0.31\ in^2$

D/C $$\frac{A_{s.p2.burst}}{A_{s5.available}}=0.71$$

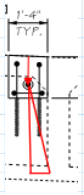
Demolition width $h:=12\ in+4\ in=16\ in$

Bursting force location $$d_{burst}:=\frac{h}{2}=8\ in$$

Therefore, we need 3-#6 U shape bars at 8" from the tendon to resist the bursting force

To transfer the vertical downward force from the anchorage to the web, the bottom slab will be in tension as shown in the sketch. This tension will be resisted by the transverse reinforcement in the bottom slab. The number of #4 required is:

$$N_{bottomslab}:=T_{vert}\cdot\frac{\tan(15\ deg)}{0.9\ (60\ ksi)\cdot A_{bar.4}}=1.17$$



The number of #4 available is:

$$N_{bottomslab.available} := 4$$

D/C

$$\frac{N_{bottomslab}}{N_{bottomslab.available}} = 0.29$$

Pier 3 Diaphragm Check

1. Description

This calculation checks and designs the pier 3 diaphragm based on AASHTO LRFD 2017. The calculations consist of three parts: 1) design for edge tension resistance ; 2) design for bursting force resistance; 3) check for concrete bearing strength.

2. Tendon Properties

Strand area	$A_{ps} := 0.217\ in^2$
Number of strand per tendon	$N_{ps} := 7$
Tendon area	$A_p := N_{ps} \cdot A_{ps} = 1.52\ in^2$
Tendon ultimate strength	$f_{pu} := 270 \cdot ksi$
Total design PT anchorage force	$F_{ps} := 1.2 \cdot 0.81 \cdot A_p \cdot f_{pu} = 398.65\ kip$

3. Mild Reinforcement Properties

Yield strength $f_y := 60 \cdot ksi$

Bar areas $A_{bar.4} := 0.2 \cdot in^2$ $A_{bar.5} := 0.31 \cdot in^2$ $A_{bar.6} := 0.44 \cdot in^2$ $A_{bar.7} := 0.60 \cdot in^2$ $A_{bar.8} := 0.79 \cdot in^2$

4. Edge Tension

Note: assuming 4% of the total PT force to be conservative

Edge Tension per tendon $T_{edge} := 0.04 \cdot F_{ps} = 15.95\ kip$

Resistance factor $\varphi := 0.9$

Required Steel $A_{s.edge} := \dfrac{T_{edge}}{\varphi \cdot f_y} = 0.3\ in^2$ AASHTO LRFD 5.5.4.2

Provide 2-#5 per anchorage, D/C is: $DC_{edge} := \dfrac{A_{s.edge}}{2 \cdot A_{bar.5}} = 0.48$

5. Bursting Force (LRFD 5.8.4.5.3)

Section height $h := 4 \cdot ft + 3 \cdot in = 51\ in$

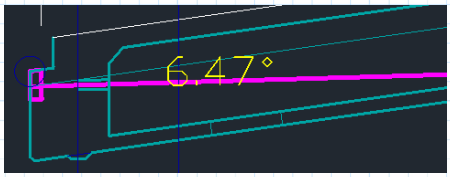
PT angle $\alpha := 6.47 \cdot deg$

Top slab thickness $t_t := 8.5\ in$

Bottom slab thickness $t_b := 6.5\ in$

Vertical offset of tendon end from top of the box $y := 13\ in + 6.5\ in = 19.5\ in$

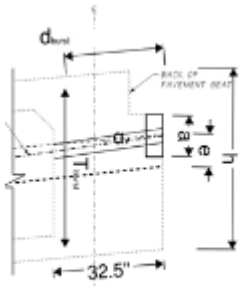
Note: The angle is measured from the autocad file "09.01.2021_XL5467_27_DE_BG_BR17.7-PTLayout". The angle is assumed to be the same between tendons



Assuming the centroid of the diaphragm cross-section is at the mid-height of the box:

Eccentricity of the anchorage device with respect to the centroid of the cross-section $e := \left| y - \dfrac{h}{2} \right| = 6\ in$

PT anchor plate dimension (use Dim B in DSI catalog) $a := 170 \cdot mm$



Bursting force (AASHTO 5.8.4.5.3) $T_{burst} := 0.25 \cdot F_{ps} \cdot \left(1 - \dfrac{a}{h}\right) + 0.5 \cdot F_{ps} \cdot \sin(\alpha) = 109.04\ kip$

Required area $A_{s.burst} := \dfrac{T_{burst}}{\varphi \cdot f_y} = 2.02\ in^2$

Bursting force location $d_{burst} := 0.5 \cdot (h - 2 \cdot e) + 5 \cdot e \cdot \sin(\alpha) = 22.88\ in$

As per AASHTO LRFD 5.9.5.6b, The bursting reinforcement should be distributed between anchor plate and 2.5d.bursting.

Conservatively assuming the PT anchor force will be take by a vertical deep beam

Compression force going to top flange $P_{up} := \dfrac{F_{ps} \cdot \left(h - y - \dfrac{t_b}{2}\right)}{h - \dfrac{t_b}{2} - \dfrac{t_t}{2}} = 258.89\ kip$

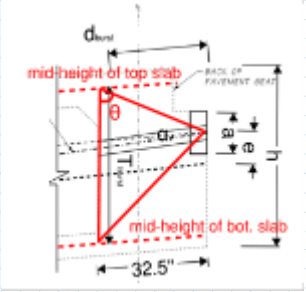
Thickness of diaphragm $t_{dia} := 32.5\ in$

Angle of the strut $\theta := \operatorname{atan}\left(\frac{t_{dia}}{y}\right) = 59.04\ deg$

So the vertical force $T := \frac{P_{up}}{\tan(\theta)} = 155.33\ kip$

Required rebar area $A_{s.f} := \frac{T}{\varphi \cdot f_y} = 2.88\ in^2$

Provide 4-#8 per anchorage, D/C is: $DC_{burst} := \frac{A_{s.f}}{4 \cdot A_{bar.8}} = 0.91$



6. Concrete bearing Strength

Approximately dead load at each web $V_{DL.web} := \frac{356 \cdot kip}{4} \cdot 1.2 = 106.8\ kip$

Note: this is from CSiBridge model load case "*DL3_noPT_AbutRemove_Short". The total shear force is divided by the number of the webs and increased by 20% to account for the uneven distribution of loads between webs

Given 3" on each side and plus the web, the bearing area width

$$B_{bearing} := 2 \cdot 3 \cdot in + 9 \cdot in = 15\ in$$

Allowable bearing strength of concrete $f_{c.b} := 0.7 \cdot 5 \cdot ksi = 3.5\ ksi$

Required length of the bearing area $L_{bearing} := \frac{1.25 \cdot V_{DL.web}}{f_{c.b} \cdot B_{bearing}} = 2.54\ in$

This is less than the thickness of the diaphragm (32.5 in)

By inspection, the check passes.

7. Summary

Required area for edge tension $A_{s.edge} = 0.3\ in^2$

Provide 2-#5 per anchorage, D/C is: $DC_{edge} := \frac{A_{s.edge}}{2 \cdot A_{bar.5}} = 0.48$

Required area for bursting $A_{s.req.burst} := \max\left(A_{s.burst}, A_{s.f}\right) = 2.88\ in^2$

Provide 4-#8 per anchorage, D/C is: $DC_{burst} := \frac{A_{s.req.burst}}{4 \cdot A_{bar.8}} = 0.91$

Required length of the bearing area $L_{bearing} = 2.54\ in$

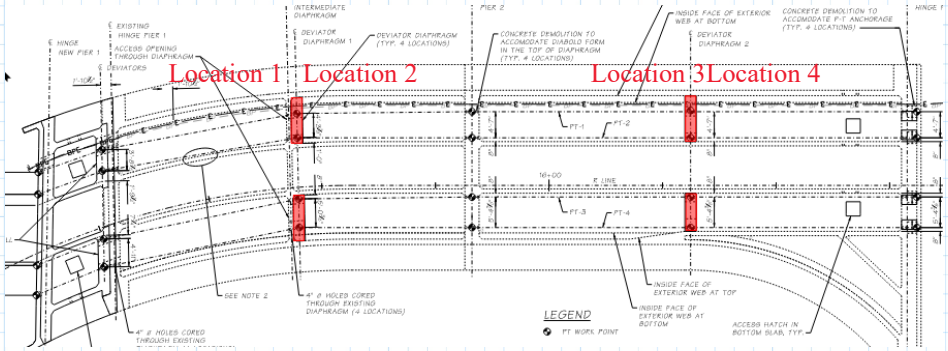
Provide the entire thickness of the diaphragm for bearing length, D/C is: $DC_{edge} := \frac{L_{bearing}}{t_{dia}} = 0.08$

Deviator Design

1. Description

The following items need to be check at the new deviator design

1. Vertical deviating force to be transferred to bottom flange and then to be transferred to webs or through a frame action to the entire section. For the tendon closed to the utility openings (PT-1), we did a frame analysis to make sure the vertical deviation force will be able to be transferred by a frame action because webs are far away. This sheet will focus on the checks for other three tendons. For a reference, the check for PT-1 is at \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\Design\Diaphragm Design\Utility Blockout Check and Analysis.
2. Longitudinal PT unbalanced longitudinal force needs to be transferred to the bottom flange.
3. Transverse PT force component needs to be transferred to the bottom flange.



2. Material Properties

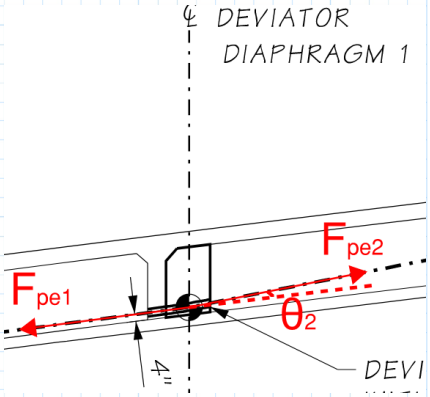
Concrete Compressive Strength	$f'_c := 5 \cdot ksi$
Typical Bar Yield Strength	$f_y := 60 \cdot ksi$
U-Shape Bolt Yield Strength	$f_{y.ushape} := 105 \cdot ksi$
L-Shape Bolt Yield Strength	$f_{y.lshape} := 60 \cdot ksi$

Note: for sizes of U-bolt: https://dalecompany.com/product-category/u-bolts/standard/

3. Structural Properties

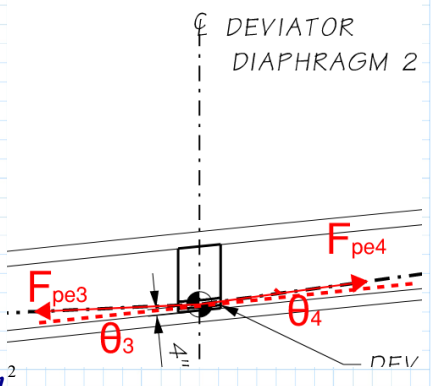
Thickness of bottom slab	$t_{slab} := 6 \cdot in$
Width of deviator	$b := 7 \cdot ft$

	deviator 1	deviator 2
L-Shape Bolt Qty per Diaphragm	$N_{b.lshape.dia.1} := 12$	$N_{b.lshape.dia.2} := 9$
Thicknesses deviators	$t_1 := 36 \cdot in$	$t_2 := 24 \cdot in$
Bolt Size	#6	#5
Bolt Area	$A_{b6} := 0.44 \cdot in^2$	$A_{b5} := 0.31 \cdot in^2$
Bolt diameter	$d_{b6} := 0.75 \cdot in$	$d_{b5} := 0.625 \cdot in$
U-Shape Bolt Qty per PT	$N_{b.ushape.pt.dev1} := 4$	$N_{b.ushape.pt.dev2} := 3$



4. Tendon Stresses

Tendon ultimate stress	$f_{pu} := 270 \ ksi$
Tendon average effective stress at Location 1	$f_{pe1} := 191 \ ksi$
Tendon average effective stress at Location 2	$f_{pe2} := 197 \ ksi$
Tendon average effective stress at Location 3	$f_{pe3} := 184 \ ksi$
Tendon average effective stress at Location 4	$f_{pe4} := 178 \ ksi$
Tendon segment 1 ULS loading stress (AASHTO 5.6.3.1)	$f_{ps1} := 15 \ ksi + f_{pe1} = 206 \ ksi$
Tendon segment 2 ULS loading stress (AASHTO 5.6.3.1)	$f_{ps2} := 15 \ ksi + f_{pe2} = 212 \ ksi$
Tendon segment 3 ULS loading stress (AASHTO 5.6.3.1)	$f_{ps3} := 15 \ ksi + f_{pe3} = 199 \ ksi$
Tendon segment 4 ULS loading stress (AASHTO 5.6.3.1)	$f_{ps4} := 15 \ ksi + f_{pe4} = 193 \ ksi$
Strand area	$A_{strand} := 0.217 \ in^2 = 0.217 \ in^2$
Tendon area	$A_{tendon} := 0.217 \ in^2 \cdot 7 = 1.519 \ in^2$



Note: The calculations for the average effective stresses are at:: \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\Design\Tendon Design\Tendon Losses, Jacking Force, and Elongations\20211117 Tendon Losses and Stresses

4. Vertical upward force check

Part 1: The first step is to check the through bolts. One thing to check here is the geometry check to determine how far the bolts need to be from the OD duct.

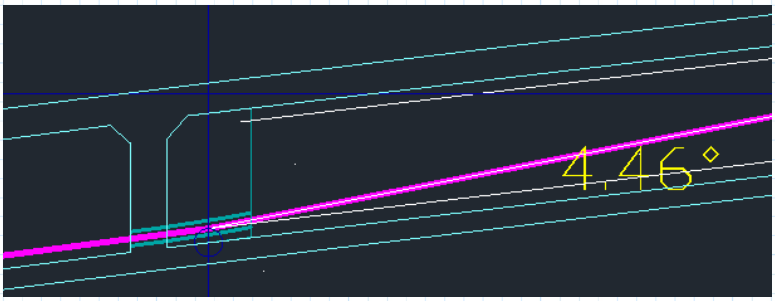
Part 2: The second step has two cases.

Case 1: the left tendon. We did the frame analysis a while ago to make sure the vertical deviation force will be able to be transferred by a frame action because webs are far away.

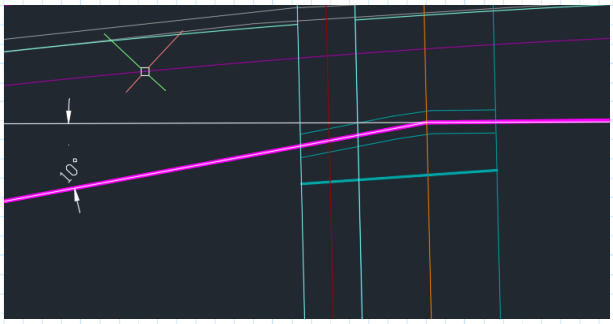
Case 2: Other tendons.

Part 1:

Demands at Deviator Diaphragm 1



Post-tensioning Elevation View at Deviator Diaphragm 1



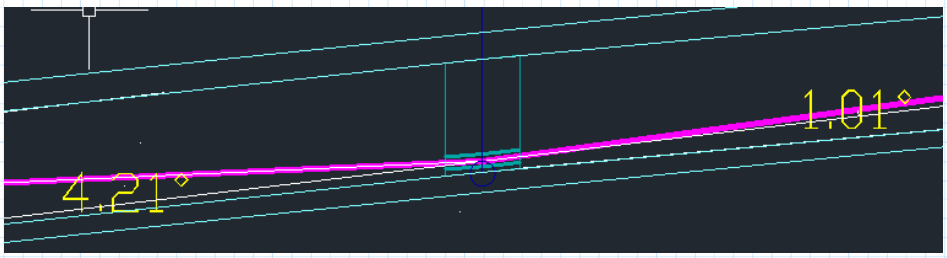
Post-tensioning Plan View at Deviator Diaphragm 1

Tendon angle change in elevation	$\theta_{1.ele} := 0\ deg$	$\theta_{2.ele} := 4.46\ deg$
Tendon angle change in plan	$\theta_{1.plan} := 10.85\ deg$	$\theta_{2.plan} := 0\ deg$
Demand factor	$\alpha_p := 1.2$	

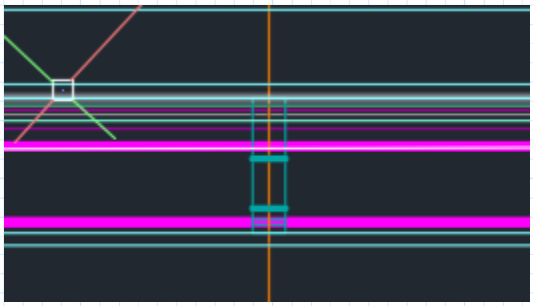
Factored tensile demand per tendon

$$T_{dev1} := \alpha_p \cdot A_{tendon} \cdot \left(f_{ps1} \cdot \sin\left(\theta_{1.ele}\right) + f_{ps2} \cdot \sin\left(\theta_{2.ele}\right)\right) = 30.05\ kip$$

Demands at Deviator Diaphragm 2



Post-tensioning Elevation View at Deviator Diaphragm 2



Post-tensioning Plan View at Deviator Diaphragm 2

Tendon angle change in elevation	$\theta_{3.ele} := 4.21$ *deg*	$\theta_{4.ele} := 1.01$ *deg*
Tendon angle change in plan	$\theta_{3.plan} := 0$ *deg*	$\theta_{4.plan} := 0$ *deg*
Demand factor	$\alpha_p := 1.2$	

Factored tensile demand per tendon

$$T_{dev2} := \alpha_p \cdot A_{tendon} \cdot \left(f_{ps3} \cdot \sin\left(\theta_{3.ele}\right) + f_{ps4} \cdot \sin\left(\theta_{4.ele}\right)\right) = 32.83 \ \textit{kip}$$

Bolts tensile capacity per tendon

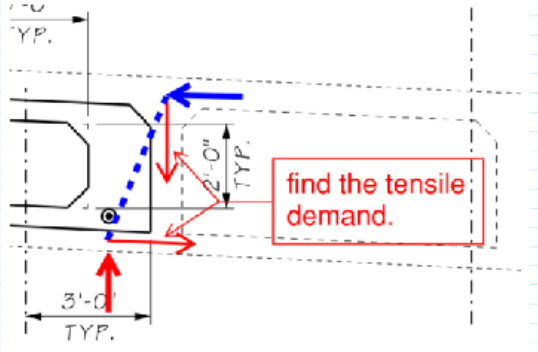
	Deviator 1	Deviator 2
Bolt Tensile Strength (17.4.1.2)	$T_{n1} := f_{y.ushape} \cdot N_{b.ushape.pt.dev1} \cdot A_{b6} \cdot 2 = 369.6$ *kip*	$T_{n2} := f_{y.ushape} \cdot N_{b.ushape.pt.dev2} \cdot A_{b5} \cdot 2 = 195.3$ *kip*
Resistance Factor (AASHTO 5.13.2.2)	$\phi_{tension} := 0.9$	$\phi_{tension} := 0.9$
Factored Bolt Resistance	$T_r := \phi_{tension} \cdot T_{n1} = 332.64$ *kip*	$T_r := \phi_{tension} \cdot T_{n2} = 175.77$ *kip*
D/C ratios	$DC_{vertcheck.bolt.1} := \frac{T_{dev1}}{T_r} = 0.17$	$DC_{vertcheck.bolt.2} := \frac{T_{dev2}}{T_r} = 0.19$

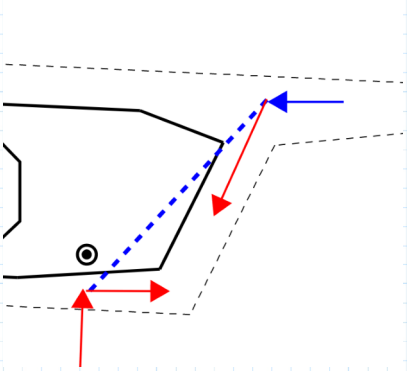
Part 2 (Case 1):

For the demand and capacity for PT-1, the check is at \\COWI.net\projects\A205000\A207833\30-ENG\LDLI\Design\Diaphragm Design\Utility Blockout Check and Analysis.

Part 2 (Case 2):

Tensile demand per tendon within web and bottom flange





Assume the angle changes and tendon stresses are close between tendons. Then the tensile demand within the web and bottom flange will increase when the eccentricity between the tendon and web increases.

Deviator Diaphragm 1 - Interior Cell

Factored tensile demand in web	$T_{web.dev1.int} := T_{dev1} = 30.05$ *kip*	
Horizontal eccentricity between tendon and cl web	$e_h := 1\ \textit{ft} + \frac{9}{2}\ \textit{in} = 1.38\ \textit{ft}$	Note: PT-2 is 1ft away from the face of interior web face. The interior web is 9 in thick
Vertical eccentricity between cl slabs	$e_v := 4.25\ \textit{ft} - \frac{(6\ \textit{in} + 8.5\ \textit{in})}{2} = 3.65\ \textit{ft}$	

Factored tensile demand in bottom slab $T_{botslab.dev1.int} := T_{dev1} \cdot \frac{e_h}{e_v} = 11.33\ kip$

Deviator Diaphragm 1 - Exterior Cell

Factored tensile demand in web $T_{web.dev1.ext} := \frac{T_{dev1}}{2} \cdot \sqrt{5} = 33.6\ kip$ Note: exterior web slope is 2:1

Horizontal eccentricity $e_h := 14\ in + \frac{10}{2}\ in + \frac{(4.25\ ft - (6\ in + 8.5\ in))}{2} = 3.1\ ft$

Vertical eccentricity between slabs $e_v := 4.25\ ft - \frac{(6\ in + 8.5\ in)}{2} = 3.65\ ft$

Factored tensile demand in bottom slab $T_{botslab.dev1.ext} := T_{dev1} \cdot \frac{e_h}{e_v} = 25.59\ kip$

Governing demand in web between interior and exterior cells $T_{web.dev1} := \max\left(T_{web.dev1.int}, T_{web.dev1.ext}\right) = 33.6\ kip$

Governing demand in bottom slab between interior and exterior cells $T_{botslab.dev1} := 0.25 \cdot \max\left(T_{botslab.dev1.int}, T_{botslab.dev1.ext}\right) = 6.4\ kip$

Deviator Diaphragm 2 - Interior Cell

Factored tensile demand in web $T_{web.dev2.int} := T_{dev2} = 32.83\ kip$

Horizontal eccentricity between tendon and web $e_h := 8\ in + \frac{9}{2}\ in = 1.04\ ft$

Vertical eccentricity between slabs $e_v := 4.25\ ft - \frac{(6.5\ in + 8.5\ in)}{2} = 3.63\ ft$

Factored tensile demand in bottom slab $T_{botslab.dev2.int} := T_{web.dev1} \cdot \frac{e_h}{e_v} = 9.65\ kip$

Deviator Diaphragm 2 - Exterior Web

Factored tensile demand in web $T_{web.dev2.ext} := \frac{T_{dev2}}{2} \cdot \sqrt{5} = 36.71\ kip$

Horizontal eccentricity $e_h := 11\ in + \frac{10}{2}\ in + \frac{(4.25\ ft - (6.5\ in + 8.5\ in))}{2} = 2.83\ ft$

Vertical eccentricity between slabs $e_v := 4.25\ ft - \frac{(6.5\ in + 8.5\ in)}{2} = 3.63\ ft$

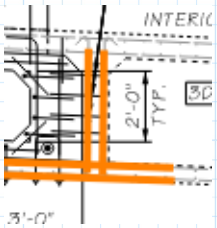
Factored tensile demand in bottom slab $T_{botslab.dev2.ext} := T_{dev2} \cdot \frac{e_h}{e_v} = 25.66\ kip$

Governing demand in web between interior and exterior cells $T_{web.dev2} := \max\left(T_{web.dev2.int}, T_{web.dev2.ext}\right) = 36.71\ kip$

Governing demand in bottom slab between interior and exterior cells (assume 25% of force taken by tension and 75% of force taken by compression on the other side) $T_{botslab.dev2} := 0.25 \cdot \max\left(T_{botslab.dev2.int}, T_{botslab.dev2.ext}\right) = 6.42\ kip$

Tensile Capacity

Deviator Diaphragm 1



Resistance Factor (AASHTO 5.13.2.2) $\phi_{tension} := 0.9$

Number of #5 in each web within 3ft thick diaphragm $N_{vert.web} := 6$

#5 bar area $A_{bar.5} := 0.31\ in^2$

Number of #4 in bottom slab within 3ft thick diaphragm $N_{hori.botslab1} := 6$

#4 bar area $A_{bar.4} := 0.2\ in^2$

Number of #5 required in web $N_{web.dev1} := \frac{T_{web.dev1}}{\phi_{tension} \cdot f_y \cdot A_{bar.5}} = 2$

D/C ratio $DC_{vertcheck.web.1} := \frac{N_{web.dev1}}{N_{vert.web}} = 0.33$

Number of #4 required in bottom slab $N_{slab.dev1} := \frac{T_{botslab.dev1}}{\phi_{tension} \cdot f_y \cdot A_{bar.4}} = 0.6$

D/C ratio $DC_{vertcheck.botslab.1} := \frac{N_{slab.dev1}}{N_{hori.botslab1}} = 0.1$

Deviator Diaphragm 2

Number of #4 in bottom slab within 2ft thick diaphragm $N_{hori.botslab2} := 4$

Number of #5 required in web $N_{web.dev2} := \frac{T_{web.dev2}}{\phi_{tension} \cdot f_y \cdot A_{bar.5}} = 2.2$

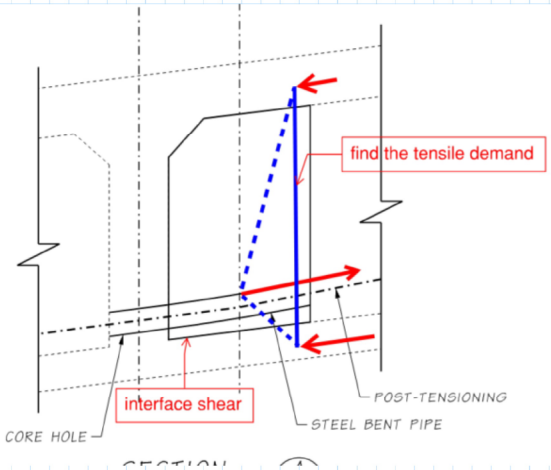
D/C ratio $DC_{vertcheck.web.2} := \frac{N_{web.dev2}}{N_{vert.web}} = 0.37$

Number of #4 required in bottom slab $N_{slab.dev2} := \frac{T_{botslab.dev2}}{\phi_{tension} \cdot f_y \cdot A_{bar.4}} = 0.6$

D/C ratio $DC_{vertcheck.botslab.2} := \frac{N_{slab.dev2}}{N_{hori.botslab2}} = 0.15$

2. longitudinal check

The longitudinal check consists of two parts:
Part 1: check the vertical rebars resistance; and
Part 2: check interface shear resistance between the bottom slab and deviator diaphragms



Part 1: check the vertical rebars resistance

Longitudinal shear demand per tendon

Deviator Diaphragm 1

Stress difference at ultimate flexural limit states:

$$V_{dev1.long.1} := \alpha_p \cdot \left(\left(0.9 \cdot f_{pu}\right) \cdot \cos\left(\theta_{1.plan}\right) \cdot \cos\left(\theta_{1.ele}\right) - \left(f_{ps2}\right) \cdot \cos\left(\theta_{2.plan}\right) \cdot \cos\left(\theta_{2.ele}\right) \right) \cdot A_{tendon} = 49.76 \ \textit{kip}$$

Maximum friction force between the tendons and duct:

Friction coefficient (AASHTO 5.9.3.2.2 takes 0.23, but taken as 0.4 to be conservative)

$\mu := 0.4$ $\quad V_{dev1.long.2} := T_{dev1} \cdot \mu = 12.02 \ \textit{kip}$

Governing longitudinal factored demands

$$V_{dev1.long} := min\left(V_{dev1.long.1}, V_{dev1.long.2}\right) = 12.02 \ \textit{kip}$$

Deviator Diaphragm 2

Stress difference at ultimate flexural limit states:

$$V_{dev2.long.1} := \alpha_p \cdot \left(\left(0.9 \cdot f_{pu}\right) \cdot \cos\left(\theta_{3.ele}\right) \cdot \cos\left(\theta_{3.plan}\right) - \left(f_{ps4}\right) \cdot \cos\left(\theta_{4.ele}\right) \cdot \cos\left(\theta_{4.plan}\right) \right) \cdot A_{tendon} = 90 \ \textit{kip}$$

Maximum friction force between the tendons and duct:

$$V_{dev2.long.2} := T_{dev2} \cdot \mu = 13.13 \ \textit{kip}$$

Governing longitudinal factored demands

$$V_{dev2.long} := min\left(V_{dev2.long.1}, V_{dev2.long.2}\right) = 13.13 \ \textit{kip}$$

Tensile demand per tendon within diaphragm

Deviator Diaphragm 1

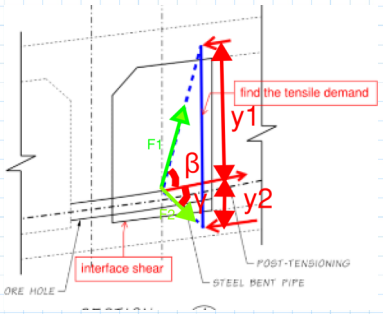
Vertical offset from top of duct to centroid of tendon (AASHTO Figure C5.9.1.6-1) $z := 0.5 \ \textit{in}$

Vertical offset from top of duct to centroid of deck $y_1 := 4.25 \ \textit{ft} - 4 \ \textit{in} - 6 \ \textit{in} - \frac{8.5}{2} \ \textit{in} - z = 3.02 \ \textit{ft}$

Vertical offset from top of duct to centroid of bot slab $y_2 := z + 4 \ \textit{in} + \frac{6}{2} \ \textit{in} = 0.63 \ \textit{ft}$

Angle β $\beta := \operatorname{atan}\left(\frac{y_1}{\frac{t_1}{2}}\right) = 63.6 \ \textit{deg}$

Angle γ $\gamma := \operatorname{atan}\left(\frac{y_2}{\frac{t_1}{2}}\right) = 22.6 \ \textit{deg}$



From free body diagram:

$$F1\cos(\beta) + F2\cos(\gamma) = V_{dev1.long}$$
$$F1\sin(\beta) - F2\sin(\gamma) = 0$$

$$multiplier := \begin{bmatrix} \cos(\beta) & \cos(\gamma) \\ \sin(\beta) & -\sin(\gamma) \end{bmatrix} \qquad v := \begin{bmatrix} V_{dev1.long} \\ 0 \end{bmatrix}$$

$$F := \operatorname{lsolve}\left(multiplier, v\right) = \begin{bmatrix} 4.63 \\ 10.79 \end{bmatrix} \ \textit{kip} \qquad F_1 := F_0 = 4.63 \ \textit{kip} \qquad F_2 := F_1 = 10.79 \ \textit{kip}$$

Factored vertical tensile demand within diaphragm:

$$T_{dia.dev1} := \sin(\beta) \cdot F_1 = 4.1 \ \textit{kip}$$

Deviator Diaphragm 2

Vertical offset from top of duct to centroid of tendon (AASHTO Figure C5.9.1.6-1) $z := 0.5 \ \textit{in}$

Vertical offset from top of duct to centroid of deck $y_1 := 4.25 \ \textit{ft} - 4 \ \textit{in} - 6.5 \ \textit{in} - \frac{8.5}{2} \ \textit{in} - z = 2.979 \ \textit{ft}$

Vertical offset from top of duct to centroid of bot slab $y_2 := z + 4 \ \textit{in} + \frac{6.5}{2} \ \textit{in} = 0.646 \ \textit{ft}$

Angle β $\beta := \operatorname{atan}\left(\frac{y_1}{\frac{t_2}{2}}\right) = 71.4 \ \textit{deg}$

Angle γ $\gamma := \operatorname{atan}\left(\frac{y_2}{\frac{t_2}{2}}\right) = 32.9 \ \textit{deg}$

From free body diagram:

$$F1\cos(\beta)+F2\cos(\gamma)=V_{dev2.long}$$
$$F1\sin(\beta)-F2\sin(\gamma)=0$$

$$multiplier:=\begin{bmatrix}\cos(\beta) & \cos(\gamma)\\ \sin(\beta) & -\sin(\gamma)\end{bmatrix}\qquad v:=\begin{bmatrix}V_{dev2.long}\\ 0\end{bmatrix}$$

$$F:=\text{lsolve}(multiplier,v)=\begin{bmatrix}7.35\\ 12.85\end{bmatrix}\ \textit{kip}$$

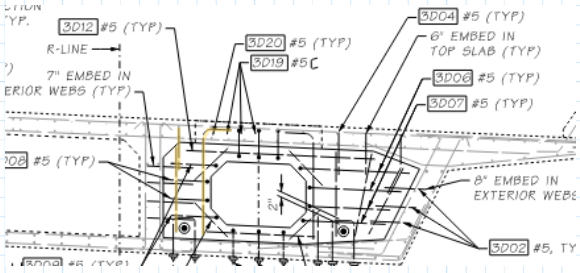
$$F_1:=F_0=7.35\ \textit{kip}$$

Factored tensile demand within diaphragm:

$$T_{dia.dev2}:=\sin(\beta)\cdot F_1=6.97\ \textit{kip}$$

Tensile Capacities

Deviator Diaphragm 1



Total tensile resistance for each tendon:

Note: As shown in the sketch above, this tensile stress will be resisted by 2-#5. However, the yield strength of the straight bar and the hook bar are reduced by 80% and 50% respectively due to the limit of development length at top. The required development length is 20" for the straight uncoated #5 and 10.2" for hooked uncoated #5, but only ~4" available for both.

$$T_{dia.dev1.vertbar}:=0.2\ \phi_{tension}\cdot f_y\cdot A_{bar.5}+0.5\ \phi_{tension}\cdot f_y\cdot A_{bar.5}=11.72\ \textit{kip}$$

D/C ratio

$$DC_{longcheck.dia.1}:=\frac{T_{dia.dev1}}{T_{dia.dev1.vertbar}}=0.35$$

Deviator Diaphragm 2

Similar to Deviator Diaphragm 1, the total tensile resistance for each tendon:

$$T_{dia.dev2.vertbar}:=0.2\ \phi_{tension}\cdot f_y\cdot A_{bar.5}+0.5\ \phi_{tension}\cdot f_y\cdot A_{bar.5}=11.72\ \textit{kip}$$

D/C ratio

$$DC_{longcheck.dia.2}:=\frac{T_{dia.dev2}}{T_{dia.dev2.vertbar}}=0.59$$

$$A_{cv.dev2}:=t_2\cdot\frac{b}{2}=\left(1.01\cdot10^3\right)\ \textit{in}^2$$

Part 2: Check interface shear resistance between diaphragm and bottom slab (per cell)

Cohesion factor (AASHTO 5.7.4.3)	$c:=0.28\cdot\textit{ksi}$	Note: assume normal weight concrete cip on clean concrete surface, free of laitance with surface roughened to an amplitude of 0.25 in
Friction factor(AASHTO 5.7.4.3)	$\mu:=1$	
Fraction of concrete strength available to resist interface shear (AASHTO 5.7.4.3)	$K_1:=0.3$	
Limiting interface shear resistance (AASHTO 5.7.4.3)	$K_2:=1.8\cdot\textit{ksi}$	

Deviator Diaphragm 1

Area of concrete considered to be engaged in interface shear transfer per cell for dev 1 (AASHTO 5.7.4.2)

$$A_{cv.dev1}:=t_1\cdot\frac{b}{2}=\left(1.51\cdot10^3\right)\ \textit{in}^2$$

Note: use 50% to be conservative

Nominal shear resistance of interface plane for deviator 1 (AASHTO 5.7.4.3)

$$V_{ni.dev1}:=c\cdot A_{cv.dev1}+\mu\cdot\left(f_{y.ushape}\cdot N_{b.ushape.pt.dev1}\cdot A_{b6}\cdot2\cdot2+f_{y.lshape}\cdot N_{b.lshape.dia.1}\cdot A_{b5}\right)=\left(1.39\cdot10^3\right)\ \textit{kip}$$

The nominal shear resistance used in the design shall not exceed either of the following:

$V_{ni.1} := K_1 \cdot f'_c \cdot A_{cv.dev1} = 2268$ *kip* $\quad V_{ni.2} := K_2 \cdot A_{cv.dev1} = 2721.6$ *kip*

Therefore: $V_{ni.limit} := min\left(V_{ni.1}, V_{ni.2}\right) = 2268$ *kip*

Check for shear force resistance for deviator 1

$$V_{ni.dev1} := \left\| \begin{array}{l} \text{if } V_{ni.dev1} \le V_{ni.limit} \\ \quad \| \, V_{ni.dev1} \\ \text{if } V_{ni.dev1} > V_{ni.limit} \\ \quad \| \, V_{ni.limit} \end{array} \right\| = \left(1.39 \cdot 10^3\right) \text{ kip}$$

Shear force resistance factor (AASHTO 5.5.4.2) $\quad \phi_{interfaceshear} := 0.9$

Factored interface shear resistance per diaphragm for dev 1 $\quad V_{ri.dev1} := V_{ni.dev1} \cdot \phi_{interfaceshear} = \left(1.25 \cdot 10^3\right)$ *kip*

Area of concrete considered to be engaged in interface shear transfer (AASHTO 5.7.4.2) $\quad A_{cv.dev1} = \left(1.51 \cdot 10^3\right)$ *in*^{2}

Minimum area of interface shear reinforcement crossing the shear plane within the area (AASHTO 5.7.4.2)

$$A_{cf} := \frac{0.05 \cdot A_{cv.dev1}}{\frac{f_{y.lshape}}{ksi}} = 1.26 \text{ in}^2$$

Area of interface shear reinforcement crossing the shear plane within the area

$A_{total} := N_{b.ushape.pt.dev1} \cdot A_{b6} \cdot 2 \cdot 2 + N_{b.lshape.dia.1} \cdot A_{b5} = 10.76$ *in*^{2}

Check for minimum area $\quad$ **if** $\left(A_{total} < A_{cf}, \text{“fail”}, \text{“ok”}\right) = \text{“ok”}$

D/C ratio for deviator diaphragm 1

$$DC_{longcheck.bolt.1} := \frac{V_{dev1.long}}{V_{ri.dev1}} = 0.01$$

Deviator Diaphragm 2

Area of concrete considered to be engaged in interface shear transfer per cell for dev 2 (AASHTO 5.7.4.2) $\quad A_{cv.dev2} := t_2 \cdot \frac{b}{2} = \left(1.01 \cdot 10^3\right)$ *in*^{2} $\quad$ Note: use 50% to be conservative

Nominal shear resistance of interface plane for deviator 2 (AASHTO 5.7.4.3)

$V_{ni.dev2} := c \cdot A_{cv.dev2} + \mu \cdot \left(f_{y.ushape} \cdot N_{b.ushape.pt.dev2} \cdot A_{b5} \cdot 2 \cdot 2 + f_{y.lshape} \cdot N_{b.lshape.dia.2} \cdot A_{b5}\right) = 840.24$ *kip*

The nominal shear resistance used in the design shall not exceed either of the following:

$V_{ni.1} := K_1 \cdot f'_c \cdot A_{cv.dev2} = 1512$ *kip* $\quad V_{ni.2} := K_2 \cdot A_{cv.dev2} = 1814.4$ *kip*

Therefore: $V_{ni.limit} := min\left(V_{ni.1}, V_{ni.2}\right) = 1512$ *kip*

Check for shear force resistance for deviator 1

$$V_{ni.dev2} := \left\| \begin{array}{l} \text{if } V_{ni.dev1} \le V_{ni.limit} \\ \quad \| \, V_{ni.dev1} \\ \text{if } V_{ni.dev1} > V_{ni.limit} \\ \quad \| \, V_{ni.limit} \end{array} \right\| = \left(1.39 \cdot 10^3\right) \text{ kip}$$

Shear force resistance factor (AASHTO 5.5.4.2) $\quad \phi_{interfaceshear} := 0.9$

Factored interface shear resistance per diaphragm for dev 1 $\quad V_{ri.dev2} := V_{ni.dev2} \cdot \phi_{interfaceshear} = \left(1.25 \cdot 10^3\right)$ *kip*

Area of concrete considered to be engaged in interface shear transfer (AASHTO 5.7.4.2) $\quad A_{cv.dev2} = \left(1.01 \cdot 10^3\right)$ *in*^{2}

Minimum area of interface shear reinforcement crossing the shear plane within the area (AASHTO 5.7.4.2)

$$A_{cf} := \frac{0.05 \cdot A_{cv.dev2}}{\frac{f_{y.lshape}}{ksi}} = 0.84\ in^2$$

Area of interface shear reinforcement crossing the shear plane within the area

$$A_{total} := N_{b.ushape.pt.dev2} \cdot A_{b5} \cdot 2 \cdot 2 + N_{b.lshape.dia.2} \cdot A_{b5} = 6.51\ in^2$$

Check for minimum area

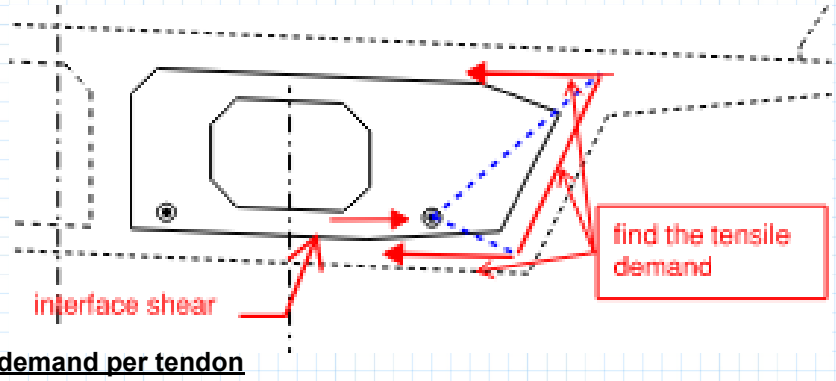
$$\text{if}\left(A_{total} < A_{cf}, \text{“fail”}, \text{“ok”}\right) = \text{“ok”}$$

D/C ratio for deviator diaphragm 1

$$DC_{longcheck.bolt.2} := \frac{V_{dev2.long}}{V_{ri.dev2}} = 0.01$$

3. Transverse force check

Three things need to be checked.
Part 1- bursting
Part 2- how the force will be transferred to bottom slab
Part 3- Interface shear



Transverse shear demand per tendon

Deviator Diaphragm 1

Stress difference at ultimate flexural limit states:

$$V_{dev1.trans.1} := \left(\left\langle f_{ps1}\right\rangle \cdot \sin\left(\theta_{1.plan}\right) \cdot \cos\left(\theta_{1.ele}\right) + \left\langle f_{ps2}\right\rangle \cdot \sin\left(\theta_{2.plan}\right) \cdot \cos\left(\theta_{2.ele}\right)\right) \cdot A_{tendon} = 58.9\ kip$$

Governing transverse factored demands

$$V_{dev1.trans} := V_{dev1.trans.1} = 58.9\ kip$$

Deviator Diaphragm 2

$$V_{dev1.trans.1} := \left(\left\langle f_{ps3}\right\rangle \cdot \sin\left(\theta_{3.plan}\right) \cdot \cos\left(\theta_{3.ele}\right) + \left\langle f_{ps4}\right\rangle \cdot \sin\left(\theta_{4.plan}\right) \cdot \cos\left(\theta_{4.ele}\right)\right) \cdot A_{tendon} = 0\ kip$$

Part 1 - Bursting

Deviator Diaphragm 1

Downward force per anchorage

$$P := V_{dev1.trans}$$

Horizontal bursting force (AASHTO C5.8.4.5.1-1)

$$F_{P2.burst} := 0.25 \cdot P = 14.73\ kip$$

Required steel area

$$A_{s.p2.burst} := \frac{F_{P2.burst}}{\phi_{tension} \cdot f_y} = 0.27\ in^2$$

Four #6 U-bar area is:

$$A_{available} := N_{b.ushape.pt.dev1} \cdot A_{b6} = 1.76\ in^2$$

D/C ratio

$$DC_{transcheck.bolt.burst.1} := \frac{A_{s.p2.burst}}{A_{available}} = 0.15$$

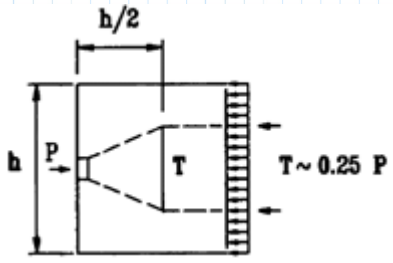
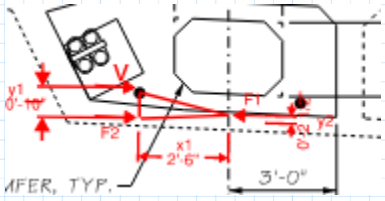


Figure C5.8.4.5.1-1—Effect of Discontinuity in Anchorage Zone

Part 2 - Tension in bottom slab and web

PT-1 and PT-3

Strut and Tie model is as shown on the right. The offsets are measured using bluebeam.



$y_1 := 10\ in \qquad y_2 := 2.5\ in \qquad x_1 := 2.5\ ft$

From free body diagram the vertical component between the tendon and the bottom slab is:
:

$$F_1 := V_{dev1.trans} \cdot \frac{(y_1 - y_2)}{x_1} = 14.73\ kip$$

Number of #5 required to resist the vertical force in the diaphragm between tendon and bottom slab

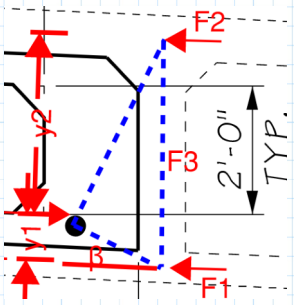
$$N_{vert.dev1} := \frac{F_2}{\phi_{tension} \cdot f_y \cdot A_{bar.5}} = 0.64$$

There are at least 2-#5 vertical legs per tendon, therefore the D/C ratio is:

$N_{vert.dev1.available} := 2$ $\qquad DC_{transcheck.vert.1} := \frac{N_{vert.dev1}}{N_{vert.dev1.available}} = 0.32$

PT-2

Vertical offset from top of duct to centroid of tendon (AASHTO Figure C5.9.1.6-1) $\qquad z := 0.5\ in$



Vertical offset from top of duct to centroid of bot slab $\qquad y_1 := z + 4\ in + \frac{6}{2}\ in = 7.5\ in$

Vertical offset from top of duct to centroid of top slab $\qquad y_2 := 4.25\ ft - \frac{8.5\ in}{2} - y_1 = 3.271\ ft$

From free body diagram: $\qquad F_2 := \frac{V_{dev1.trans}}{\left(1 + \frac{y_2}{y_1}\right)} = 9.45\ kip \qquad F_1 := V_{dev1.trans} - F_2 = 49.45\ kip$

From free body diagram:

$$\frac{F_1}{F_3} = \frac{1}{\tan(\beta)}$$

Horizontal offset from duct to the intersection of centerlines of web and bottom slab $\qquad x_1 := 1\ ft + \frac{9\ in}{2} = 1.38\ ft$

Angle β $\qquad \beta := \operatorname{atan}\left(\frac{y_1}{x_1}\right) = 24.4\ deg$

$$F_3 := \frac{F_1}{\frac{1}{\tan(\beta)}} = 22.48\ kip$$

Number of #4 required in bottom slab for each tendon:

$$N_{botslab.dev1} := \frac{F_1}{\phi_{tension} \cdot f_y \cdot A_{bar.4}} = 4.58$$

D/C ratio $\qquad DC_{transcheck.botslab.1} := \frac{N_{botslab.dev1}}{N_{hori.botslab1}} = 0.76$

Number of #5 required in web for each tendon:

$$N_{web.dev1} := \frac{F_3}{\phi_{tension} \cdot f_y \cdot A_{bar.5}} = 1.34$$

D/C ratio $\qquad DC_{transcheck.intweb.1} := \frac{N_{web.dev1}}{N_{vert.web}} = 0.22$

Number of #4 available in top slab for each tendon:

$$N_{hori.topslab} := 6$$

Number of #4 required in top slab for each tendon:

$$N_{topslab.dev1} := \frac{F_2}{\phi_{tension} \cdot f_y \cdot A_{bar.4}} = 0.87$$

D/C ratio $\qquad DC_{transcheck.topslab.1} := \frac{N_{topslab.dev1}}{N_{hori.topslab}} = 0.15$

PT-4

Vertical offset from top of duct to centroid of tendon (AASHTO Figure C5.9.1.6-1)

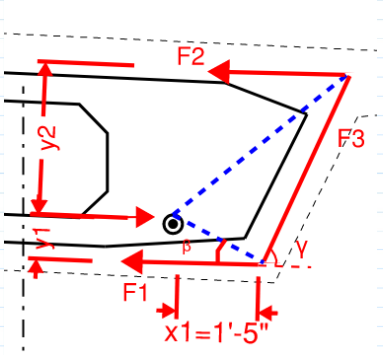
$$z := 0.5\ in$$

Vertical offset from top of duct to centroid of bot slab

$$y_1 := z + 4\ in + \frac{6}{2}\ in = 7.5\ in$$

Vertical offset from top of duct to centroid of top slab

$$y_2 := 4.25\ ft - \frac{8.5\ in}{2} - y_1 = 3.271\ ft$$



From free body diagram:

$$F_2 := \frac{V_{dev1.trans}}{\left(1 + \frac{y_2}{y_1}\right)} = 9.45\ kip \qquad F_1 := V_{dev1.trans} - F_2 = 49.45\ kip$$

From free body diagram:

$$\frac{F_1 - F_3 \cdot \cos(\gamma)}{F_3 \cdot \sin(\gamma)} = \frac{1}{\tan(\beta)}$$

Horizontal offset from duct to the intersection of centerlines of web and bottom slab (measured using bluebeam)

$$x_1 := 1\ ft + 5\ in = 1.417\ ft$$

Angle β

$$\beta := \operatorname{atan}\left(\frac{y_1}{x_1}\right) = 23.8\ deg$$

Angle γ

$$\gamma := \operatorname{atan}\left(\frac{2}{(1)}\right) = 63.43\ deg$$

$$F_3 := \frac{F_1}{\frac{1}{\tan(\beta)} \cdot \sin(\gamma) + \cos(\gamma)} = 19.98\ kip$$

Number of #4 required in bottom slab for each tendon:

$$N_{botslab.dev1} := \frac{F_1}{\phi_{tension} \cdot f_y \cdot A_{bar.4}} = 4.58$$

D/C ratio

$$DC_{transcheck.botslab.1} := \frac{N_{botslab.dev1}}{N_{hori.botslab1}} = 0.76$$

Number of #5 required in web for each tendon:

$$N_{web.dev1} := \frac{F_3}{\phi_{tension} \cdot f_y \cdot A_{bar.5}} = 1.19$$

D/C ratio

$$DC_{transcheck.extweb.1} := \frac{N_{web.dev1}}{N_{vert.web}} = 0.2$$

Number of #4 available in top slab for each tendon:

$$N_{hori.topslab} := 6$$

Number of #4 required in top slab for each tendon:

$$N_{topslab.dev1} := \frac{F_2}{\phi_{tension} \cdot f_y \cdot A_{bar.4}} = 0.87$$

D/C ratio

$$DC_{transcheck.topslab.1} := \frac{N_{topslab.dev1}}{N_{hori.topslab}} = 0.15$$

Part 3 - Interface shear

Resistance is the same as the interface shear resistance for the longitudinal shear, therefore:

D/C ratio for deviator diaphragm 1

$$DC_{transcheck.bolt.intershear.1} := \frac{V_{dev1.trans}}{V_{ri.dev1}} = 0.05$$

4. Summary

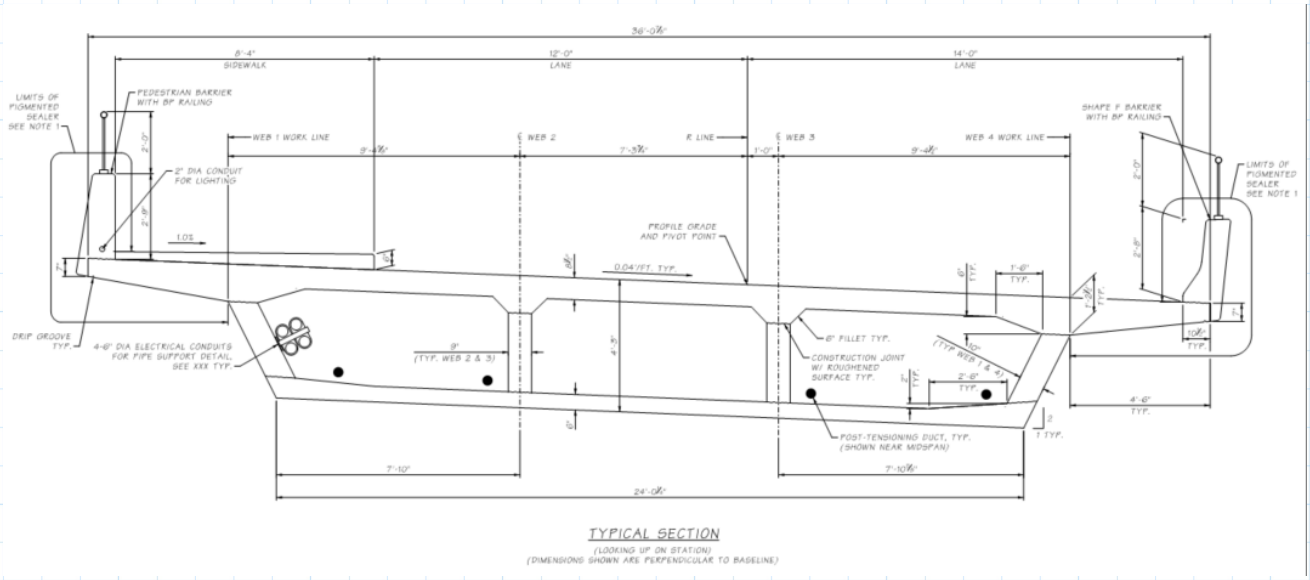
For U-shape bolts: (8-#6 vertical legs around each tendon for deviator 1 and 6-#5 for deviator 2)	Deviator 1	$DC_{vertcheck.bolt.1} + DC_{longcheck.bolt.1} + DC_{transcheck.bolt.burst.1} + DC_{transcheck.bolt.intershear.1} = 0.4$
	Deviator 2	$DC_{vertcheck.bolt.2} + DC_{longcheck.bolt.2} = 0.2$
For web reinforcement: (4-#5 legs in each web)	Deviator 1	$DC_{vertcheck.web.1} + DC_{transcheck.intweb.1} = 0.6$
	Deviator 2	$DC_{vertcheck.web.2} = 0.4$
For bottom slab transverse reinforcement: (6-#4 below deviator 1 and 4-#4 below deviator 2)	Deviator 1	$DC_{vertcheck.botslab.1} + DC_{transcheck.botslab.1} = 0.9$
	Deviator 2	$DC_{vertcheck.botslab.2} = 0.1$
For diaphragm vertical reinforcement: (2-#5 at least around each tendon)	Deviator 1	$DC_{longcheck.dia.1} + DC_{transcheck.vert.1} = 0.7$
	Deviator 2	$DC_{longcheck.dia.2} = 0.6$
For top slab transverse reinforcement: (6-#4 within at deviator 1)	Deviator 1	$DC_{transcheck.topslab.1} := \frac{N_{topslab.dev1}}{N_{hori.topslab}} = 0.15$

Deviator Diaphragm 1 PT-1 Deviation Force and Capacity Check

1. Purpose

1. The purpose is to calculate the force effects and calculate the capacity in the top and bottom slabs at the deviator diaphragm 1 when there is a utility opening on the diaphragm.

2. Bridge Typical Cross-Section



Note:
1. The cross-section super-elevation is assumed to be zero.
2. All the webs are assumed to be vertical. The thickness of the webs are adjusted such that they have the same area as shown in the actual cross-section.
3. The thicknesses of the top and bottom slab are assumed to be uniform. In other word, fillets and tapered overhang are ignored, but their area are accounted for in the calculation of slab areas and equivalent slab thicknesses.

Deck top slab width	$B_{top} := 36\ ft + 0.875\ in = 36.073\ ft$
Deck bottom slab width	$B_{bottom} := 24\ ft + 0.25\ in = 24.021\ ft$
Overhang flange width	$B_{overhang} := 4.5\ ft$
Deck total height	$H_{total} := 4.25\ ft$
Top slab area	$A_{top} := 30.1\ ft^2$
Bottom slab area	$A_{bottom} := 14.8\ ft^2$
Exterior web area	$A_{extweb} := 2.2\ ft^2$
Interior web area	$A_{intweb} := 1.9\ ft^2$
Top slab thickness	$t_{topslab} := \frac{A_{top}}{B_{top}} = 10.013\ in$
Bottom slab thickness	$t_{botslab} := \frac{A_{bottom}}{B_{bottom}} = 7.394\ in$
Web height	$H_{web} := H_{total} - t_{topslab} - t_{botslab} = 2.799\ ft$
Exterior web thickness	$t_{extweb} := \frac{A_{extweb}}{H_{web}} = 9.43\ in$

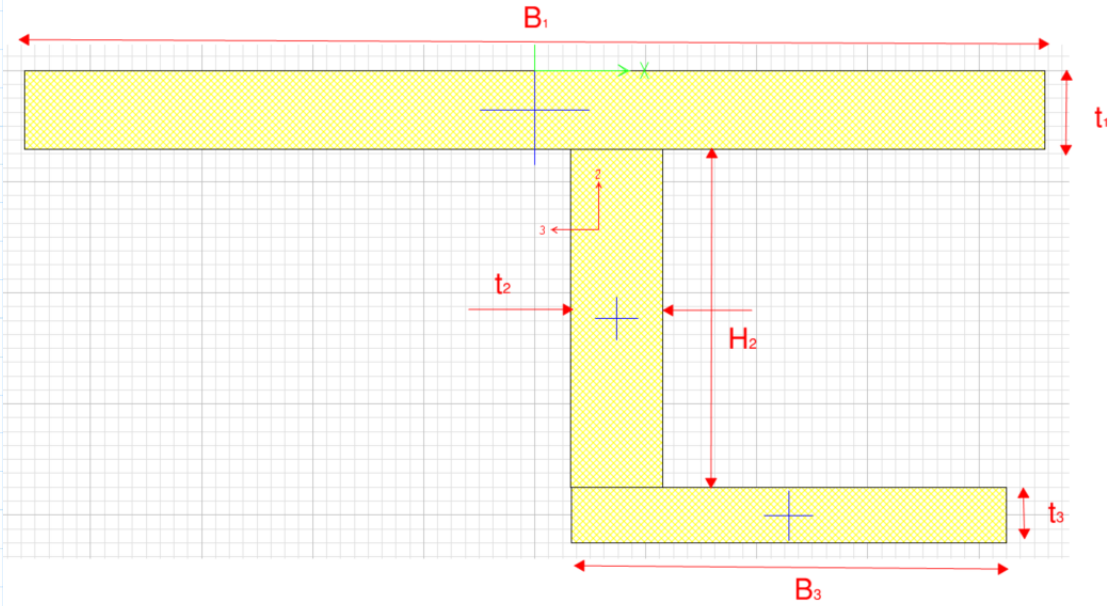
Interior web thickness $t_{intweb} := \frac{A_{intweb}}{H_{web}} = 8.144\ in$

3. Moment of Inertia of Individual Web

The moment of inertia of individual web about the horizontal axis is determined using tributary flange width of each web.

Top width of left cell $B_{c1t} := 9\ ft + 4.625\ in = 9.385\ ft$
Top width of mid cell $B_{c2t} := 8\ ft + 3.75\ in = 8.313\ ft$
Top width of right cell $B_{c3t} := 9\ ft + 4.5\ in = 9.375\ ft$
Bottom width of left cell $B_{c1b} := 7\ ft + 10\ in = 7.833\ ft$
Bottom width of mid cell $B_{c3b} := 7\ ft + 10.625\ in = 7.885\ ft$
Bottom width of right cell $B_{c2b} := B_{bottom} - B_{c1b} - B_{c3b} = 8.302\ ft$

3.1 Moment of Inertia of Exterior Webs



For left exterior web:

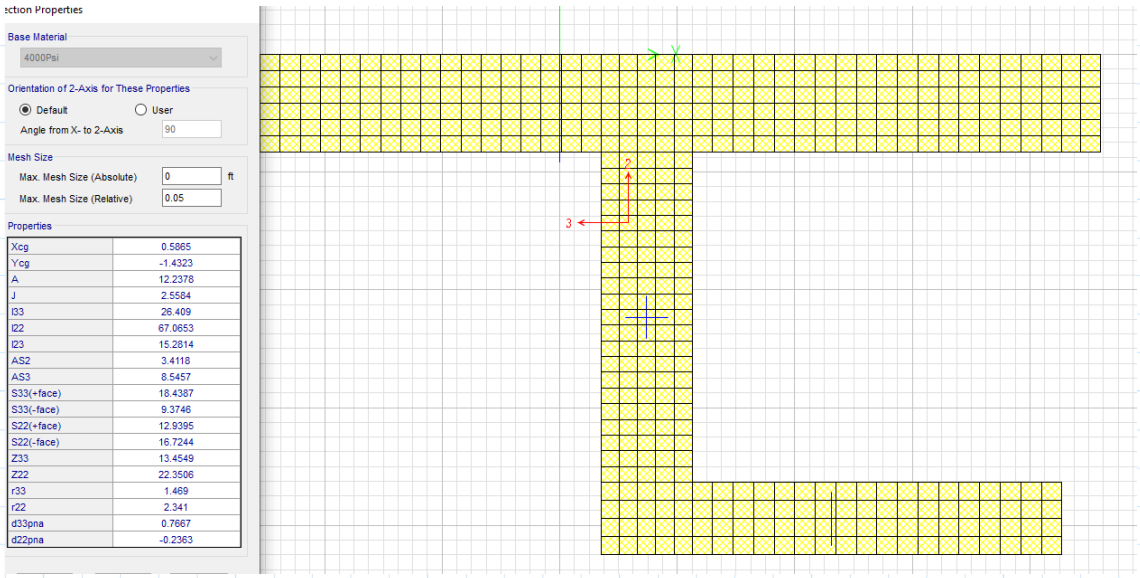
$B_1 := B_{overhang} + \frac{B_{c1t}}{2} = 9.193\ ft$ $t_1 := t_{topslab} = 10.013\ in$

$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.799\ ft$ $t_2 := t_{extweb} = 9.43\ in$

$B_3 := \frac{B_{c1b}}{2} = 3.917\ ft$ $t_3 := t_{botslab} = 7.394\ in$

The moment of inertia about horizontal axis is estimated using csibridge cross-section designer:

$$I_{web1} := 26.41\ ft^4$$



For right exterior web:

$$B_1 := B_{overhang} + \frac{B_{c3t}}{2} = 9.188\ \textit{ft}$$

$$t_1 := t_{topslab} = 10.013\ \textit{in}$$

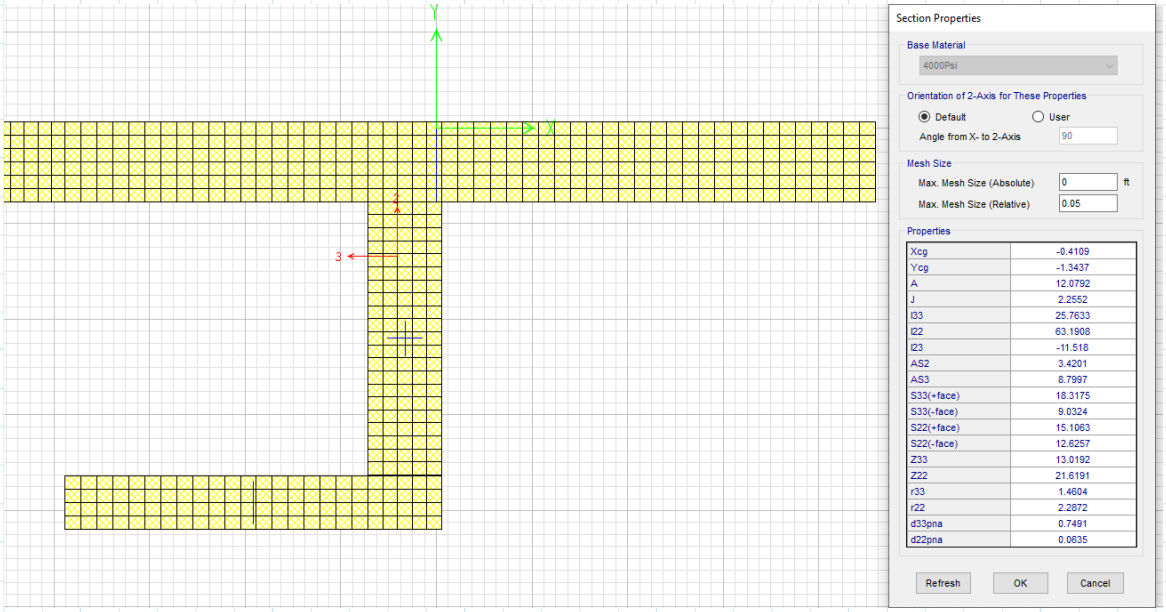
$$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.799\ \textit{ft}$$

$$t_2 := t_{extweb} = 9.43\ \textit{in}$$

$$B_3 := \frac{B_{c3b}}{2} = 3.943\ \textit{ft}$$

$$t_3 := t_{botslab} = 7.394\ \textit{in}$$

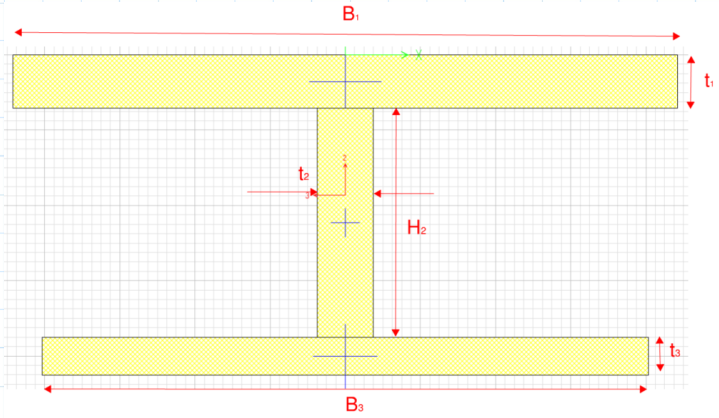
$$I_{web4} := 26.127\ \textit{ft}^4$$



3.2 Moment of Inertia of Interior Webs

Note: Similar to the estimation of exterior webs, the fillets are ignored and the super-elevation of the cross-section is assumed to be zero

For left interior web:



$$B_1 := \frac{B_{c1t}}{2} + \frac{B_{c2t}}{2} = 8.849\ ft \qquad t_1 := t_{topslab} = 10.013\ in$$

$$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.799\ ft \qquad t_2 := t_{intweb} = 8.144\ in$$

$$B_3 := \frac{B_{c1b}}{2} + \frac{B_{c2b}}{2} = 8.068\ ft \qquad t_3 := t_{botslab} = 7.394\ in$$

$$I_{web2} := 40.223\ ft^4$$

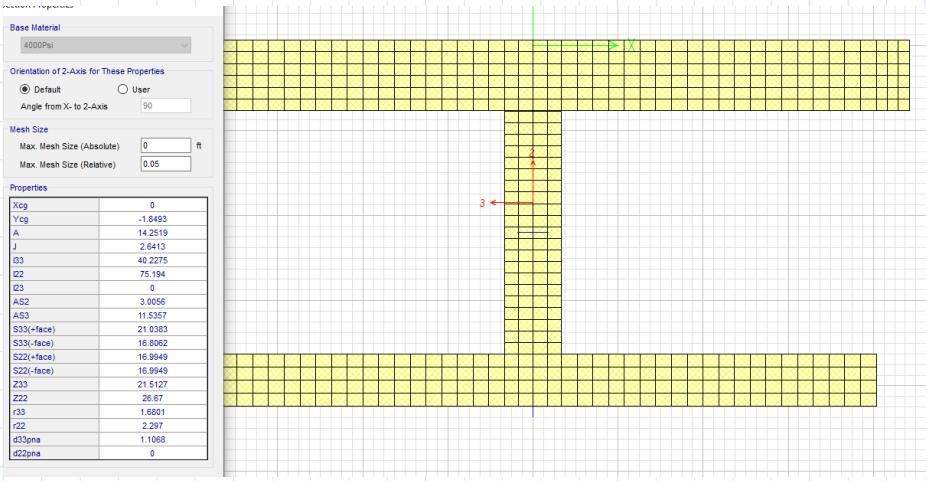
For right interior web:

$$B_1 := \frac{B_{c2t}}{2} + \frac{B_{c3t}}{2} = 8.844\ ft \qquad t_1 := t_{topslab} = 10.013\ in$$

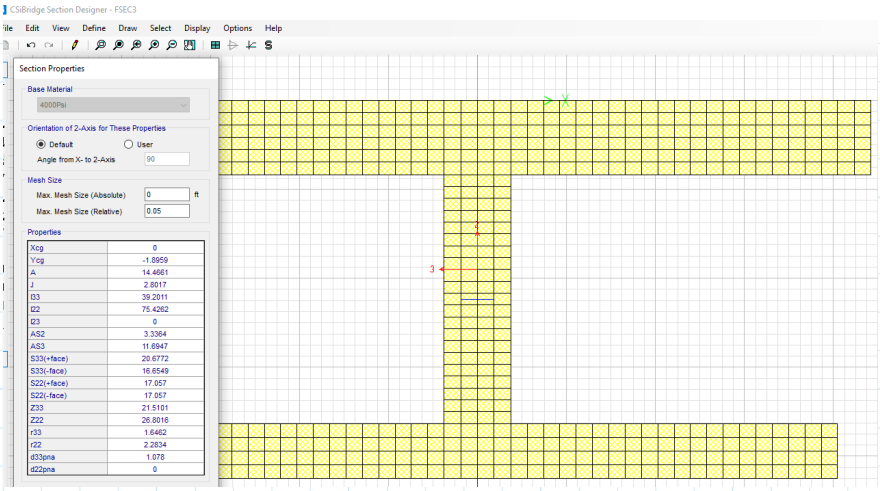
$$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.799\ ft \qquad t_2 := t_{intweb} = 8.144\ in$$

$$B_3 := \frac{B_{c2b}}{2} + \frac{B_{c3b}}{2} = 8.094\ ft \qquad t_3 := t_{botslab} = 7.394\ in$$

$$I_{web3} := 39.201\ ft^4$$



Screenshot of left interior web cross-section properties



Screenshot of right interior web cross-section properties

4. Stiffness of Individual Webs

A 100 kip point load is applied at the deviator and the deflection at the point is:

$$F_{deviation} := 100\ \boldsymbol{kip}$$
$$\Delta := 0.0072\ \boldsymbol{ft} = 0.086\ \boldsymbol{in}$$

Therefore the overall vertical deflection stiffness at the deviator location is:

$$k_{total} := \frac{F_{deviation}}{\Delta} = \left(1.157 \cdot 10^{3}\right)\ \frac{\boldsymbol{kip}}{\boldsymbol{in}}$$

The vertical spring stiffness of each web is determined based on the moment of inertia of each web calculated in Section 3

$$k_{web1} := k_{total} \cdot \frac{I_{web1}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 231.638\ \frac{\boldsymbol{kip}}{\boldsymbol{in}}$$

$$k_{web2} := k_{total} \cdot \frac{I_{web2}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 352.789\ \frac{\boldsymbol{kip}}{\boldsymbol{in}}$$

$$k_{web3} := k_{total} \cdot \frac{I_{web3}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 343.825\ \frac{\boldsymbol{kip}}{\boldsymbol{in}}$$

$$k_{web4} := k_{total} \cdot \frac{I_{web4}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 229.155\ \frac{\boldsymbol{kip}}{\boldsymbol{in}}$$

5. Deviation Forces

Note: The deviation forces are calculated based on the tendon vertical profiles and tendon forces at the strength limit state.

Tendon area $A_p := 0.217\ in^2 \cdot 7 = 1.519\ in^2$

Tendon force after losses (from csibridge) $P_e := 257.12\ kip$

Tendon stress after losses $f_{pe} := \frac{P_e}{A_p} = 169.269\ ksi$

Tendon stress at strength limit state $f_{ps} := 15\ ksi + f_{pe} = 184.269\ ksi$

Tendon force at the strength limit state $P_{ps} := A_p \cdot f_{ps} = 279.905\ kip$

Each tendon is divided into segments based on the slope. Assuming the origin is at the east end of the bridge at the top of the superstructure, the horizontal (longitudinal) and vertical coordinates of the ends of the tendon segments adjacent to the deviator are:

$x_{tendon1} := 76.63\ ft$ $y_{tendon1} := 1.063\ ft$

$x_{tendon2} := 105.63\ ft$ $y_{tendon2} := 3.479\ ft$

$x_{tendon3} := 138.93\ ft$ $y_{tendon3} := 3.479\ ft$

The angles between the tendon and the horizontal (longitudinal) axis on the x-y plane are:

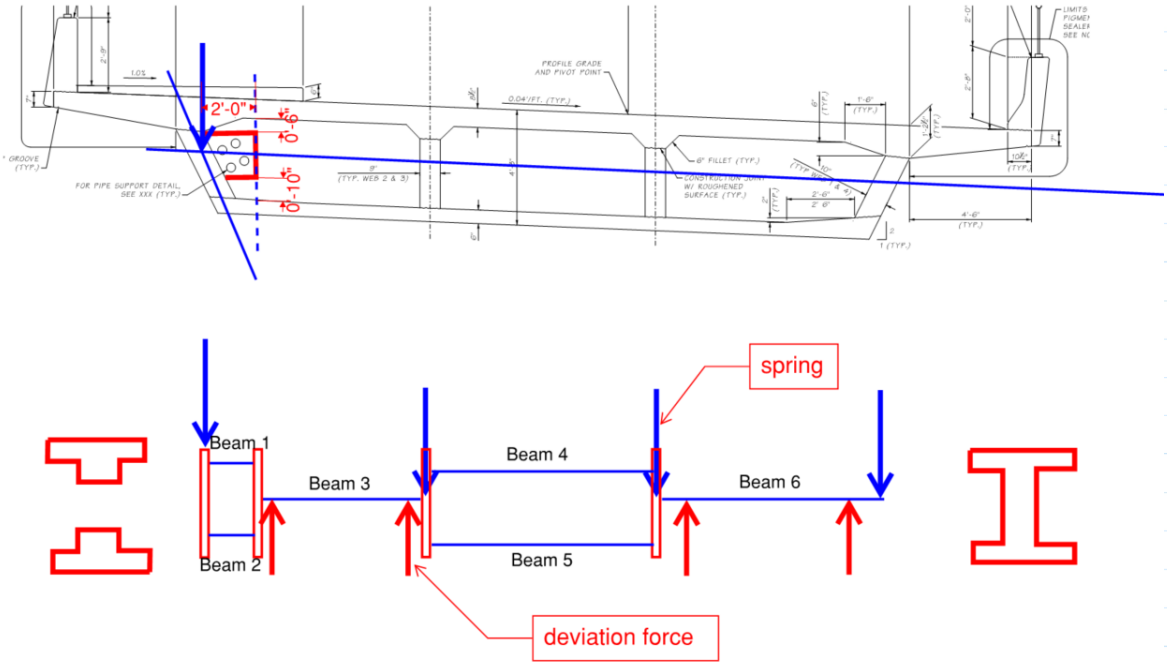
$$\alpha_1 := \frac{y_{tendon2} - y_{tendon1}}{x_{tendon2} - x_{tendon1}} = 0.083 \qquad \operatorname{atan}(\alpha_1) = 4.762\ deg$$

$$\alpha_2 := \frac{y_{tendon3} - y_{tendon2}}{x_{tendon3} - x_{tendon2}} = 0 \qquad \operatorname{atan}(\alpha_2) = 0\ deg$$

Therefore, the deviation force at the deviator is:

$$F_{deviation} := P_{ps} \cdot \sin(\operatorname{atan}(\alpha_1)) - P_{ps} \cdot \sin(\operatorname{atan}(\alpha_2)) = 23.238\ kip$$

6. Model Setup



Beam 1 and Beam 2

From csibridge cross-section, the centroid of the T section of beam 1 is at **0.56'** below the top surface of the cross-section and the centroid of the inversed T section of beam 2 is at **0.59'** above the bottom surface of the cross-section. Therefore, the vertical spacing between beams 1 and 2 is:

$$y_{12} := H_{total} - 0.56\ ft - 0.59\ ft = 3.1\ ft$$

The length of beams 1 and 2 is the horizontal offsets between the edge of the opening to the centerline of the exterior web. The offset is measured using bluebeam as:

$$L_{12} := 2\ ft$$

Beam 3 and Beam 6

The cross-section of beams 3 and 6 are in I shape. The flange thickness are consistent the slab widths. The flange widths equal to the height of the box girder cross-section. The width of the web equals the thickness of the diaphragm.

The length of beam 3 is the horizontal distance from the edge of the opening to the centerline of the left interior web:

$$L_3 := B_{c1t} - L_{12} = 7.385\ ft$$

The lengths of 6 are assumed to equal to the top width of the cells:

$$L_6 := B_{c3t} = 9.375\ ft$$

The location of beams 3 and 6 are assumed to be at the centroid I shape, which is at **2.07 ft** below the top of the deck. The location of the centroid if obtained from the csibridge section designer.

Beam 4 and Beam 5

The cross-section of beams 4 and 5 are in rectangular shape with thickness equal to the thickness of the slab and width equal to the height of the box-girder.

The lengths of beams 4 and 5 are assumed to equal to the top width of the cells:

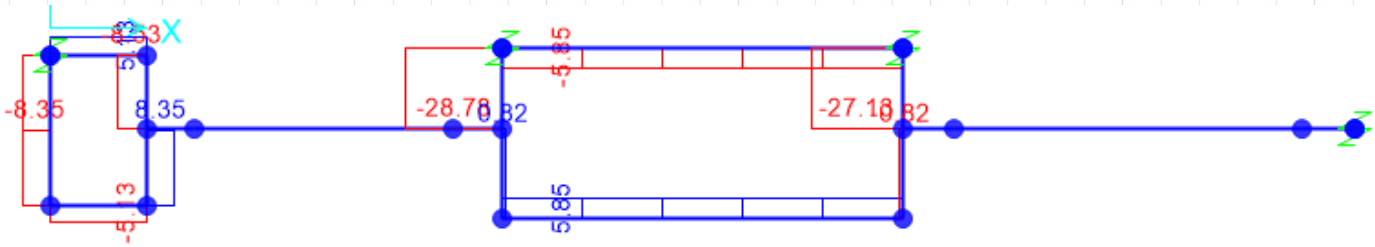
$$L_4 := B_{c2t} = 8.313\ ft$$
$$L_5 := B_{c2t} = 8.313\ ft$$

The locations of beams 4 and 5 are assumed to be at centroid of the slabs, so beam 4 is at **10.01"/2=5.0"=0.42'** below the top of the deck and beam 5 is at **4.25'-7.39"/2=3.94'** below the top of the deck

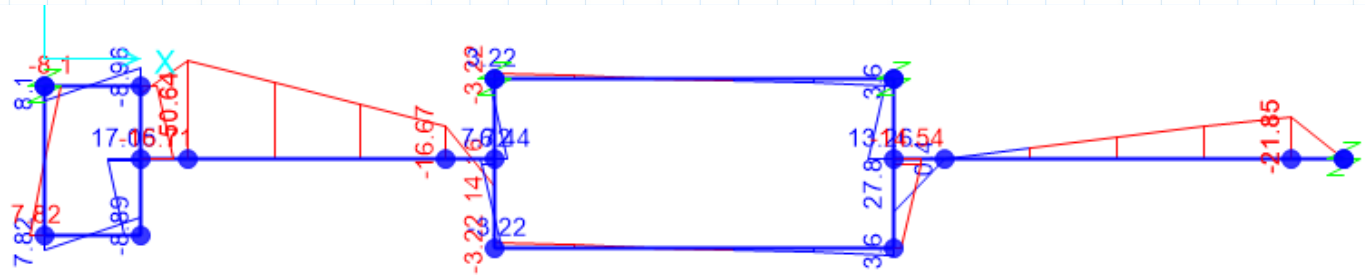
The location of the deviation force are assumed to be 8 in from the adjacent vertical cell surface

7. Results

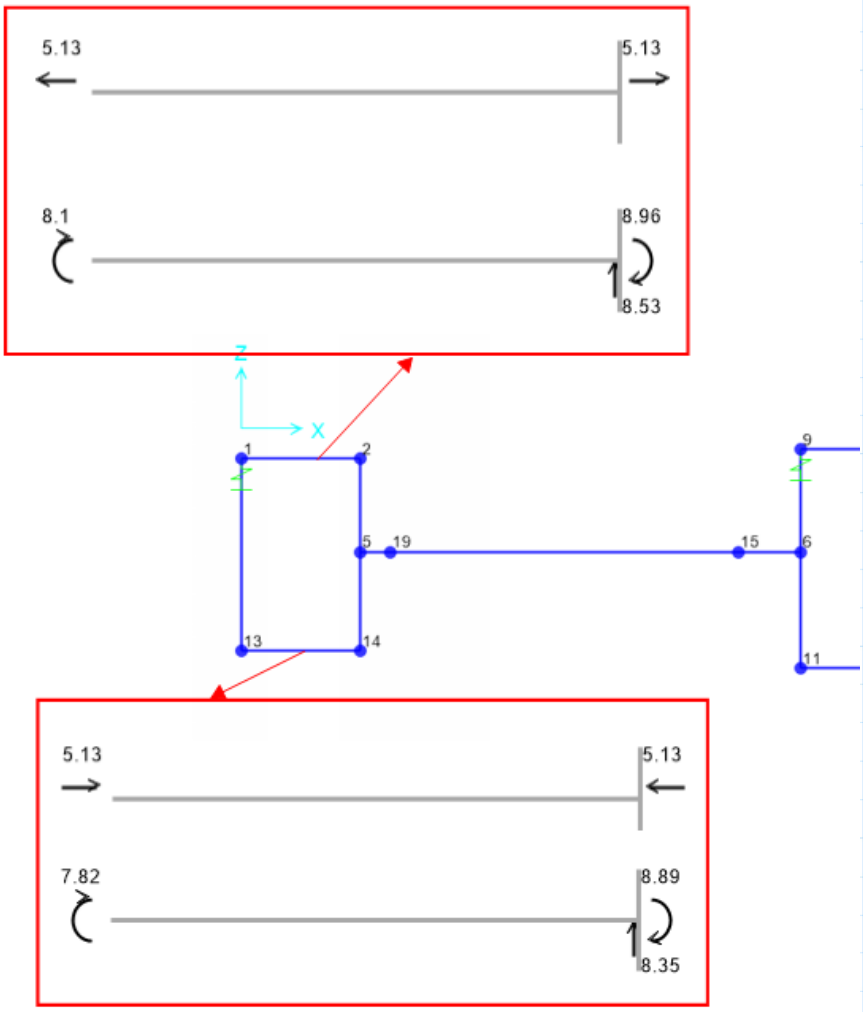
Axial Force Diagram (kip)



Bending Moment Diagram (kip.ft)



Forces on the Diaphragm with openings



8. Capacity Summary

8.1 Resistance Factor

As per AASHTO 5.5.4.2, the resistance factor for non-prestressed members is:

and for nonprestressed members such that:

$$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9 \qquad (5.5.4.2\text{-}2)$$

Tension controlled strain limit (AASHTO C T5.6.2.1-1) $\varepsilon_{tl} := 0.005$

Compression controlled strain limit (AASHTO C T5.6.2.1-1) $\varepsilon_{cl} := 0.002$

Therefore, if the tensile strain in the reinforcement is greater than 0.005, the resistance factor is 0.9.

8.2 Demand and Factored Capacity

Note: the capacities are obtained from RESPONSE 2000

Top Diaphragm

Demand	$N_{tp} := 5.13\ kip$	$M_{tp} := 8.1\ kip \cdot ft$	$N_{tn} := 5.13\ kip$	$M_{tn} := -8.96\ kip \cdot ft$
Unfactored Capacity	$N_{tpc} := 5.13\ kip$	$M_{tpc} := 107.3\ kip \cdot ft$	$N_{tnc} := 5.13\ kip$	$M_{tnc} := -141.3\ kip \cdot ft$

Tensile Strain in Extreme Tension Steel: $\varepsilon_{ttp} := 0.00487$ $\varepsilon_{ttn} := 0.00265$

Resistance Factor: $\phi_{tp} := 0.75 + \frac{0.15 \cdot (\varepsilon_{ttp} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.894$ $\phi_{pn} := 0.75 + \frac{0.15 \cdot (\varepsilon_{ttn} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.783$

D/C: $\frac{M_{tp}}{M_{tpc} \cdot \phi_{tp}} = 0.1$ $\frac{M_{tn}}{M_{tnc} \cdot \phi_{pn}} = 0.1$

Bottom Diaphragm

Demand	$N_{bp} := -5.13\ kip$	$M_{bp} := 7.82\ kip \cdot ft$	$N_{bn} := -5.13\ kip$	$M_{bn} := -8.89\ kip \cdot ft$
Unfactored Capacity	$N_{bpc} := -5.13\ kip$	$M_{bpc} := 90\ kip \cdot ft$	$N_{bnc} := -5.13\ kip$	$M_{bnc} := -49.9\ kip \cdot ft$

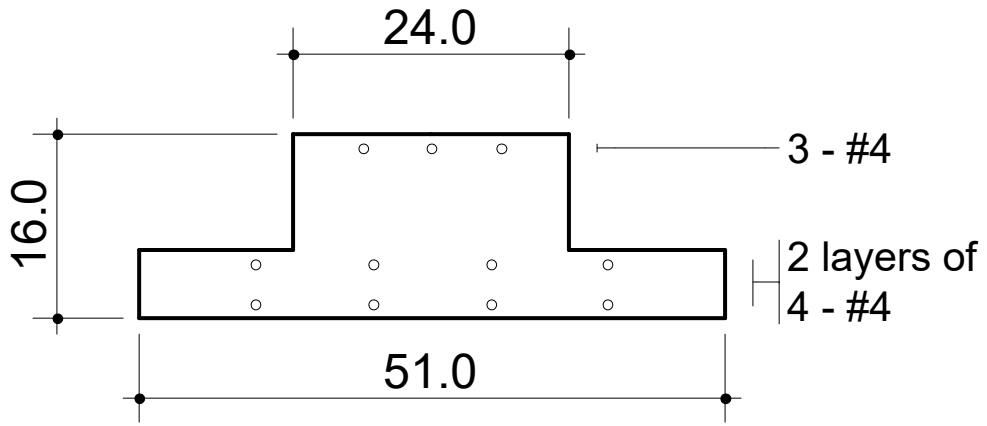
Tensile Strain in Extreme Tension Steel: $\varepsilon_{tbp} := 0.00231$ $\varepsilon_{tbn} := 0.00265$

Resistance Factor: $\phi_{bp} := 0.75 + \frac{0.15 \cdot (\varepsilon_{tbp} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.766$ $\phi_{bn} := 0.75 + \frac{0.15 \cdot (\varepsilon_{tbn} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.783$

D/C: $\frac{M_{bp}}{M_{bpc} \cdot \phi_{bp}} = 0.1$ $\frac{M_{bn}}{M_{bnc} \cdot \phi_{bn}} = 0.2$

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	546.0	561.5
Inertia (in^4)	11526.3	11986.5
y_t (in)	9.5	9.5
y_b (in)	6.5	6.5
S_t (in^3)	1215.4	1262.8
S_b (in^3)	1768.8	1841.8



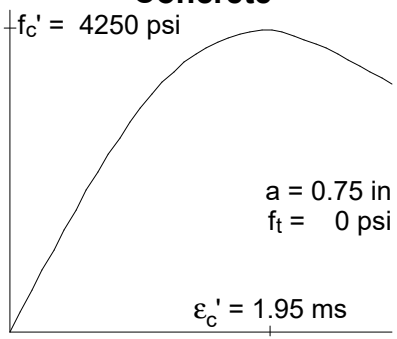
Crack Spacing

2 x dist + 0.1 d_b /ρ

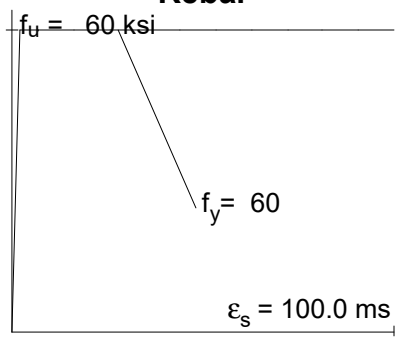
Loading (N,M,V + dN,dM,dV)

-5.13 , 7.82 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.00 in

WSpan Bottom Diaphragm

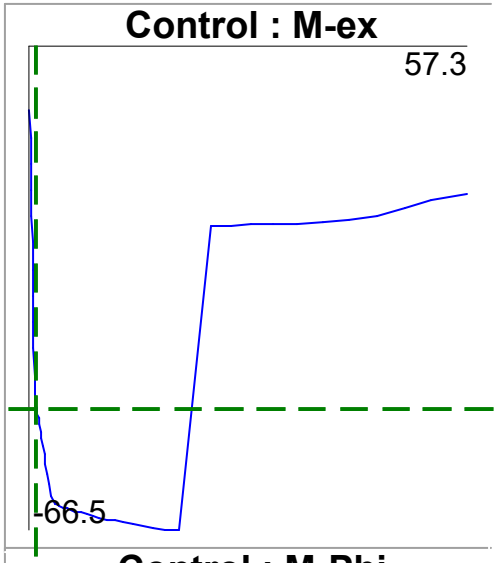
2021/3/26

Response-2000 v 1.0.5

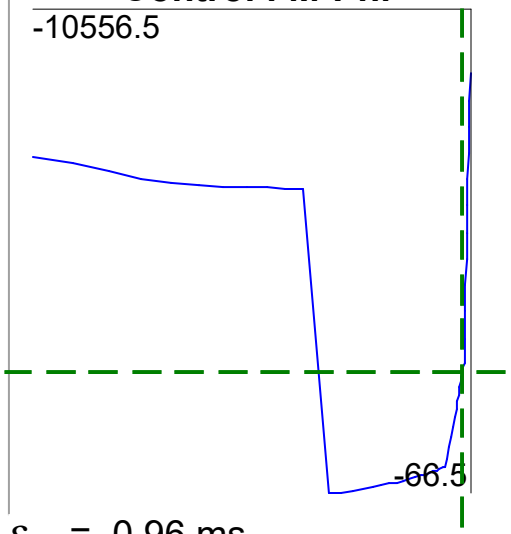
WSpan Bottom Diaphragm

2021/3/26 - 11:18 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.96 ms

ϕ = -199.30 rad/10^6 in

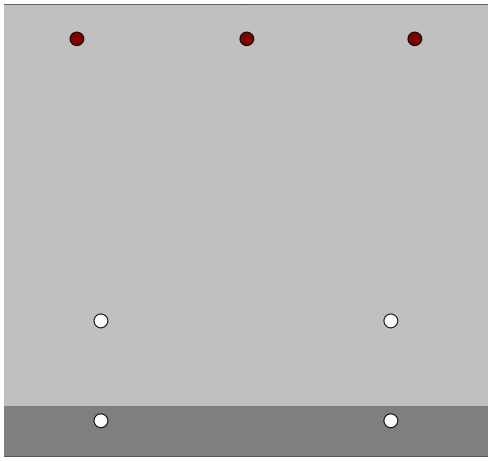
γ_{xy}(avg) = 0.00 ms

Axial Load = -5.1 kips

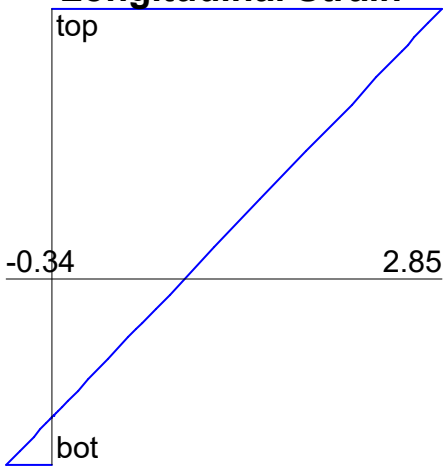
Moment:= -49.9 ft-kips

Shear = 0.0 kips

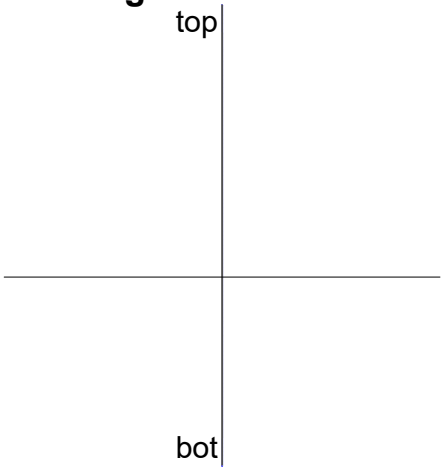
Cross Section



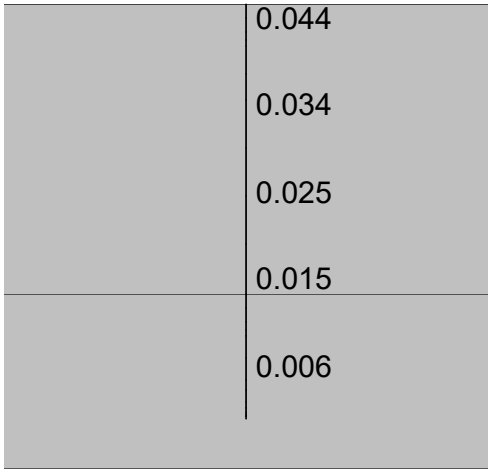
Longitudinal Strain



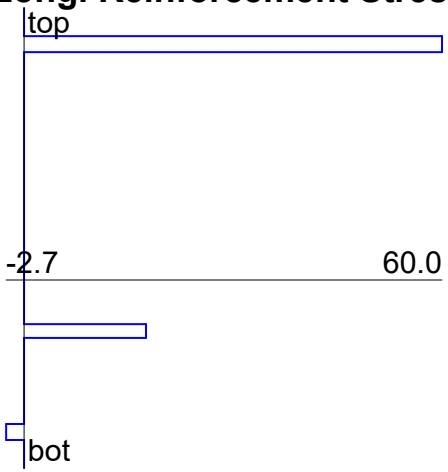
Shrinkage & Thermal Strain



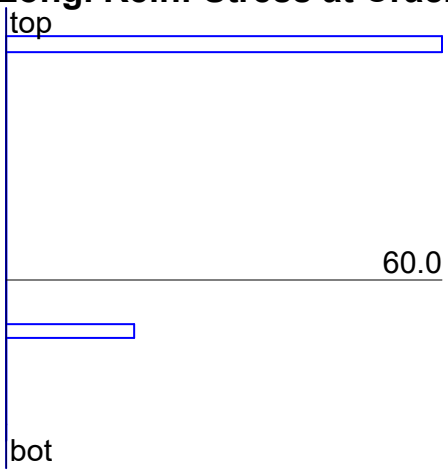
Crack Diagram



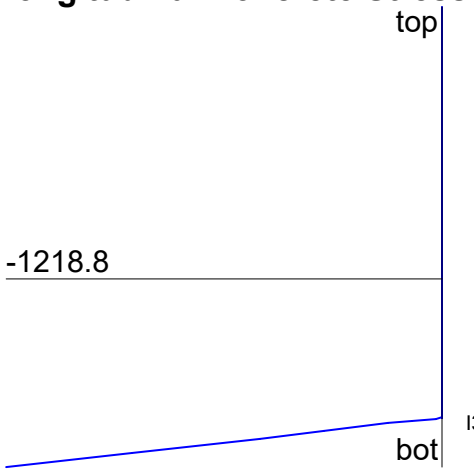
Long. Reinforcement Stress



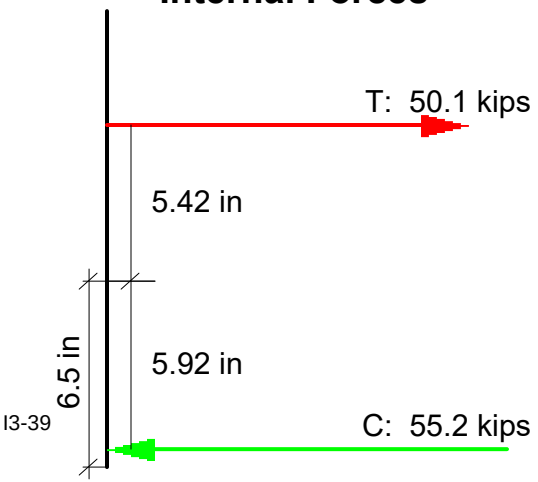
Long. Reinf Stress at Crack



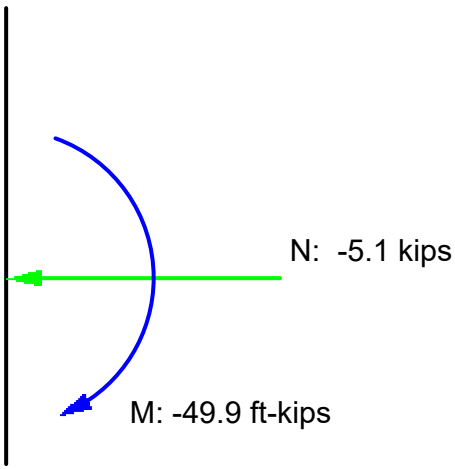
Longitudinal Concrete Stress



Internal Forces



N+M

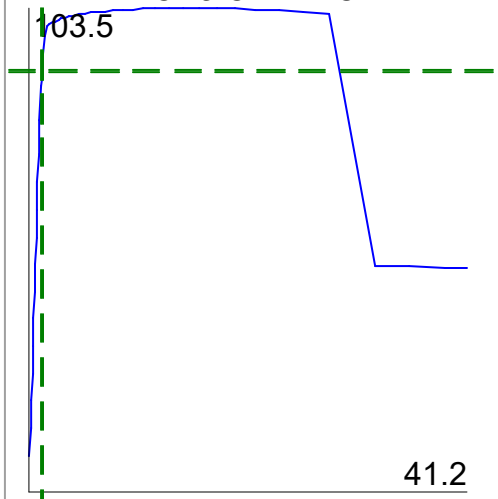


Response-2000 v 1.0.5

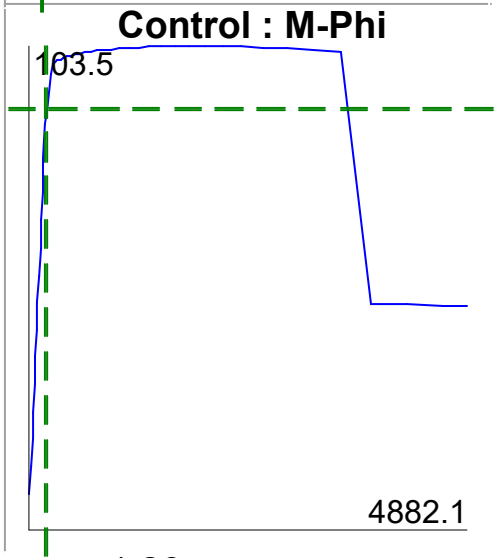
WSpan Bottom Diaphragm

2021/3/26 - 11:14 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.26 ms

ϕ = 199.09 rad/10^6 in

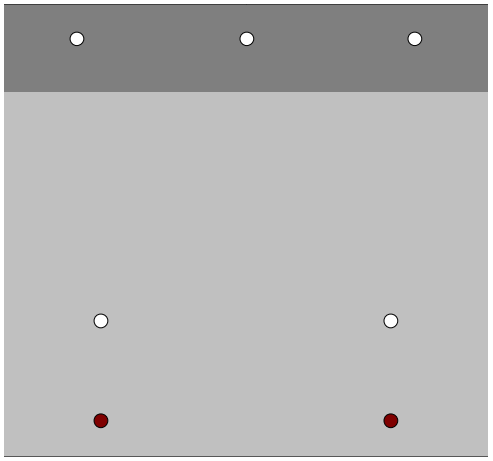
γ_{xy}(avg) = 0.00 ms

Axial Load = -5.1 kips

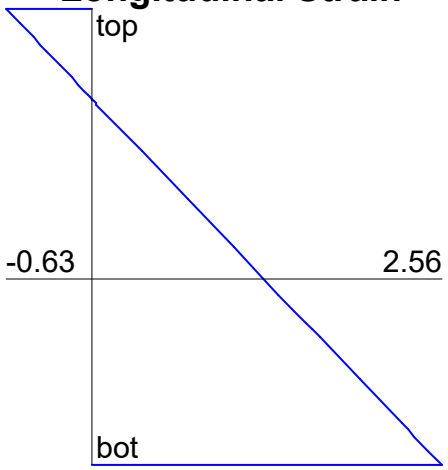
Moment:= 90.0 ft-kips

Shear = 0.0 kips

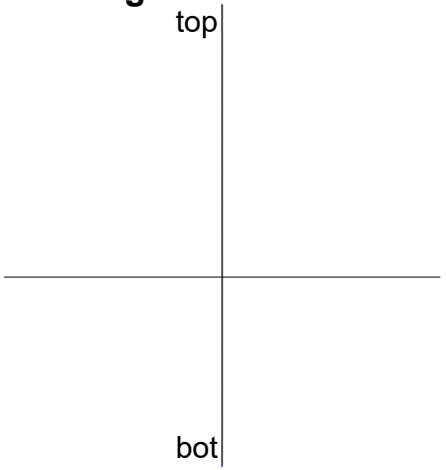
Cross Section



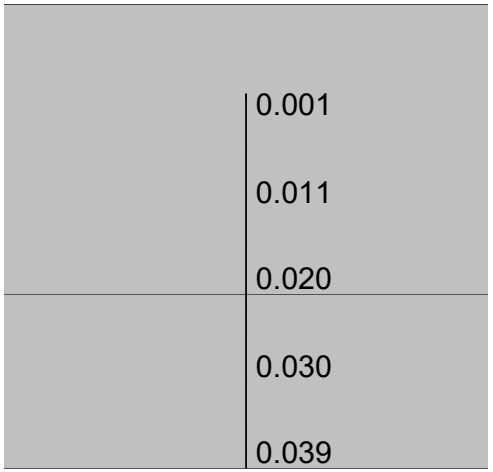
Longitudinal Strain



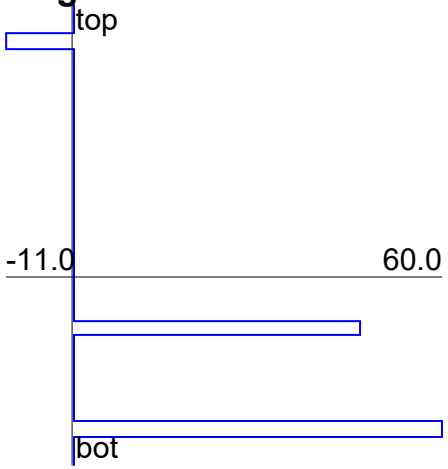
Shrinkage & Thermal Strain



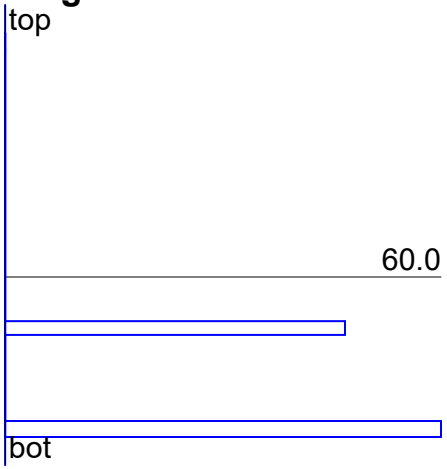
Crack Diagram



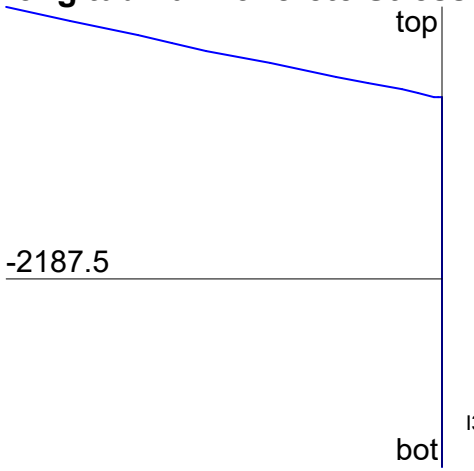
Long. Reinforcement Stress



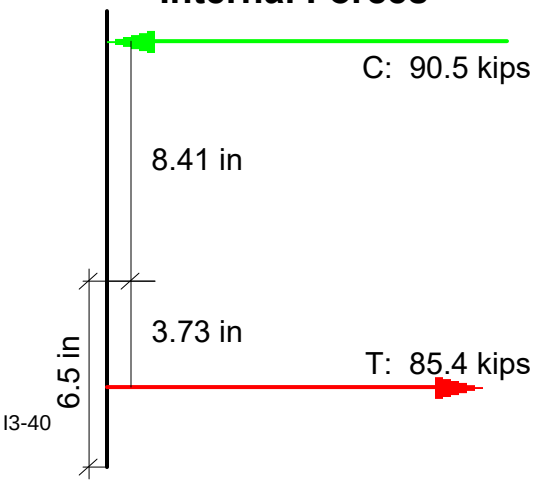
Long. Reinf Stress at Crack



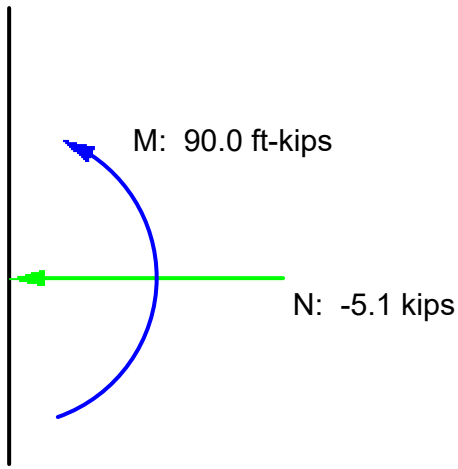
Longitudinal Concrete Stress



Internal Forces

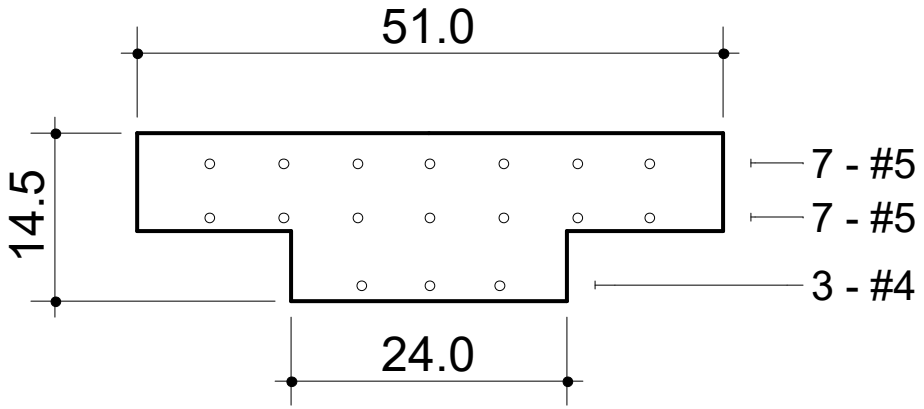


N+M



Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	577.5	612.3
Inertia (in^4)	8723.7	9141.8
y_t (in)	6.1	6.0
y_b (in)	8.4	8.5
S_t (in^3)	1440.1	1511.2
S_b (in^3)	1033.3	1081.8



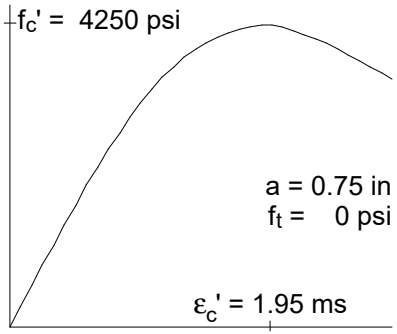
Crack Spacing

2 x dist + 0.1 d_b /ρ

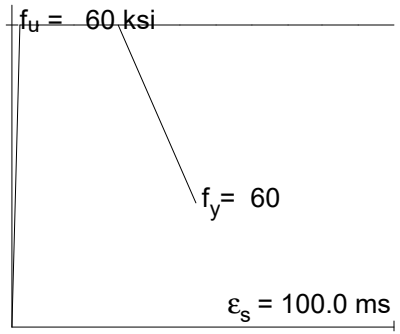
Loading (N,M,V + dN,dM,dV)

0.0 , -0.0 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.05 in

WSpan Top Diaphragm

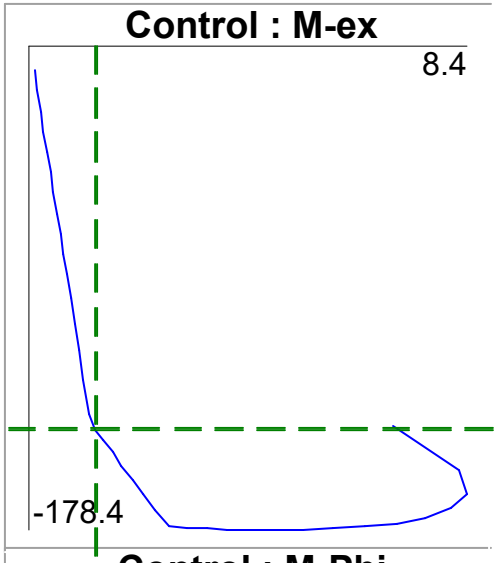
2021/3/25

Response-2000 v 1.0.5

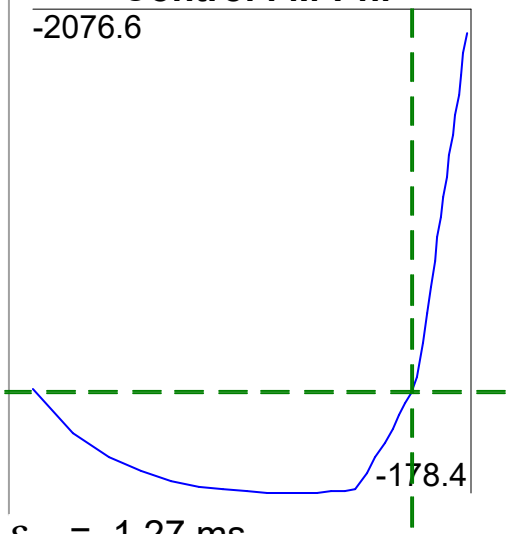
WSpan Top Diaphragm

2021/3/26 - 11:05 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.27 ms

ϕ = -280.61 rad/10^6 in

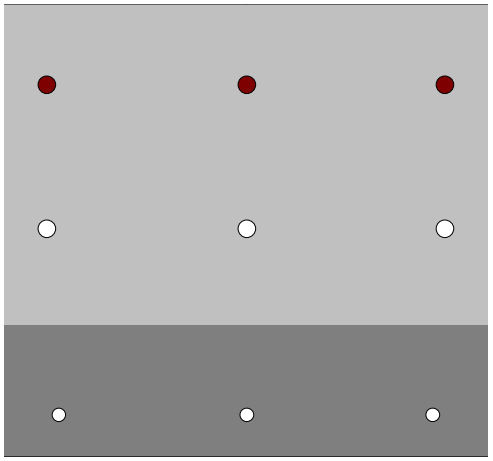
γ_{xy}(avg) = 0.00 ms

Axial Load = 5.2 kips

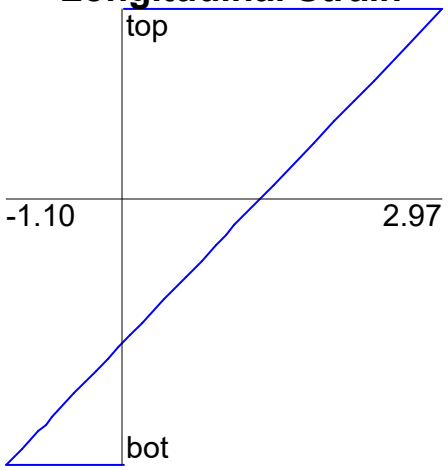
Moment:= -141.3 ft-kips

Shear = 0.0 kips

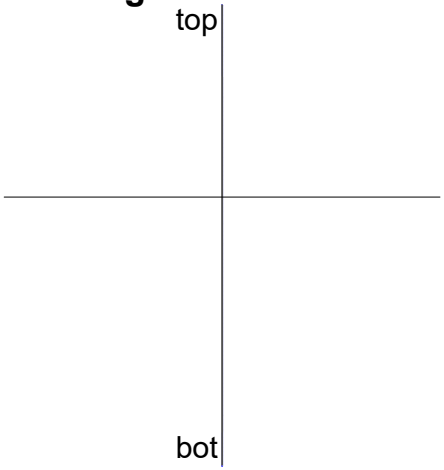
Cross Section



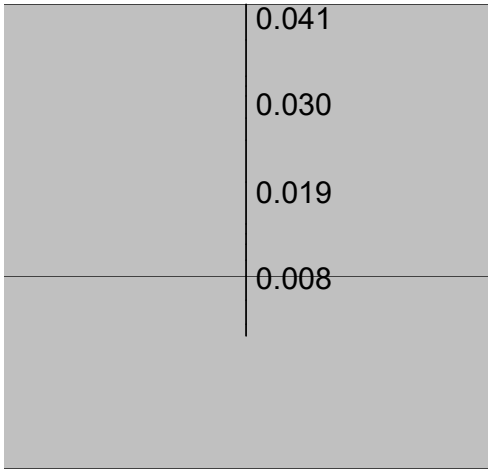
Longitudinal Strain



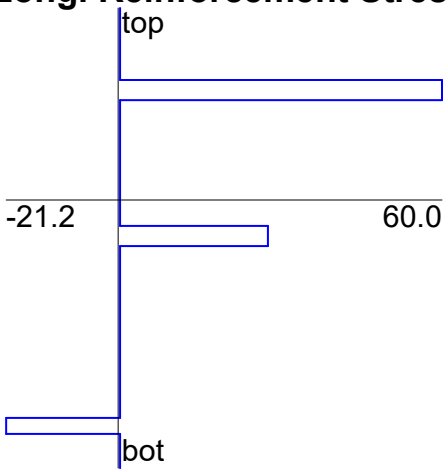
Shrinkage & Thermal Strain



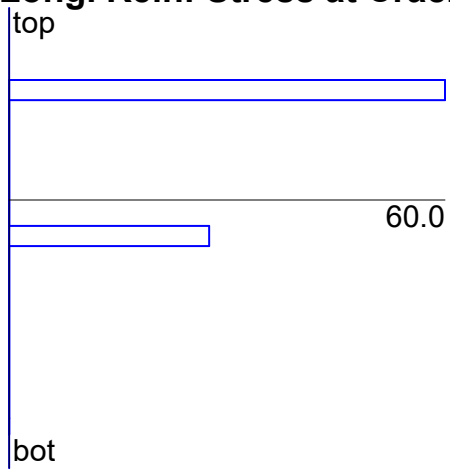
Crack Diagram



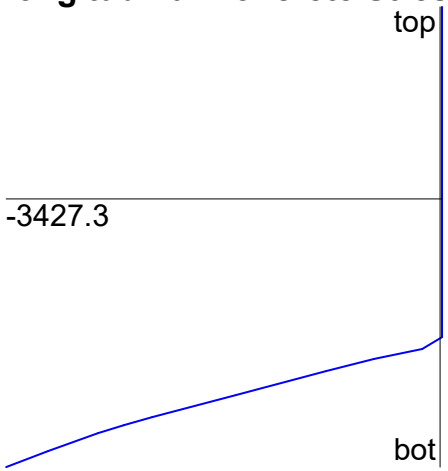
Long. Reinforcement Stress



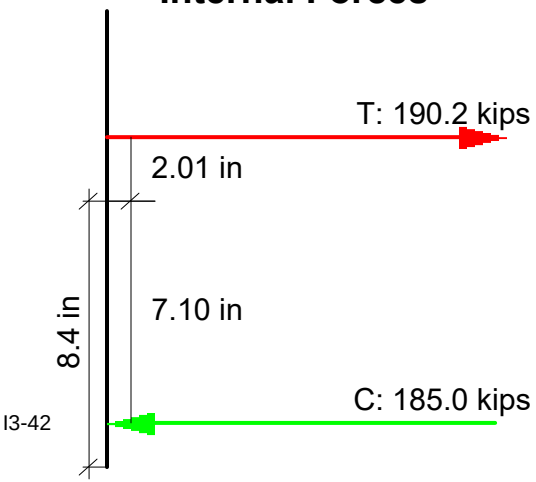
Long. Reinf Stress at Crack



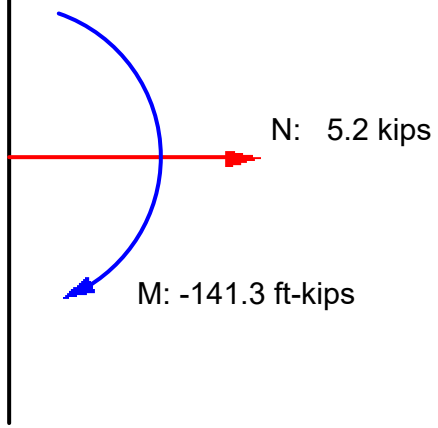
Longitudinal Concrete Stress



Internal Forces



N+M

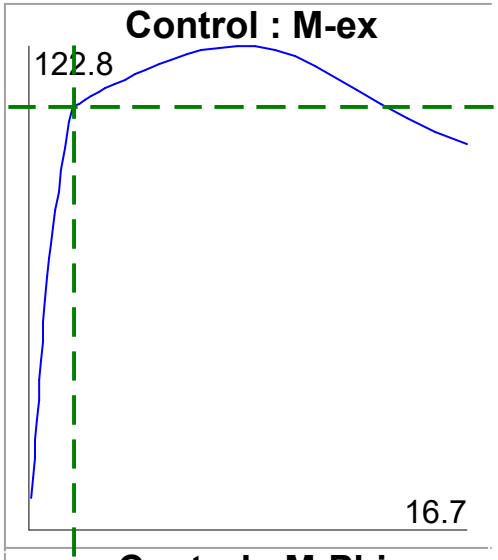


Response-2000 v 1.0.5

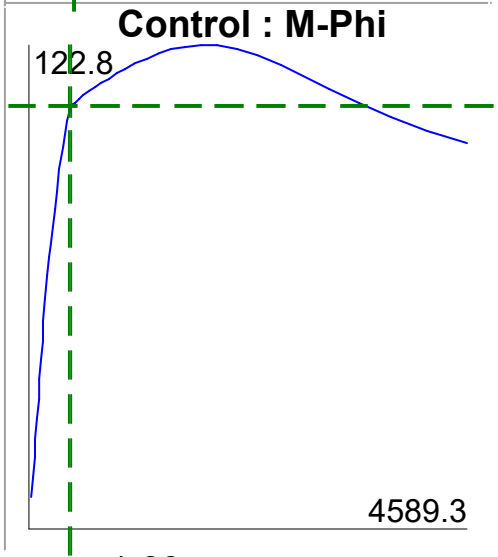
WSpan Top Diaphragm

2021/3/26 - 11:04 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.69 ms

ϕ = 430.69 rad/10^6 in

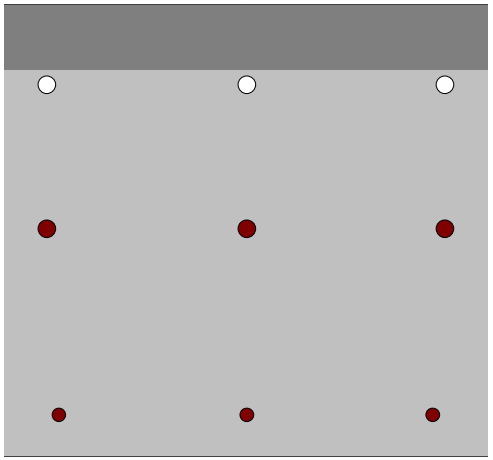
γ_{xy}(avg) = 0.00 ms

Axial Load = 5.1 kips

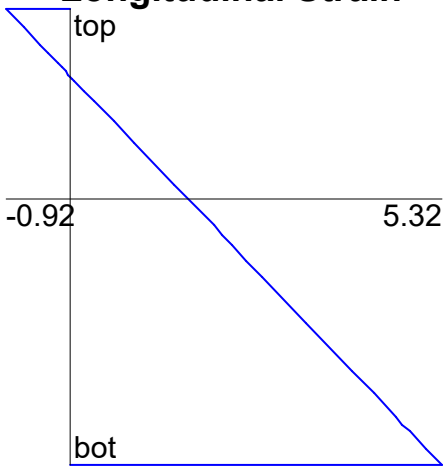
Moment:= 107.3 ft-kips

Shear = 0.0 kips

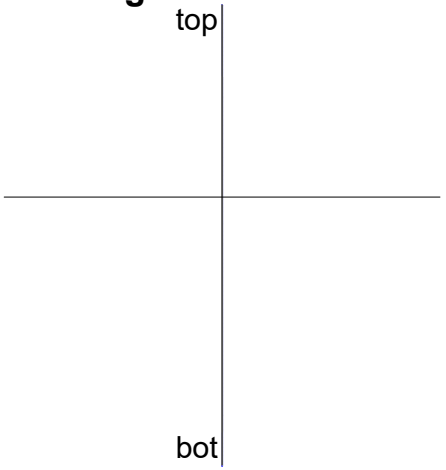
Cross Section



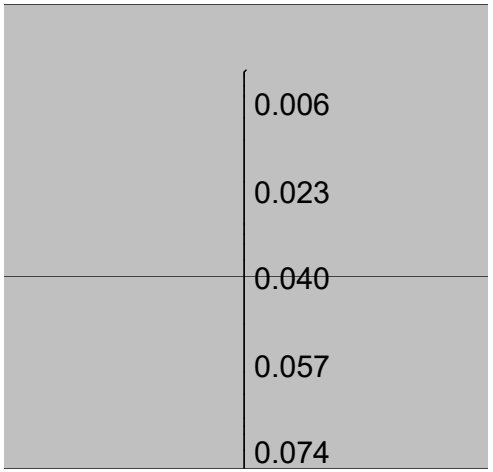
Longitudinal Strain



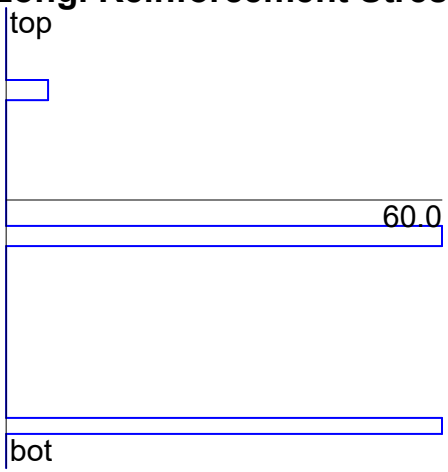
Shrinkage & Thermal Strain



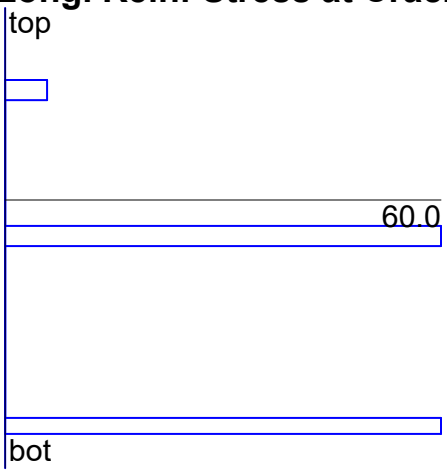
Crack Diagram



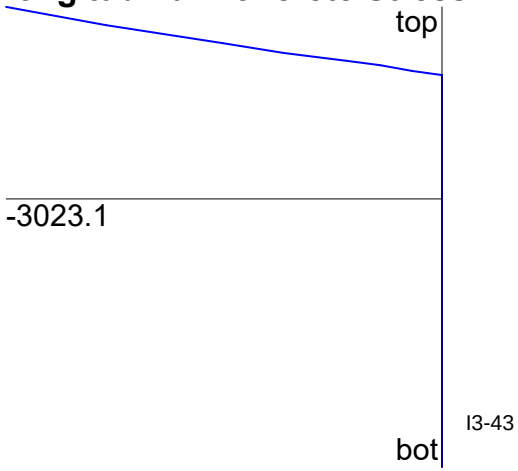
Long. Reinforcement Stress



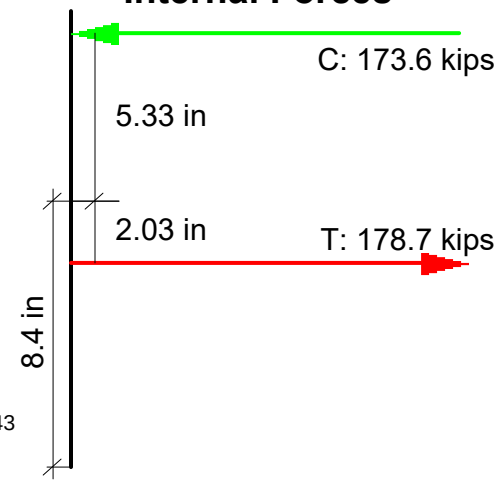
Long. Reinf Stress at Crack



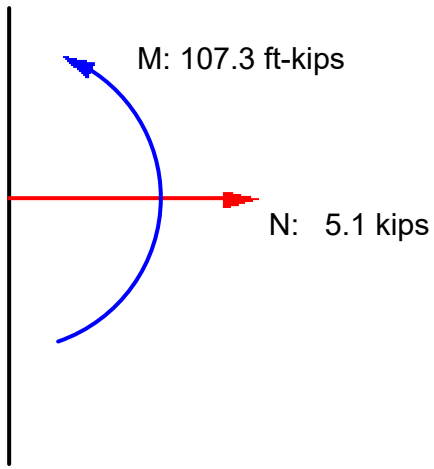
Longitudinal Concrete Stress



Internal Forces



N+M

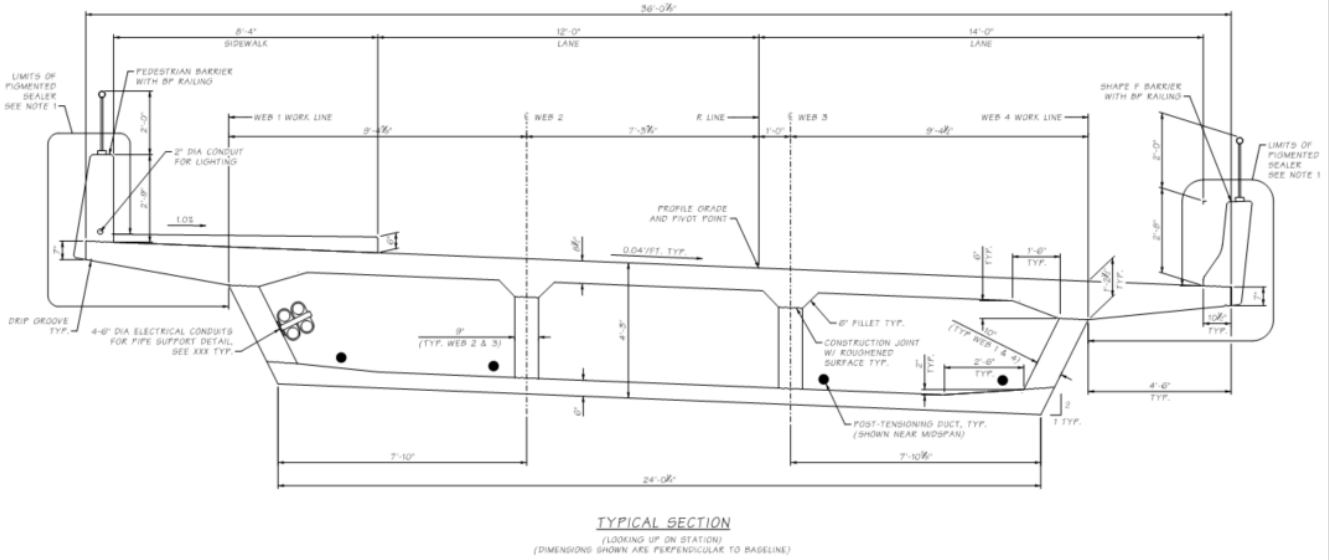


Deviator Diaphragm 2 PT-1 Deviation Force and Capacity Check

1. Purpose

1. The purpose is to calculate the force effects and calculate the capacity in the top and bottom slabs at the deviator diaphragm 2 when there is a utility opening on the diaphragm.

2. Bridge Typical Cross-Section



Note:
1. The cross-section super-elevation is assumed to be zero.
2. All the webs are assumed to be vertical. The thickness of the webs are adjusted such that they have the same area as shown in the actual cross-section.
3. The thicknesses of the top and bottom slab are assumed to be uniform. In other word, fillets and tapered overhang are ignored, but their area are accounted for in the calculation of slab areas and equivalent slab thicknesses.

Deck top slab width	$B_{top} := 36\ ft + 0.875\ in = 36.073\ ft$
Deck bottom slab width	$B_{bottom} := 24\ ft + 0.25\ in = 24.021\ ft$
Overhang flange width	$B_{overhang} := 4.5\ ft$
Deck total height	$H_{total} := 4.25\ ft$
Top slab area	$A_{top} := 30.1\ ft^2$
Bottom slab area	$A_{bottom} := 13.5\ ft^2$
Exterior web area	$A_{extweb} := 2.2\ ft^2$
Interior web area	$A_{intweb} := 1.9\ ft^2$
Top slab thickness	$t_{topslab} := \frac{A_{top}}{B_{top}} = 10.013\ in$
Bottom slab thickness	$t_{botslab} := \frac{A_{bottom}}{B_{bottom}} = 6.744\ in$
Web height	$H_{web} := H_{total} - t_{topslab} - t_{botslab} = 2.854\ ft$
Exterior web thickness	$t_{extweb} := \frac{A_{extweb}}{H_{web}} = 9.252\ in$

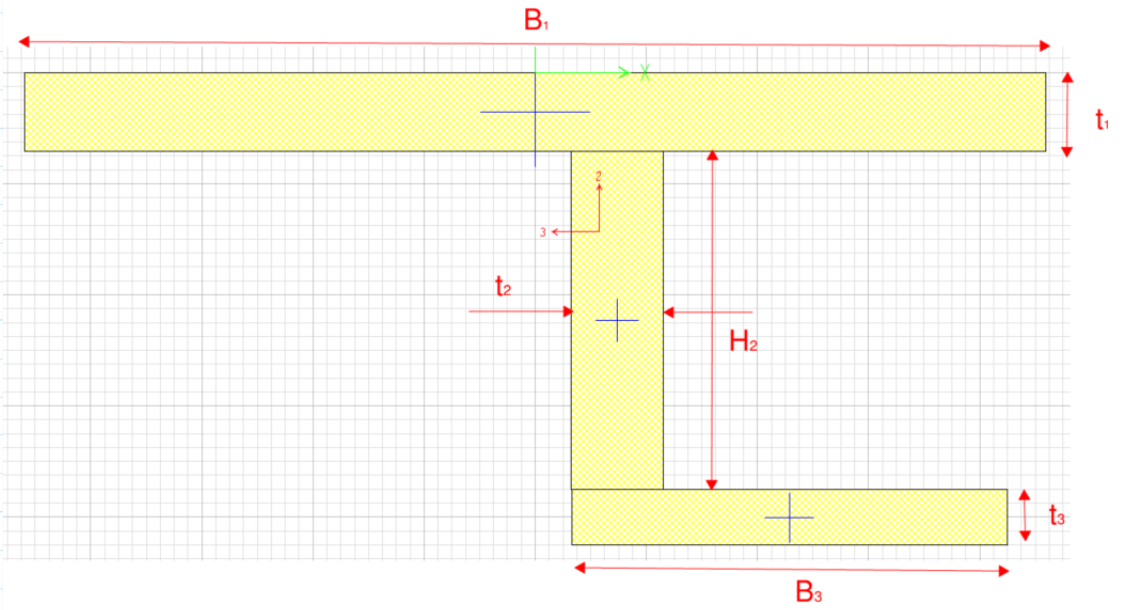
Interior web thickness $t_{intweb} := \frac{A_{intweb}}{H_{web}} = 7.99\ in$

3. Moment of Inertia of Individual Web

The moment of inertia of individual web about the horizontal axis is determined using tributary flange width of each web.

Top width of left cell $B_{c1t} := 9\ ft + 4.625\ in = 9.385\ ft$
Top width of mid cell $B_{c2t} := 8\ ft + 3.75\ in = 8.313\ ft$
Top width of right cell $B_{c3t} := 9\ ft + 4.5\ in = 9.375\ ft$
Bottom width of left cell $B_{c1b} := 7\ ft + 10\ in = 7.833\ ft$
Bottom width of mid cell $B_{c3b} := 7\ ft + 10.625\ in = 7.885\ ft$
Bottom width of right cell $B_{c2b} := B_{bottom} - B_{c1b} - B_{c3b} = 8.302\ ft$

3.1 Moment of Inertia of Exterior Webs



For left exterior web:

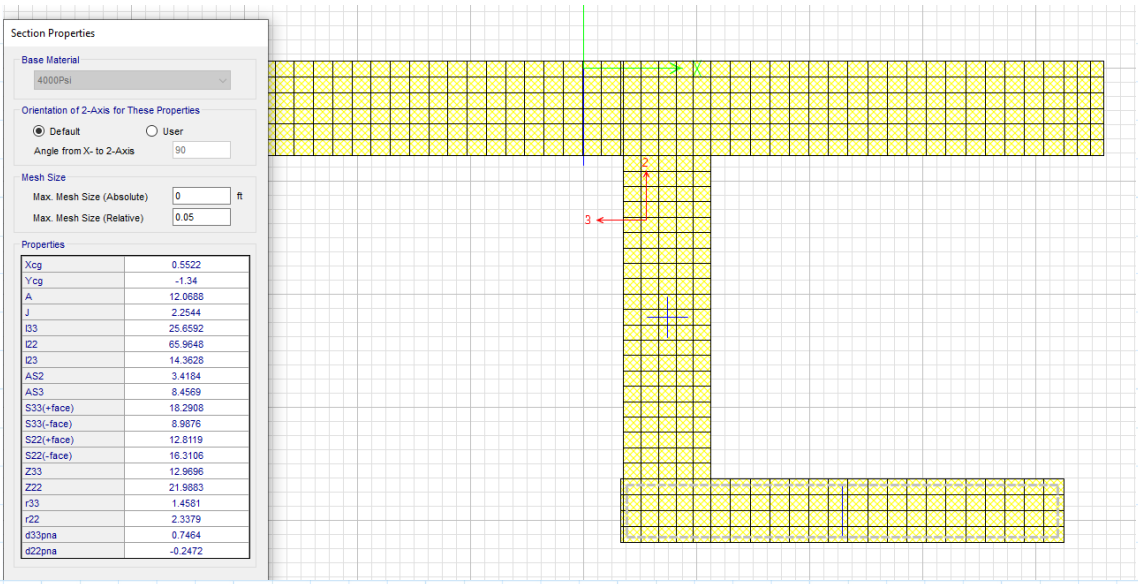
$B_1 := B_{overhang} + \frac{B_{c1t}}{2} = 9.193\ ft$ $t_1 := t_{topslab} = 10.013\ in$

$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.854\ ft$ $t_2 := t_{extweb} = 9.252\ in$

$B_3 := \frac{B_{c1b}}{2} = 3.917\ ft$ $t_3 := t_{botslab} = 6.744\ in$

The moment of inertia about horizontal axis is estimated using csibridge cross-section designer:

$$I_{web1} := 25.65\ ft^4$$



For right exterior web:

$$B_1 := B_{overhang} + \frac{B_{c3t}}{2} = 9.188\ ft$$

$$t_1 := t_{topslab} = 10.013\ in$$

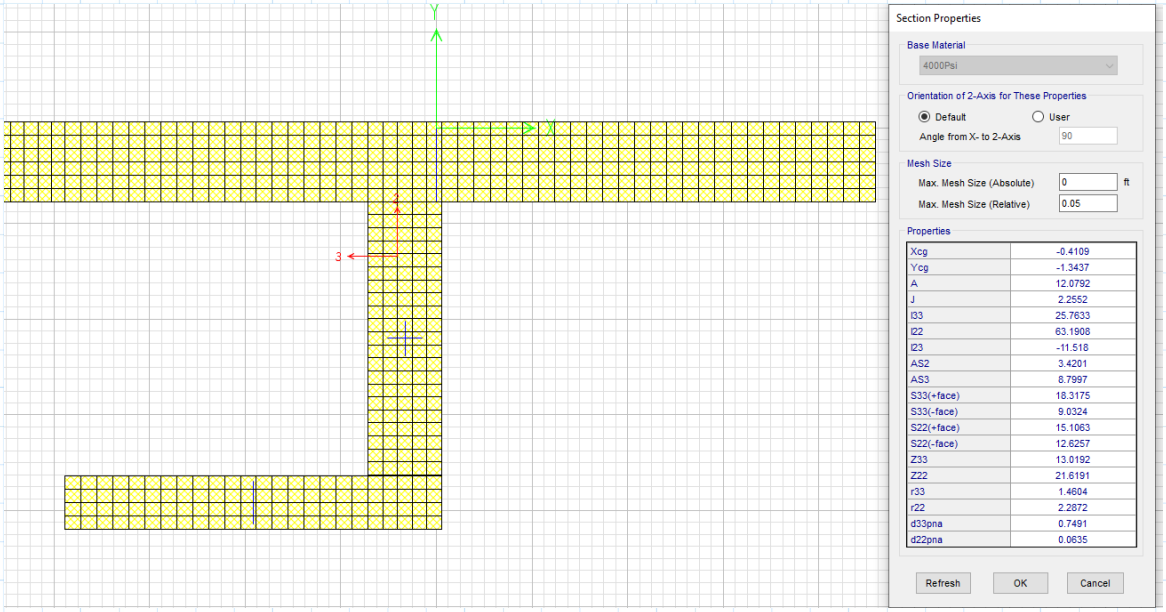
$$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.854\ ft$$

$$t_2 := t_{extweb} = 9.252\ in$$

$$B_3 := \frac{B_{c3b}}{2} = 3.943\ ft$$

$$t_3 := t_{botslab} = 6.744\ in$$

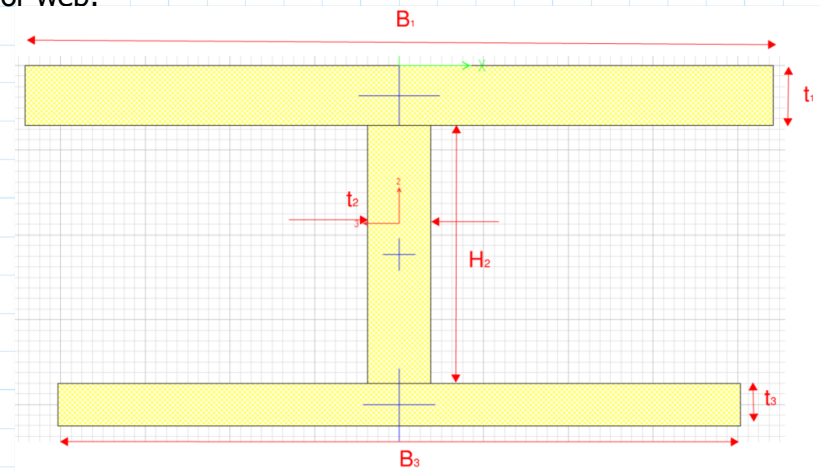
$$I_{web4} := 25.76\ ft^4$$



3.2 Moment of Inertia of Interior Webs

Note: Similar to the estimation of exterior webs, the fillets are ignored and the super-elevation of the cross-section is assumed to be zero

For left interior web:



$$B_1 := \frac{B_{c1t}}{2} + \frac{B_{c2t}}{2} = 8.849\ ft \qquad t_1 := t_{topslab} = 10.013\ in$$

$$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.854\ ft \qquad t_2 := t_{intweb} = 7.99\ in$$

$$B_3 := \frac{B_{c1b}}{2} + \frac{B_{c2b}}{2} = 8.068\ ft \qquad t_3 := t_{botslab} = 6.744\ in$$

$$I_{web2} := 37.824\ ft^4$$

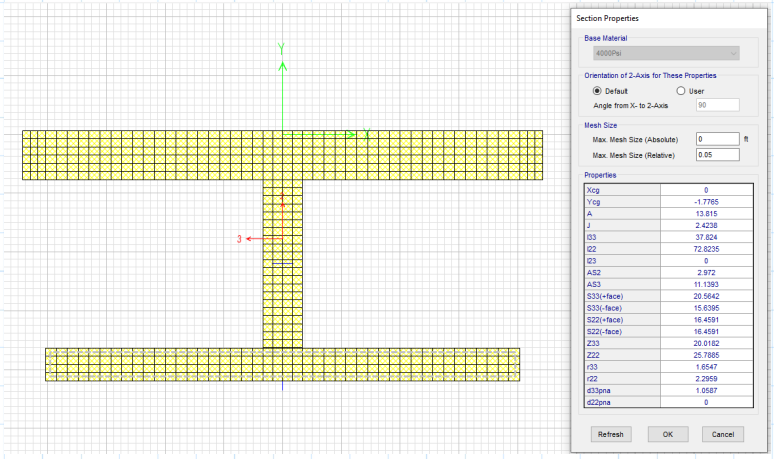
For right interior web:

$$B_1 := \frac{B_{c2t}}{2} + \frac{B_{c3t}}{2} = 8.844\ ft \qquad t_1 := t_{topslab} = 10.013\ in$$

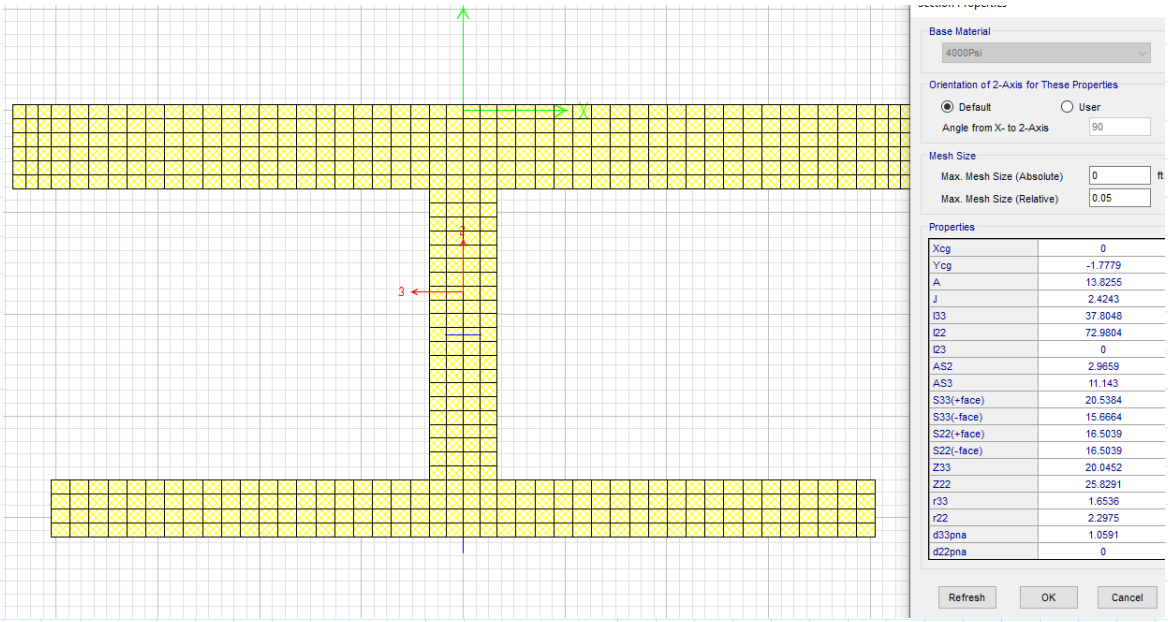
$$H_2 := H_{total} - t_{topslab} - t_{botslab} = 2.854\ ft \qquad t_2 := t_{intweb} = 7.99\ in$$

$$B_3 := \frac{B_{c2b}}{2} + \frac{B_{c3b}}{2} = 8.094\ ft \qquad t_3 := t_{botslab} = 6.744\ in$$

$$I_{web3} := 37.804\ ft^4$$



Screenshot of left interior web cross-section properties



Screenshot of right interior web cross-section properties

4. Stiffness of Individual Webs

A 100 kip point load is applied at the deviator and the deflection at the point is:

$$F_{deviation} := 100\ kip$$
$$\Delta := 0.0072\ ft = 0.086\ in$$

Therefore the overall vertical deflection stiffness at the deviator location is:

$$k_{total} := \frac{F_{deviation}}{\Delta} = \left(1.157 \cdot 10^{3}\right)\ \frac{kip}{in}$$

The vertical spring stiffness of each web is determined based on the moment of inertia of each web calculated in Section 3

$$k_{web1} := k_{total} \cdot \frac{I_{web1}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 233.69\ \frac{kip}{in}$$

$$k_{web2} := k_{total} \cdot \frac{I_{web2}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 344.604\ \frac{kip}{in}$$

$$k_{web3} := k_{total} \cdot \frac{I_{web3}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 344.422\ \frac{kip}{in}$$

$$k_{web4} := k_{total} \cdot \frac{I_{web4}}{I_{web1} + I_{web2} + I_{web3} + I_{web4}} = 234.692\ \frac{kip}{in}$$

5. Deviation Forces

Note: The deviation forces are calculated based on the tendon vertical profiles and tendon forces at the strength limit state.

Tendon effective force (from csibridge) $P_e := 221.68\ kip$

Note: this is not updated based on the latest global model, but it's conservative

Tendon area $A_p := 0.217\ in^2 \cdot 7 = 1.519\ in^2$

Tendon effective stress after losses $f_{pe} := \frac{P_e}{A_p} = 145.938\ ksi$

Tendon strength at strength limit state (AASHTO C5.6.3.1.2) $f_{ps} := 15\ ksi + f_{pe} = 160.938\ ksi$

Tendon force at strength limit state $P_{ps} := A_p \cdot f_{ps} = 244.465\ kip$

Each tendon is divided into segments based on the slope. Assuming the origin is at the east end of the bridge at the top of the superstructure, the horizontal (longitudinal) and vertical coordinates of the ends of the tendon segments adjacent to the deviator are:

$x_{tendon1} := 0\ ft$ $y_{tendon1} := 1.646\ ft$

$x_{tendon2} := 41.63\ ft$ $y_{tendon2} := 3.438\ ft$

$x_{tendon3} := 76.63\ ft$ $y_{tendon3} := 1.063\ ft$

The angles between the tendon and the horizontal (longitudinal) axis on the x-y plane are:

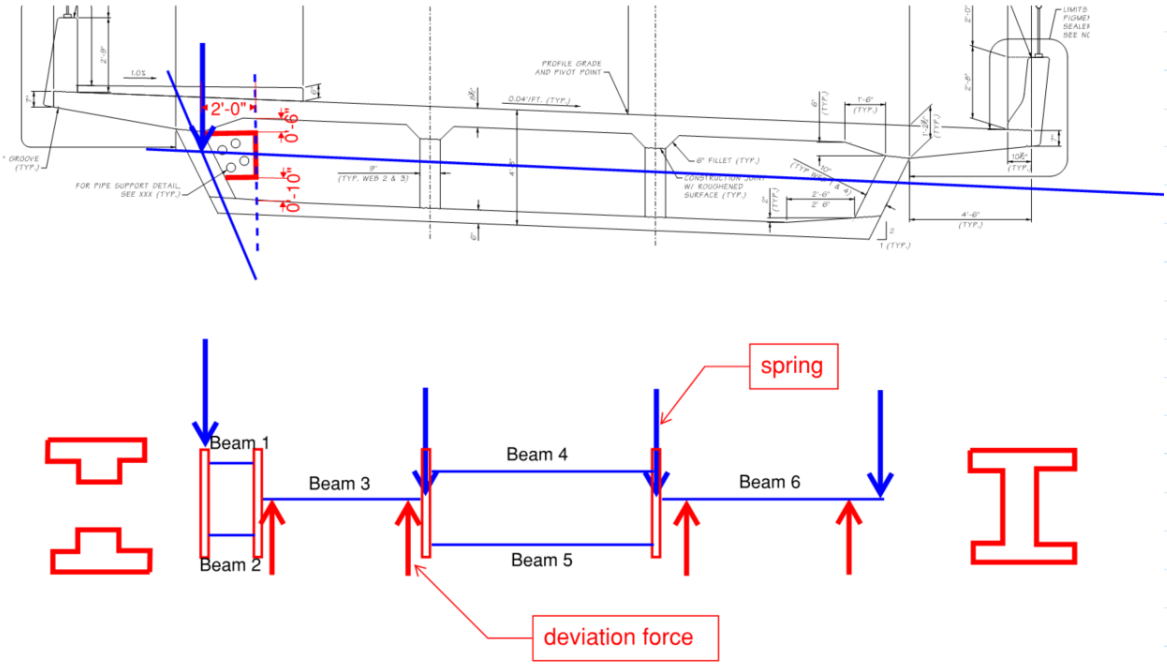
$\alpha_1 := \frac{y_{tendon2} - y_{tendon1}}{x_{tendon2} - x_{tendon1}} = 0.043$ $\operatorname{atan}(\alpha_1) = 2.465\ deg$

$\alpha_2 := \frac{y_{tendon3} - y_{tendon2}}{x_{tendon3} - x_{tendon2}} = -0.068$ $\operatorname{atan}(\alpha_2) = -3.882\ deg$

Therefore, the deviation force at the deviator is:

$$F_{deviation} := P_{ps} \cdot \sin(\operatorname{atan}(\alpha_1)) - P_{ps} \cdot \sin(\operatorname{atan}(\alpha_2)) = 27.064\ kip$$

6. Model Setup



Beam 1 and Beam 2

From csibridge cross-section, the centroid of the T section of beam 1 is at **0.56'** below the top surface of the cross-section and the centroid of the inversed T section of beam 2 is at **0.57'** above the bottom surface of the cross-section. Therefore, the vertical spacing between beams 1 and 2 is:

$$y_{12} := H_{total} - 0.56\ ft - 0.57\ ft = 3.12\ ft$$

The length of beams 1 and 2 is the horizontal offsets between the edge of the opening to the centerline of the exterior web. The offset is measured using bluebeam as:

$$L_{12} := 2\ ft$$

Beam 3 and Beam 6

The cross-section of beams 3 and 6 are in I shape. The flange thickness are consistent the slab widths. The flange widths equal to the height of the box girder cross-section. The width of the web equals the thickness of the diaphragm.

The length of beam 3 is the horizontal distance from the edge of the opening to the centerline of the left interior web:

$$L_3 := B_{c1t} - L_{12} = 7.385\ ft$$

The lengths of 6 are assumed to equal to the top width of the cells:

$$L_6 := B_{c3t} = 9.375\ ft$$

The location of beams 3 and 6 are assumed to be at the centroid I shape, which is at **2.05 ft** below the top of the deck. The location of the centroid if obtained from the csibridge section designer.

Beam 4 and Beam 5

The cross-section of beams 4 and 5 are in rectangular shape with thickness equal to the thickness of the slab and width equal to the height of the box-girder.

The lengths of beams 4 and 5 are assumed to equal to the top width of the cells:

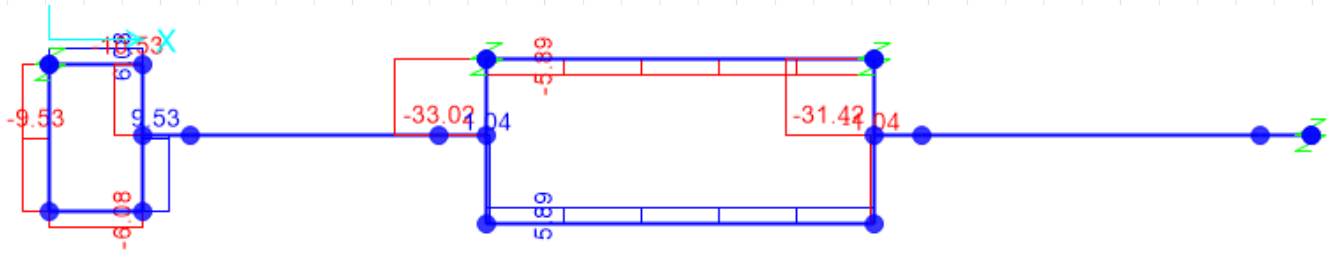
$$L_4 := B_{c2t} = 8.313\ ft$$
$$L_5 := B_{c2t} = 8.313\ ft$$

The locations of beams 4 and 5 are assumed to be at centroid of the slabs, so beam 4 is at **10.01"/2=5.0"=0.42'** below the top of the deck and beam 5 is at **4.25'-6.74"/2=3.97'** below the top of the deck

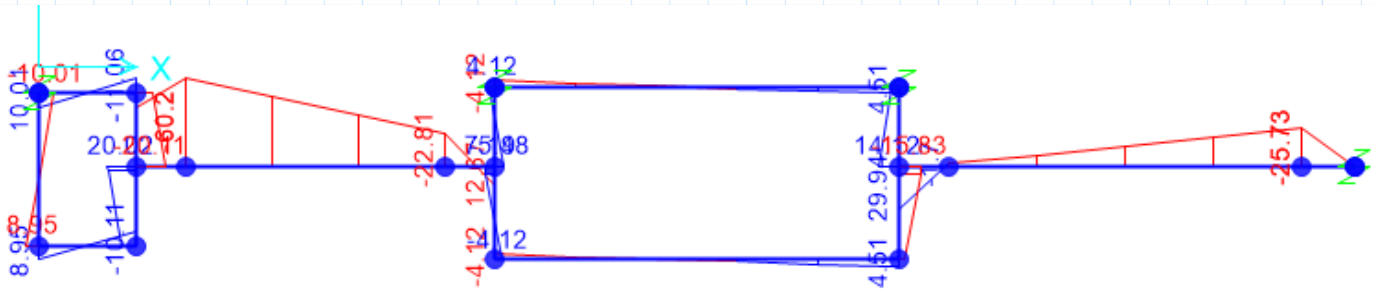
The location of the deviation force are assumed to be 8 in from the adjacent vertical cell surface

7. Results

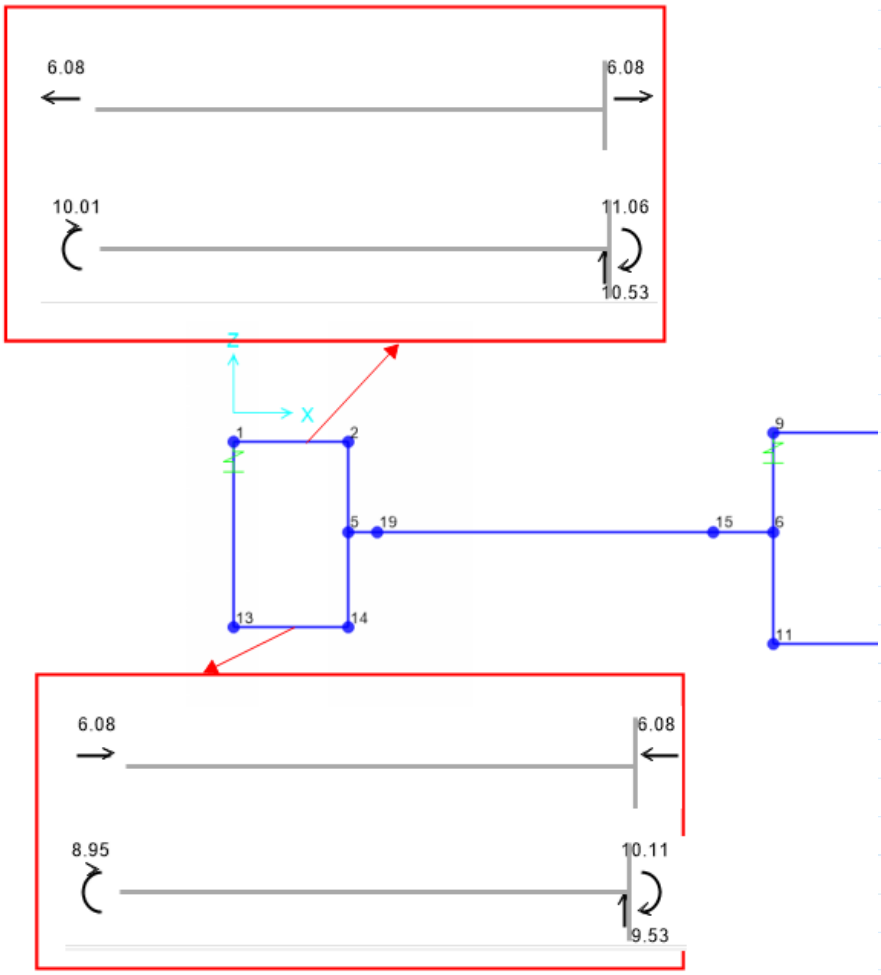
Axial Force Diagram (kip)



Bending Moment Diagram (kip.ft)



Forces on the Diaphragm with openings (kip or kip.ft)



8. Capacity Summary

8.1 Resistance Factor

As per AASHTO 5.5.4.2, the resistance factor for non-prestressed members is:

and for nonprestressed members such that:

$$0.75 \le \phi = 0.75 + \frac{0.15(\varepsilon_t - \varepsilon_{cl})}{(\varepsilon_{tl} - \varepsilon_{cl})} \le 0.9 \qquad (5.5.4.2\text{-}2)$$

Tension controlled strain limit (AASHTO C T5.6.2.1-1)	$\varepsilon_{tl} := 0.005$
Compression controlled strain limit (AASHTO C T5.6.2.1-1)	$\varepsilon_{cl} := 0.002$

Therefore, if the tensile strain in the reinforcement is greater than 0.005, the resistance factor is 0.9.

8.2 Demand and Factored Capacity

Note: the capacities are obtained from RESPONSE 2000

Top Diaphragm

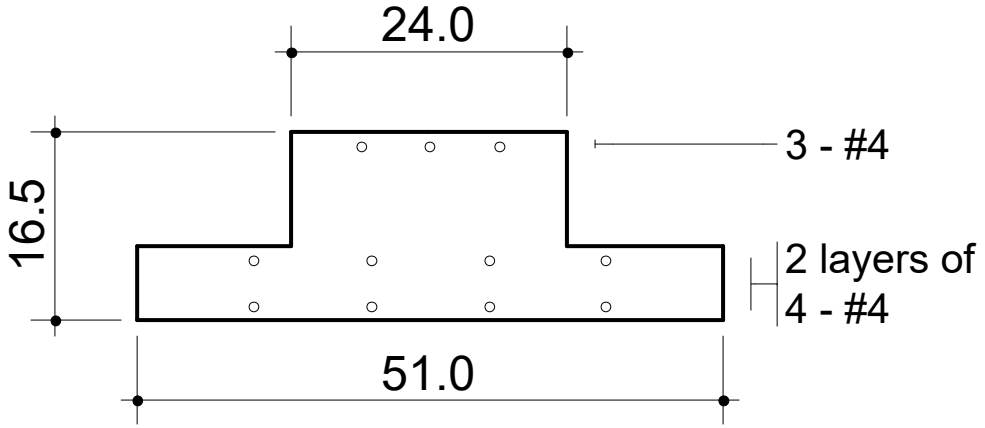
Demand	$N_{tp} := 6.08\ kip$	$M_{tp} := 10.01\ kip \cdot ft$	$N_{tn} := 6.08\ kip$	$M_{tn} := -11.06\ kip \cdot ft$
Unfactored Capacity	$N_{tpc} := 6.08\ kip$	$M_{tpc} := 106.4\ kip \cdot ft$	$N_{tnc} := 6.08\ kip$	$M_{tnc} := -135.7\ kip \cdot ft$
Tensile Strain in Extreme Tension Steel	$\varepsilon_{ttp} := 0.00451$		$\varepsilon_{ttn} := 0.0030$	
Resistance Factor	$\phi_{tp} := 0.75 + \frac{0.15 \cdot (\varepsilon_{ttp} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.876$		$\phi_{pn} := 0.75 + \frac{0.15 \cdot (\varepsilon_{ttn} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.8$	
D/C	$\frac{M_{tp}}{M_{tpc} \cdot \phi_{tp}} = 0.1$		$\frac{M_{tn}}{M_{tnc} \cdot \phi_{pn}} = 0.1$	

Bottom Diaphragm

Demand	$N_{bp} := -6.08\ kip$	$M_{bp} := 8.95\ kip \cdot ft$	$N_{bn} := -6.08\ kip$	$M_{bn} := -10.11\ kip \cdot ft$
Unfactored Capacity	$N_{bpc} := -6.08\ kip$	$M_{bpc} := 102.7\ kip \cdot ft$	$N_{bnc} := -6.08\ kip$	$M_{bnc} := -65.5\ kip \cdot ft$
Tensile Strain in Extreme Tension Steel	$\varepsilon_{tbp} := 0.00324$		$\varepsilon_{tbn} := 0.0078$	
Resistance Factor	$\phi_{bp} := 0.75 + \frac{0.15 \cdot (\varepsilon_{tbp} - \varepsilon_{cl})}{\varepsilon_{tl} - \varepsilon_{cl}} = 0.812$		$\phi_{bn} := 0.9$	
D/C	$\frac{M_{bp}}{M_{bpc} \cdot \phi_{bp}} = 0.1$		$\frac{M_{bn}}{M_{bnc} \cdot \phi_{bn}} = 0.2$	

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	571.5	587.0
Inertia (in^4)	12642.3	13130.4
y_t (in)	9.8	9.8
y_b (in)	6.7	6.7
S_t (in^3)	1292.0	1341.1
S_b (in^3)	1882.8	1957.0



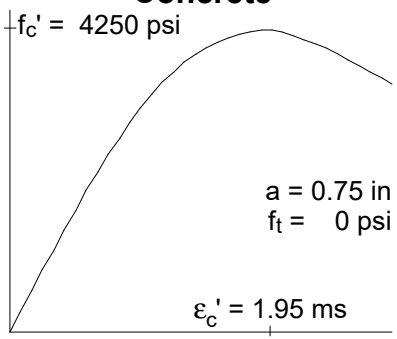
Crack Spacing

2 x dist + 0.1 d_b /ρ

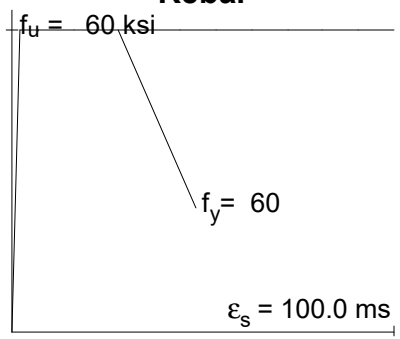
Loading (N,M,V + dN,dM,dV)

-6.08 , -10.11 , 0.0 + 0.0 , -1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.00 in

ESpan Bot Diaphragm

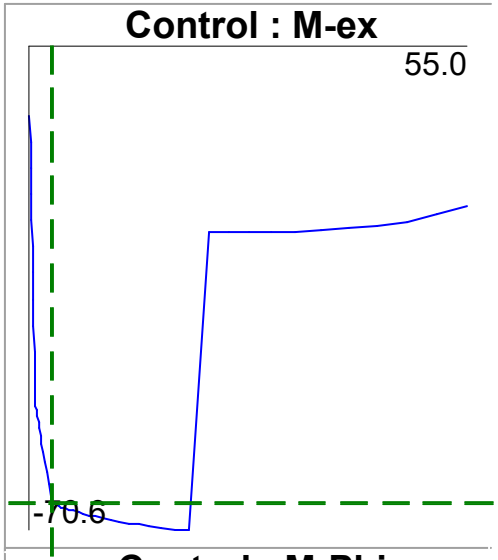
2021/3/24

Response-2000 v 1.0.5

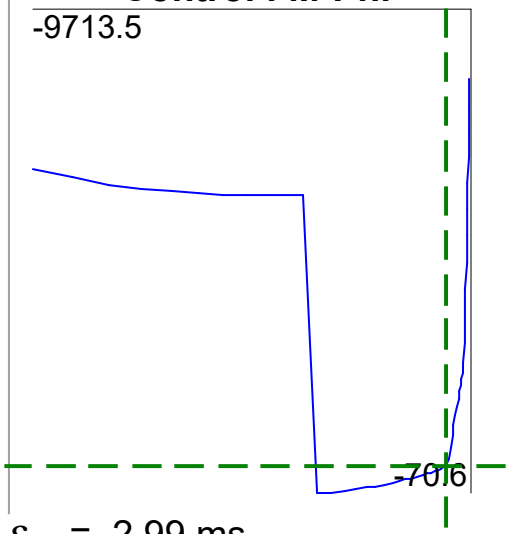
ESpan Bot Diaphragm (N=-6.08 kip&M=8.95kip.ft)

2021/3/26 - 9:08 am

Control : M-ex



Control : M-Phi



ε_{x0} = 2.99 ms

ϕ = -556.67 rad/10^6 in

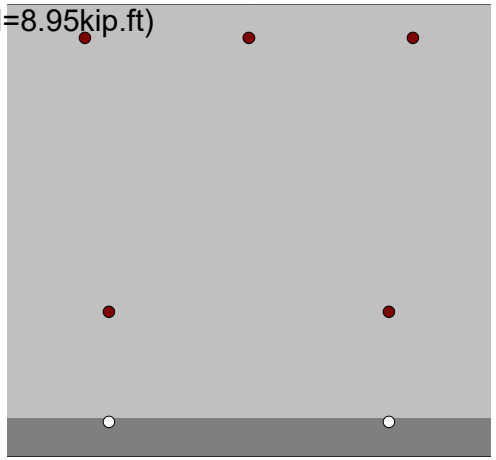
γ_{xy}(avg) = 0.00 ms

Axial Load = -6.1 kips

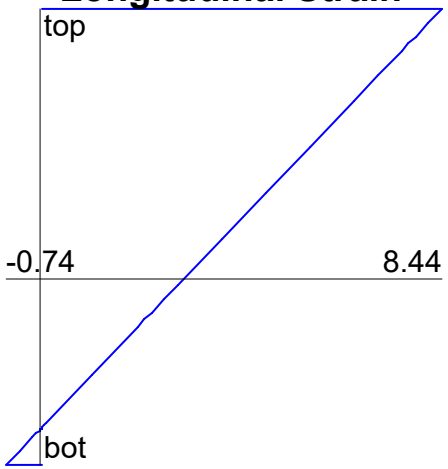
Moment:= -66.7 ft-kips

Shear = 0.0 kips

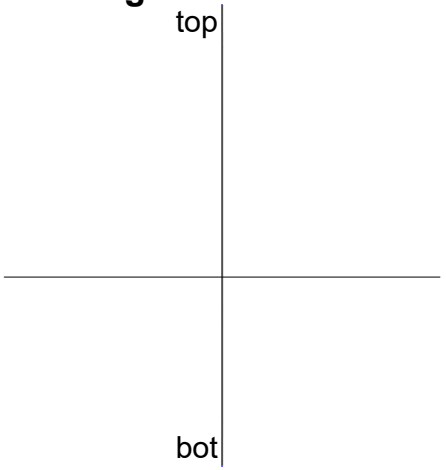
Cross Section



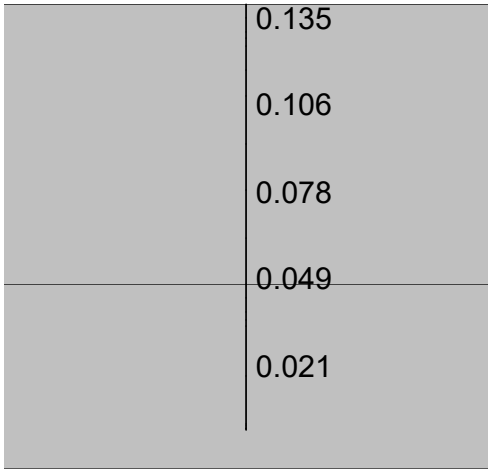
Longitudinal Strain



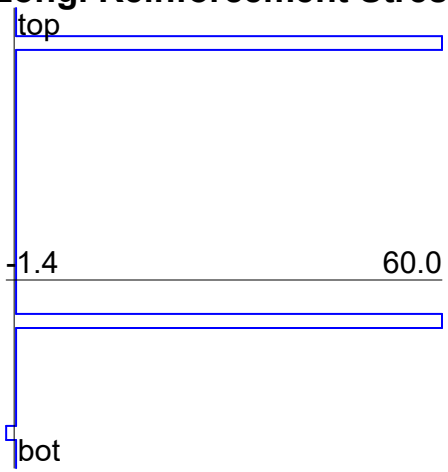
Shrinkage & Thermal Strain



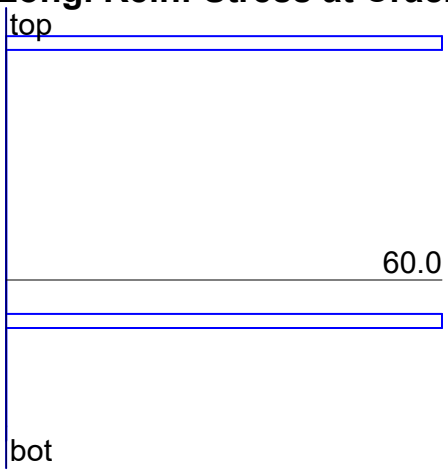
Crack Diagram



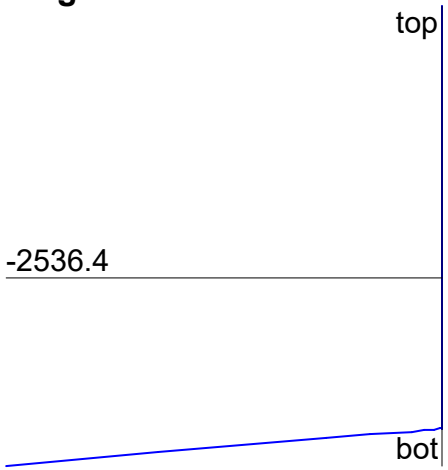
Long. Reinforcement Stress



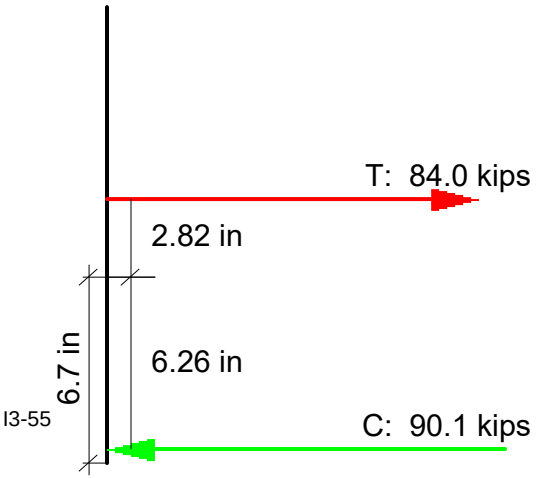
Long. Reinf Stress at Crack



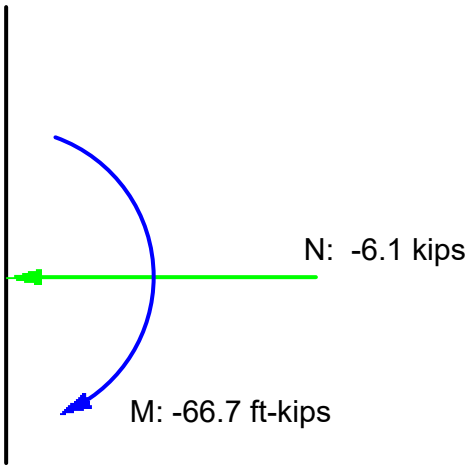
Longitudinal Concrete Stress



Internal Forces



N+M

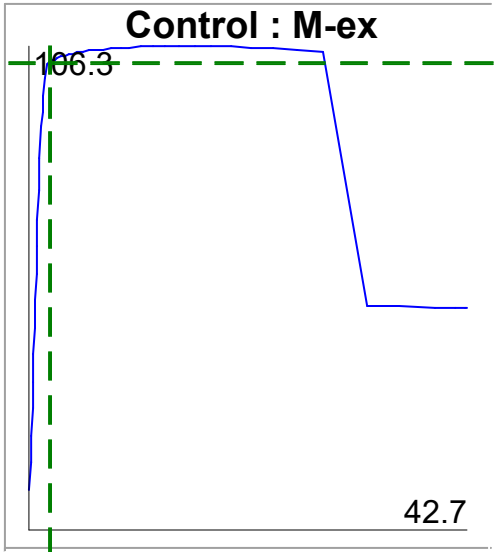


Response-2000 v 1.0.5

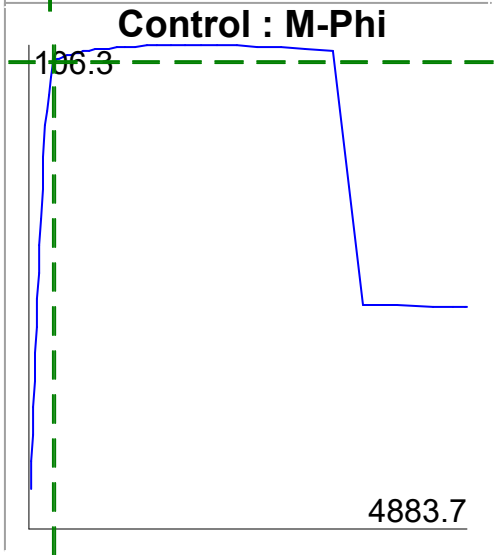
ESpan Bot Diaphragm (N=-6.08 kip&M=8.95kip.ft)

2021/3/26 - 11:22 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.98 ms

ϕ = 284.46 rad/10^6 in

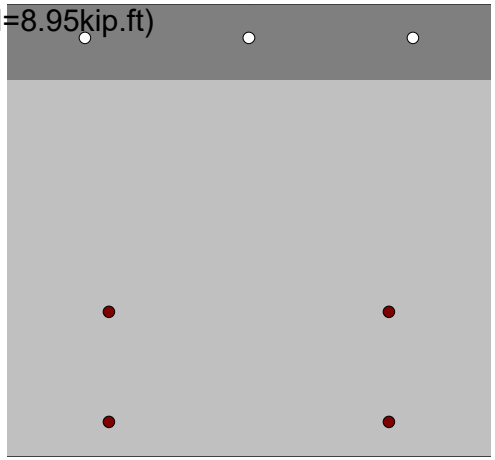
γ_{xy}(avg) = 0.00 ms

Axial Load = -6.1 kips

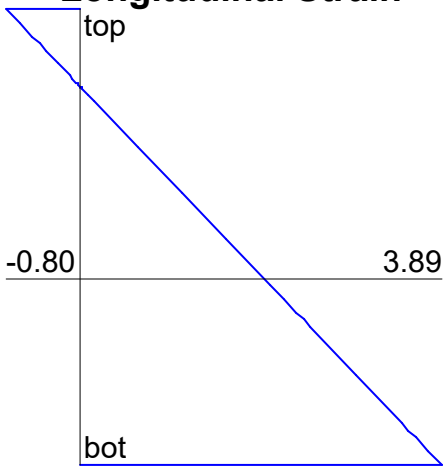
Moment:= 102.7 ft-kips

Shear = 0.0 kips

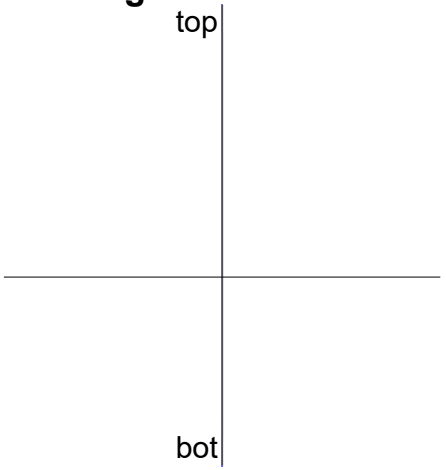
Cross Section



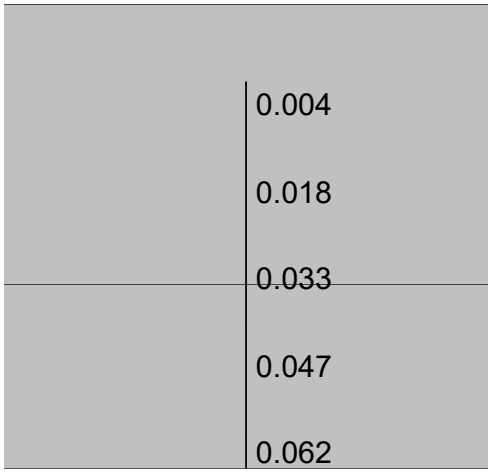
Longitudinal Strain



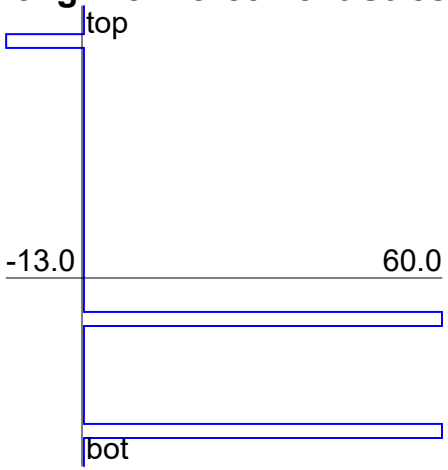
Shrinkage & Thermal Strain



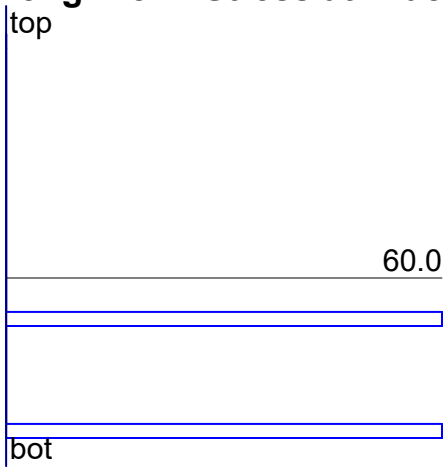
Crack Diagram



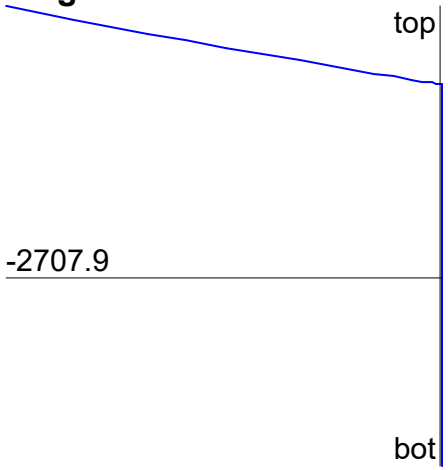
Long. Reinforcement Stress



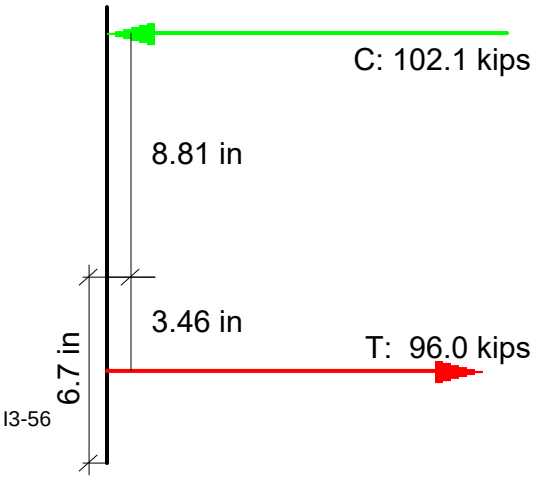
Long. Reinf Stress at Crack



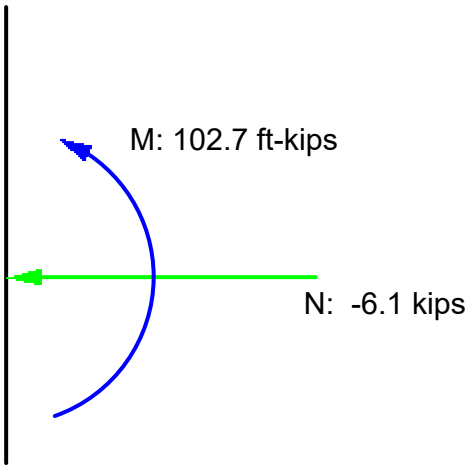
Longitudinal Concrete Stress



Internal Forces

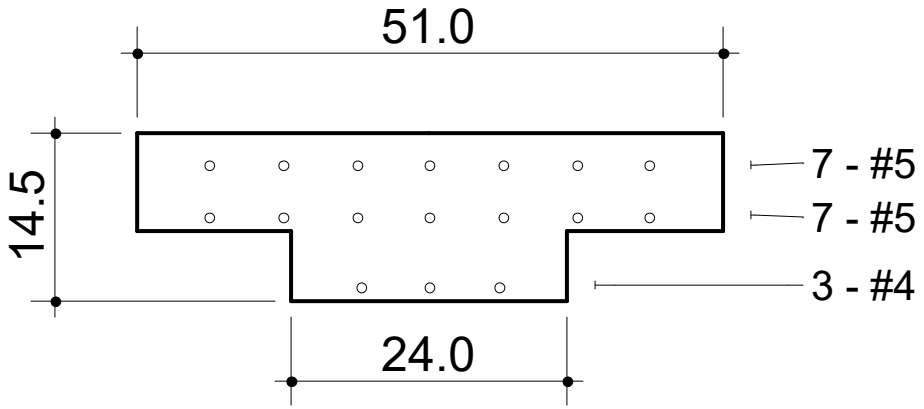


N+M



Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	577.5	612.3
Inertia (in^4)	8723.7	9123.9
y_t (in)	6.1	6.1
y_b (in)	8.4	8.4
S_t (in^3)	1440.1	1507.0
S_b (in^3)	1033.3	1080.3



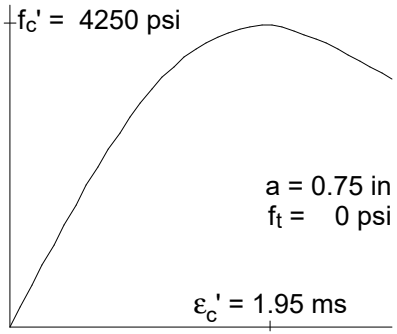
Crack Spacing

2 x dist + 0.1 d_b /ρ

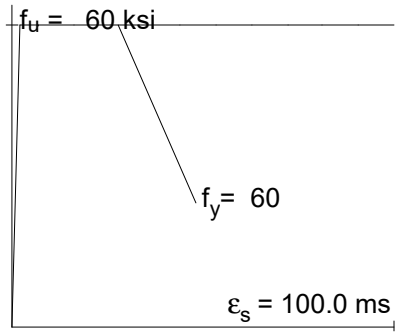
Loading (N,M,V + dN,dM,dV)

6.08 , -10.01 , 0.0 + 0.0 , -1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.00 in

ESpan Top Diaphragm

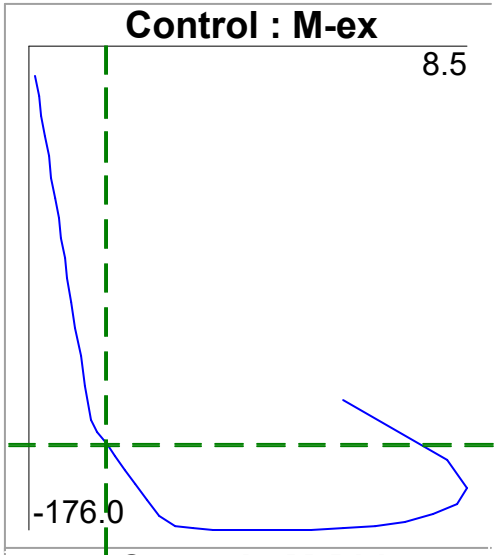
2021/3/26

Response-2000 v 1.0.5

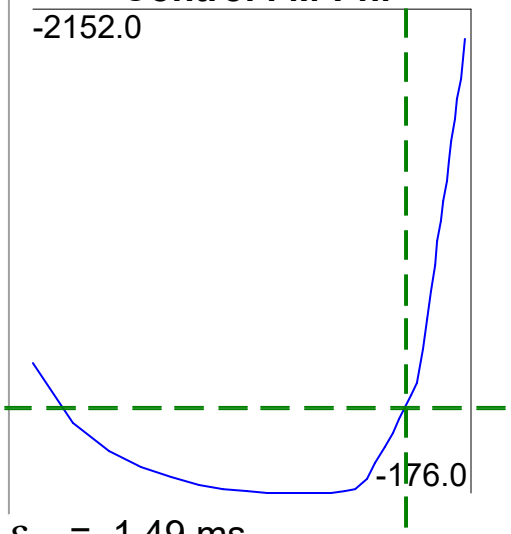
ESpan Top Diaphragm

2021/3/26 - 10:48 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.49 ms

ϕ = -319.88 rad/10^6 in

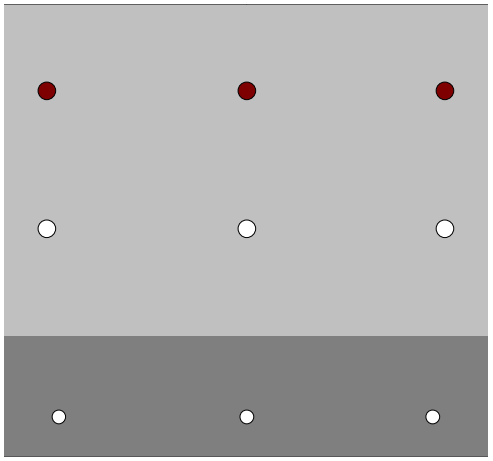
γ_{xy}(avg) = 0.00 ms

Axial Load = 6.0 kips

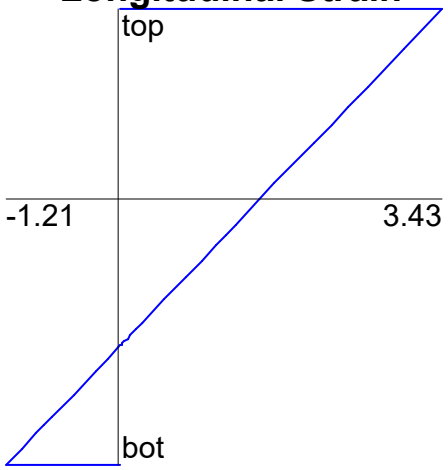
Moment:= -144.8 ft-kips

Shear = 0.0 kips

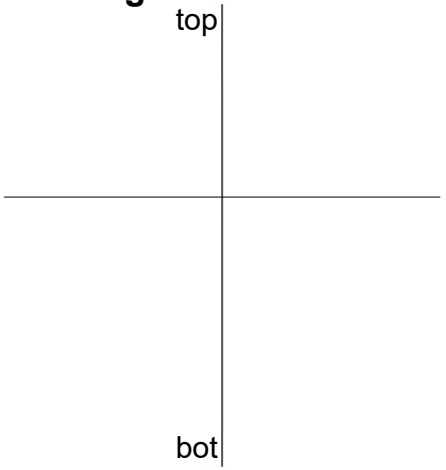
Cross Section



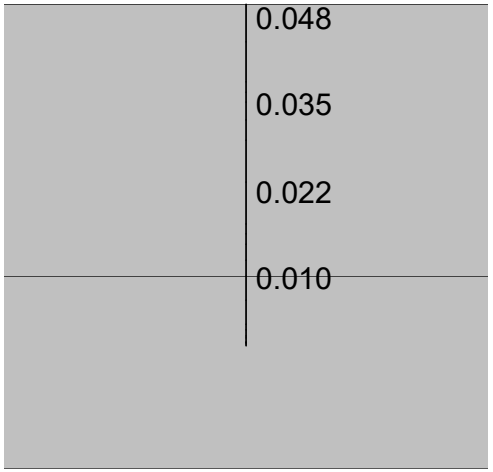
Longitudinal Strain



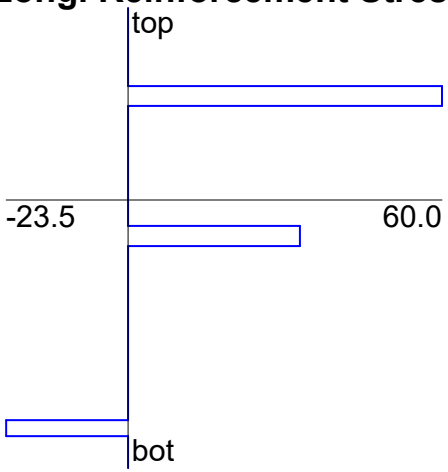
Shrinkage & Thermal Strain



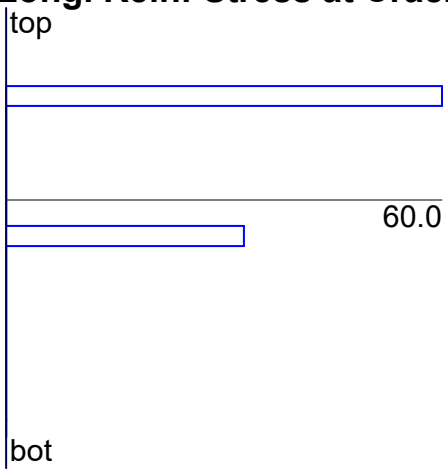
Crack Diagram



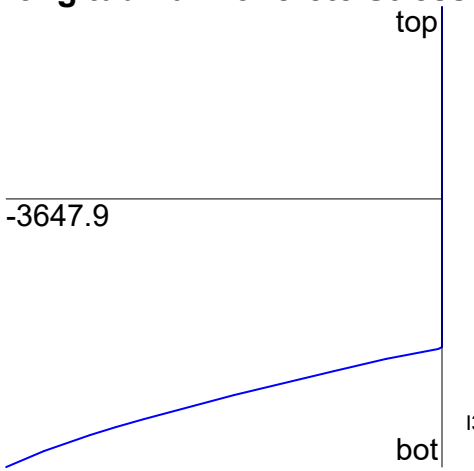
Long. Reinforcement Stress



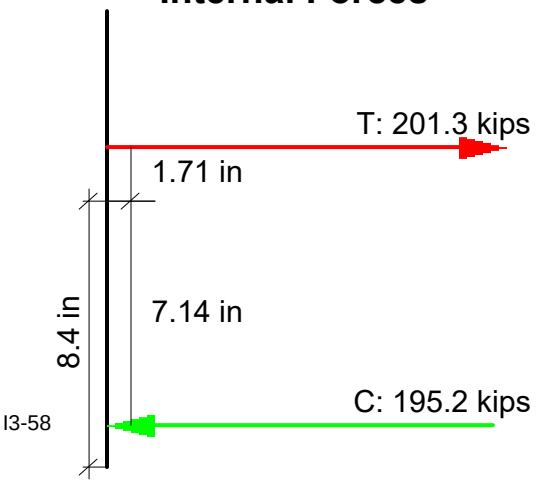
Long. Reinf Stress at Crack



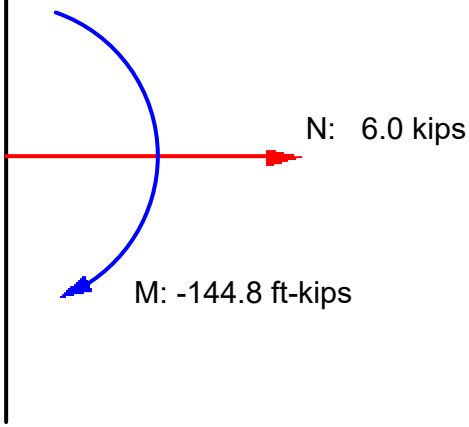
Longitudinal Concrete Stress



Internal Forces



N+M

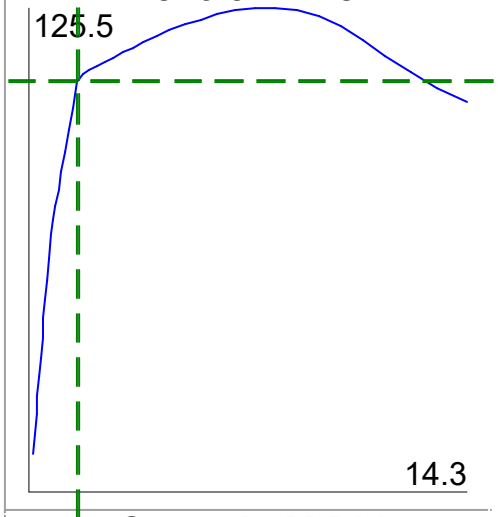


Response-2000 v 1.0.5

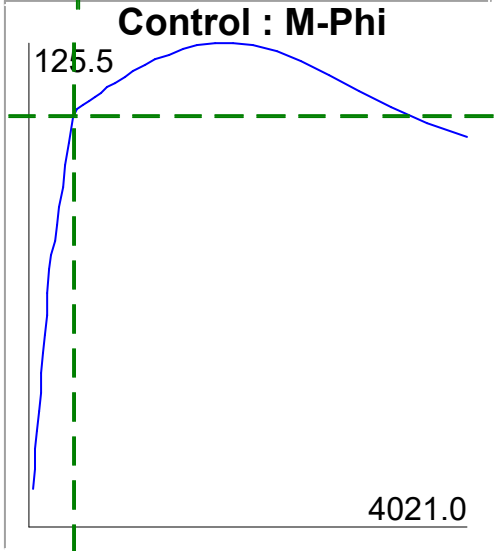
ESpan Top Diaphragm

2021/3/26 - 9:24 am

Control : M-ex



Control : M-Phi



ε_{x0} = 1.58 ms

ϕ = 408.24 rad/10^6 in

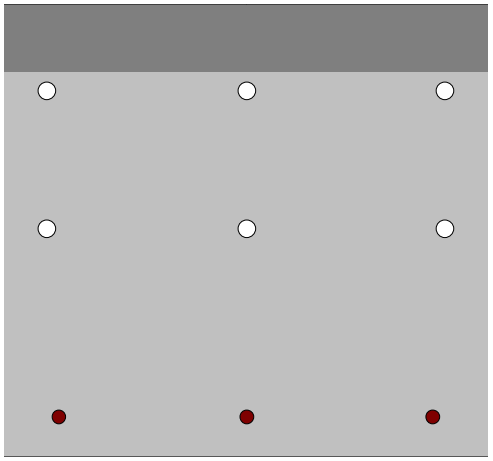
γ_{xy}(avg) = 0.00 ms

Axial Load = 6.1 kips

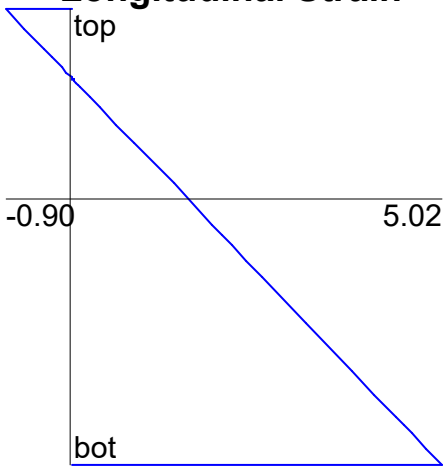
Moment:= 106.4 ft-kips

Shear = 0.0 kips

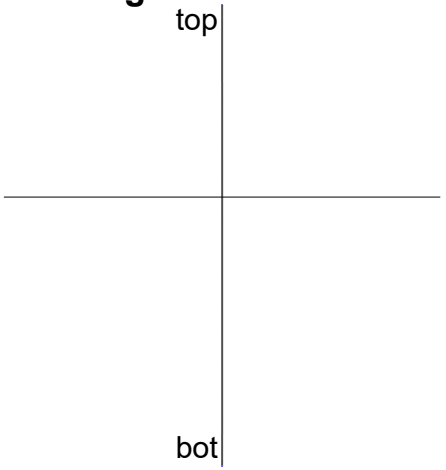
Cross Section



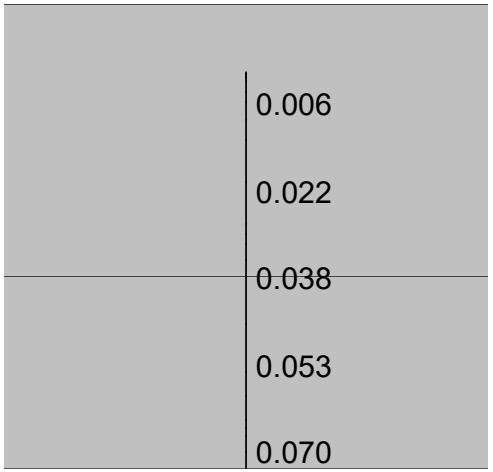
Longitudinal Strain



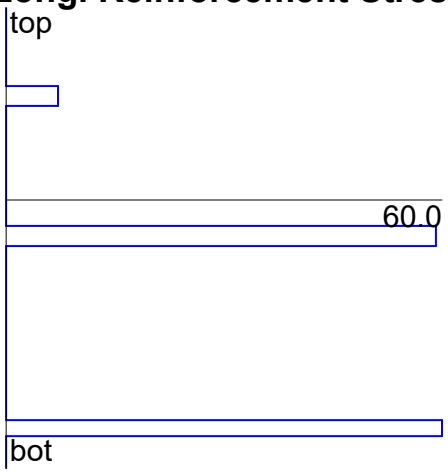
Shrinkage & Thermal Strain



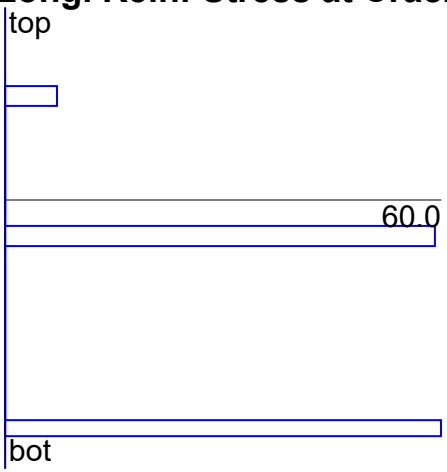
Crack Diagram



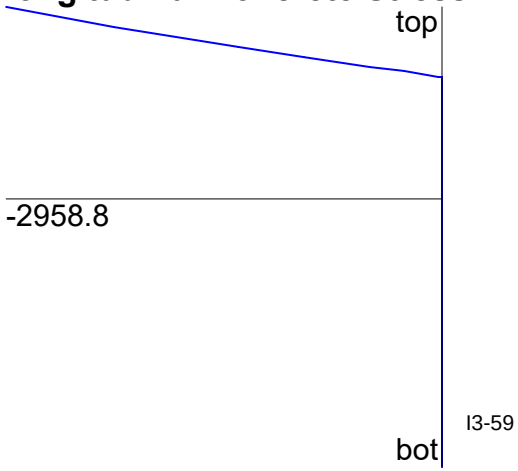
Long. Reinforcement Stress



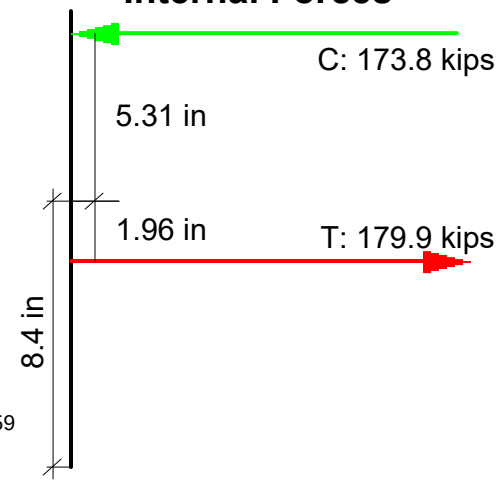
Long. Reinf Stress at Crack



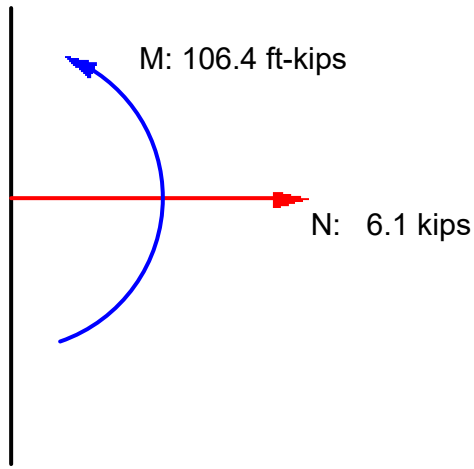
Longitudinal Concrete Stress



Internal Forces



N+M



COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I4

CALC BOOK COMPLETION DATE: **2021 Oct 26**

Title: Transverse Box Girder Design

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I4-0	-	I4-0	Calc Register (this sheet)	N/A	N/A	
2	I4-1	-	I4-1	Transverse Reinforcement Summary	LDLI	FCCA	
3	I4-2	-	I4-9	Transverse Deck Design	LDLI	FCCA	
4	I4-10	-	I4-15	Transverse Bottom Slab Design	LDLI	FCCA	
5	I4-16	-	I4-24	Overhang Design - End Zone Barrier Type F	LDLI	FCCA	
6	I4-25		I4-33	Overhang Design - Interior Zone Barrier Type F	LDLI	FCCA	
7	I4-34	-	I4-42	Overhang Design - End Zone Pedestrian Barrier	LDLI	FCCA	
8	I4-43	-	I4-51	Overhang Design - Interior Zone Pedestrian Barrier	LDLI	FCCA	
9	I4-52	-	I4-52	Develolpment Length Spreadsheet	LDLI	FCCA	

Transverse Reinforcement Summary

For deck (roadway slab):

	New West Span		
	Long rebar	Short rebar	design governed by
Top trans. bar – overhang (interior)	#5 @ 6”	#5@12"	ultimate capacity at face of barrier (AASHTO 5.6.3.2)
Bottom trans. bar – overhang (interior)	#4 @ 6"	-	
Top trans. bar – overhang (near Pier1)	#5 @ 6”	#6 @ 6”	ultimate capacity at face of barrier (AASHTO 5.6.3.2)
Bottom trans. bar – overhang (near Pier1)	#4 @ 6"	-	
Top trans. bar – over webs *	#5 @ 6”	-	minimum reinforcement check for cracking moment at cl of web (neg moment) (AASHTO 5.6.3.3)
Bott trans. bar – b/w webs	#5 @ 6”	-	distribution of reinforcement (AASHTO 9.7.3.2/BDM Fig. 5.7.2-2) and ultimate capacity for the positive moment at the cl of mid cell

For bottom slab:

	New West Span		
	Long rebar	Short rebar	design governed by
Top trans. bar – over webs	#4 @ 12”	-	minimum transverse reinforcement area in bottom slab (BDM 5.12.3.5.2b)
Bott trans. bar – b/w webs	#4 @ 12”	-	minimum transverse reinforcement area in bottom slab (BDM 5.12.3.5.2b)

Transverse Deck Design

Purpose and Assumptions

1. Calculate the capacity of the box girder top flange in resisting transverse bending moments resulting from both permanent and live loads. The permanent loads include self weight, barriers, sidewalks, and utilities.
2. The demands and capacities are calculated based on AASHTO LRFD Bridge Design Specification using the Approximate Elastic Method, or "Equivalent Strip Method" (AASHTO 4.6.2.1), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.
3. As per design criteria, the dimensions of the new portion of the box girder shall match the existing box girder

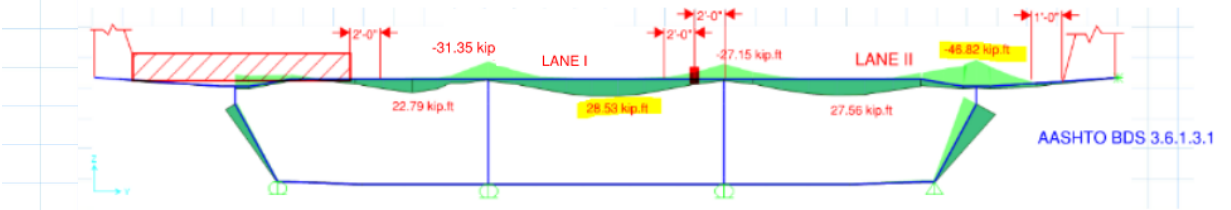
Material and Section Properties

Concrete strength	$f'_c := 5\ ksi$
Steel yield strength	$f_y := 60\ ksi$
Concrete top cover in deck	$c_{top} := 2.5\ in$
Concrete bottom cover in deck	$c_{bottom} := 1\ in$
Top Deck Thickness at max pos. M	$h_{top\_pos} := 8.5\ in$
Top Deck Thickness at max neg. M	$h_{top\_neg} := 8.5\ in + 6\ in = 14.5\ in$
Deck Width	$b := 12\ in$

Unfactored Live Loads (HL-93)

One or more axles (representing lanes of traffic) are placed on the deck and moved transversely to maximize the moments. Pedestrian live load, impact loads, and multiple presence factors are considered along with the truck loads or tandem loads. Since the magnitude of the truck loads are greater than those for the tandem, the maximum moment is due to truck loads. Therefore, in the numerical model, 1 truck/ 2 trucks are combined with the pedestrian live load to find the maximum positive and negative moment along the deck. The minimum transverse distance between the wheels of two adjacent axles is 4 ft and the minimum distance between the wheels and the edge of the lane is 2 ft. The resulting moments are divided by the strip width calculated from AASHTO LRFD Table 4.6.2.1.3-1 to obtain the live load moment per unit length of bridge.

The live load results are obtained from model **"Transverse Analysis" and load case "2 Truck".** The impact load and multiple presence factor are included in the results. The moment envelope is:



Maximum Positive Moment @ centerline of interior cell

$$LL_{pos} := \frac{28.53}{\cos(22\ deg)}\ kip \cdot ft$$

Maximum Negative Moment @ centerline of right exterior web

$$LL_{neg} := \frac{-46.82}{\cos(22\ deg)}\ kip \cdot ft$$

Spacing of supporting component

$$S := 8.31\ ft$$

Note: The demands are increased by the consine factor to account for the bridge skew and transverse reinforcement orientation

Note: to be conservative, this spacing is taken as the minimum web spacing in this box-girder. The spacing is obtained from the typical section ("CROSS-SECTION FOR TRANSVERSE MODEL2").

Width of transverse strip

$$W_{pos} := \frac{\left(26 + 6.6\,\frac{S}{ft}\right)\cdot ft}{12} = 6.74\ ft$$ AASHTO LRFD Table 4.6.2.1.3-1

$$W_{neg} := \frac{\left(48 + 3\,\frac{S}{ft}\right)}{12}\cdot ft = 6.08\ ft$$ AASHTO LRFD Table 4.6.2.1.3-1

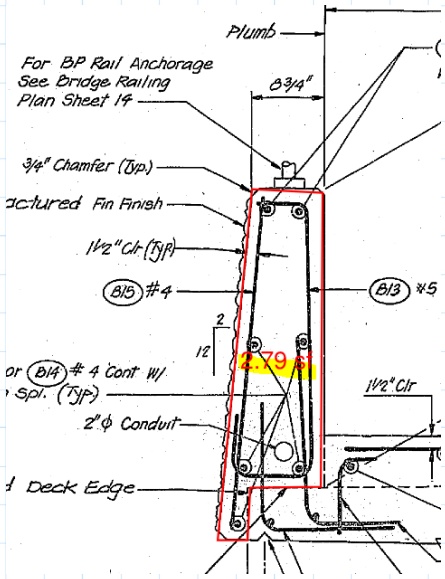
Maximum Positive Moment due to LL in deck

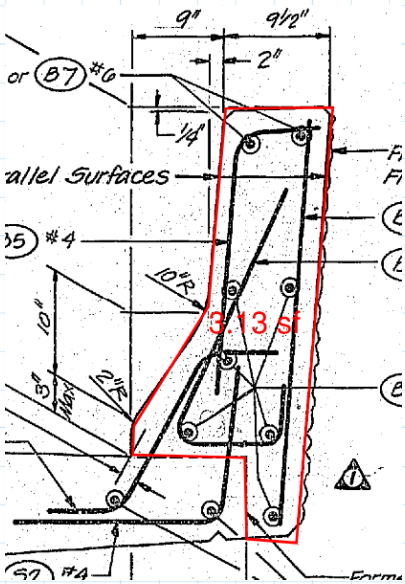
$$M_{LL\_pos} := \frac{LL_{pos}}{\frac{W_{pos}}{ft}} = 4.57\ kip\cdot ft$$

$$M_{LL\_neg} := \frac{LL_{neg}}{\frac{W_{neg}}{ft}} = -8.31\ kip\cdot ft$$

Unfactored Dead Loads

Dead Loads moments are calculated for a 1.00 ft wide section of the bridge deck using approved structural analysis methods, which include continuous beam equations, moment distribution, simple beam equations, or finite element analysis. Dead Load bending moments shown below are produced by a 2D transverse model in CSiBridge using a uniform load for sidewalk and two point loads for barriers. The self weight of the cross-section is considered by defining the material density. The concrete material density is increased from 0.155 kcf to 0.166 kcf to account for the self weight of utilities, tendons, and grout.





Dead Load calculation for analysis model:

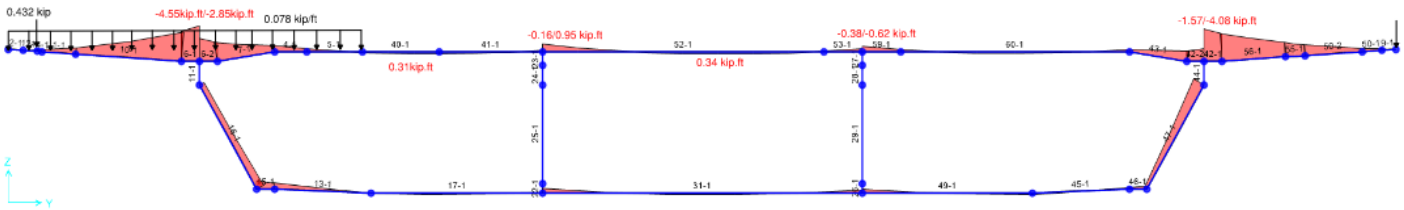
$$DC_{Barrier1} := 2.79\ ft^2 \cdot 0.155\ \frac{kip}{ft^3} = 0.43\ \frac{kip}{ft}$$ approximately at 4" from the edge of deck

$$DC_{Barrier2} := 3.13\ ft^2 \cdot 0.155\ \frac{kip}{ft^3} = 0.49\ \frac{kip}{ft}$$ approximately at the edge of deck

$$DC_{Sidewalk} := 0.155\ \frac{kip}{ft^3}\cdot 0.5\ ft = 0.08\ \frac{kip}{ft^2}$$ 0.875 ft from edge of deck to 9.205 ft from edge of deck

where: DC Barrier - self-weight of barrier
DC Sidewalk - self-weight of sidewalk

Results of analysis for deck:



@ centerline of interior cell $M_{DL\_pos} := \frac{0.34}{\cos(22\ deg)}\ kip \cdot ft = 0.37\ kip \cdot ft$

@ centerline of left exterior web $M_{DL\_neg} := \frac{-2.85}{\cos(22\ deg)}\ kip \cdot ft = -3.07\ kip \cdot ft$

Factored Design Loads

Strength I load Combinations

$$M_{u\_pos} := 1.25\ M_{DL\_pos} + 1.75\ M_{LL\_pos} = 8.45\ kip \cdot ft$$
$$M_{u\_neg} := 1.25\ M_{DL\_neg} + 1.75\ M_{LL\_neg} = -18.38\ kip \cdot ft$$

Reinforcement Sizes and Spacing

#5 @ 6 inches for both top and bottom reinforcements

$A_s := 0.31\ in^2$ $s := 6\ in$ $d_{bar} := 0.625\ in$

Deck Strength Design

Positive Moment Capacity

Resistance Factor $\varphi := 0.9$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber $d_{pos} := h_{top\_pos} - c_{bottom} - \frac{d_{bar}}{2} = 7.19\ in$

$$A_{s\_bottom\_min} := \frac{0.85\ f'_c \cdot b \cdot d_{pos} - \sqrt{(0.85\ f'_c \cdot b \cdot d_{pos})^2 - 1.7 \cdot f'_c \cdot b \cdot \frac{M_{u\_pos}}{\varphi}}}{f_y} = 0.27\ in^2$$

Area of Steel for Top Layer $A_{s\_bottom} := \frac{A_s}{\frac{s}{\frac{in}{12}}} = 0.62\ in^2$

Compression Block Depth $a_{pos} := \frac{A_{s\_bottom} \cdot f_y}{0.85\ f'_c \cdot b} = 0.73\ in$

Moment Resistance $M_{n\_pos} := A_{s\_bottom} \cdot f_y \cdot \left(d_{pos} - \frac{a_{pos}}{2} \right) = 21.15 \ kip \cdot ft$

$\frac{D}{C}$ $\frac{M_{u\_pos}}{\varphi \cdot M_{n\_pos}} = 0.44$

Negative Moment Capacity

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber $d_{neg} := h_{top\_neg} - c_{top} - \frac{d_{bar}}{2} = 11.69 \ in$

$$A_{s\_top\_min} := \frac{0.85 \ f'_c \cdot b \cdot d_{neg} - \sqrt{\left(0.85 \ f'_c \cdot b \cdot d_{neg}\right)^2 + 1.7 \cdot f'_c \cdot b \cdot \frac{M_{u\_neg}}{\varphi}}}{f_y} = 0.36 \ in^2$$

Area of Steel for Top Layer $A_{s\_top} := \frac{A_s}{\frac{s}{in}} = 0.62 \ in^2$
12

Compression Block Depth $a_{neg} := \frac{A_{s\_top} \cdot f_y}{0.85 \ f'_c \cdot b} = 0.73 \ in$

Moment Resistance $M_{n\_neg} := -A_{s\_top} \cdot f_y \cdot \left(d_{neg} - \frac{a_{neg}}{2} \right) = -35.1 \ kip \cdot ft$

$\frac{D}{C}$ $\frac{M_{u\_neg}}{\varphi \cdot M_{n\_neg}} = 0.58$

Minimum Reinforcement (AASHTO 2017 5.6.3.3)

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, Mr = φMn, at least equal to the lesser of:

- 1.33 times the factored ultimate moment
- Cracking moment

Flexural cracking variability factor $\gamma_1 := 1.6$

Ratio of specified min. yield strength to ultimate tensile strength for AASHTO M31 (ASTM A615), Grade 60 reinforcement $\gamma_3 := 0.67$

Concrete density modification factor AASHTO 5.4.2.8 $\lambda := 1$

Modulus of rupture of concrete AASHTO 5.4.2.6 $f_r := 0.24 \ ksi \ \lambda \sqrt{\frac{f'_c}{ksi}} = 0.54 \ ksi$

Section modulus of design section for positive moment

$$S_{c\_pos} := \frac{b \cdot h_{top\_pos}^{\ 2}}{6} = 144.5\ in^3$$

Positive cracking moment AASHTO 5.6.3.3

$$M_{cr\_pos} := \gamma_3 \cdot (\gamma_1 \cdot f_r) \cdot S_{c\_pos} = 6.93\ kip \cdot ft$$

Note: M_{cr_pos} governs

1.33 times the positive factored ultimate moment

$$1.33\ M_{u\_pos} = 11.24\ kip \cdot ft$$

$\frac{D}{C}$

$$\frac{M_{cr\_pos}}{\varphi \cdot M_{n\_pos}} = 0.36$$

Section modulus of design section for negative moment

$$S_{c\_neg} := \frac{b \cdot h_{top\_neg}^{\ 2}}{6} = 420.5\ in^3$$

Negative cracking moment AASHTO 5.6.3.3

$$M_{cr\_neg} := -\gamma_3 \cdot (\gamma_1 \cdot f_r) \cdot S_{c\_neg} = -20.16\ kip \cdot ft$$

Note: M_{cr_neg} governs

1.33 times the negative factored ultimate moment

$$1.33\ M_{u\_neg} = -24.45\ kip \cdot ft$$

$\frac{D}{C}$

$$\frac{M_{cr\_neg}}{\varphi \cdot M_{n\_neg}} = 0.64$$

Minimum Reinforcement for Shrinkage and Temperature (AASHTO 2017 5.10.6)

Minimum reinforcement is required in all directions to accommodate shrinkage and temperature changes near the surface of the slab. Longitudinal reinforcement on each face shall meet the following

$$A_S \geq \frac{1.3 b\, t_{Deck}}{2(b + t_{Deck}) f_y}$$

$$0.11 \leq A_S \leq 0.60$$

AASHTO 5.10.6-1
AASHTO 5.10.6-2

For top slab:

$$A_{smin} := \frac{1.3\ b \cdot h_{top\_neg}}{2 \cdot \frac{(b + h_{top\_neg})}{in} \cdot \frac{f_y}{ksi}} = 0.071\ in^2$$

Note: to be conservative, minimum area is calculated based on max thickness of deck

$$A_{smin} := \max\left(A_{smin}, 0.11\ in^2\right) = 0.11\ in^2$$

$$A_{smin} := min\left(A_{smin}, 0.6\ in^2\right) = 0.11\ in^2$$

$\frac{D}{C}$

$$\frac{A_{smin}}{A_{s\_top}} = 0.177$$

Therefore, the deck has minimum reinforcement for shrinkage and temperature

Control of Cracking at Service Limit State (AASHTO 2017 5.6.7)

$$M_{u\_service\_pos} := 1\ M_{DL\_pos} + 1\ M_{LL\_pos} = 4.93\ kip \cdot ft$$

$$M_{u\_service\_neg} := 1\ M_{DL\_neg} + 1\ M_{LL\_neg} = -11.38\ kip \cdot ft$$

Check cracking at the bottom of deck

Exposure factor Class 2 Exposure (AASHTO 5.6.7)

$$\gamma_{e\_bot} := 0.75$$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto

$$d_{c\_bottom} := c_{bottom} + \frac{1}{2} \cdot d_{bar} = 1.31\ in$$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (EQ 5.6.7-2)

$$\beta_{s\_bottom} := 1 + \frac{d_{c\_bottom}}{0.7 \cdot \left(h_{top\_pos} - d_{c\_bottom}\right)} = 1.26$$

Tension Reinforcement Ratio

$$\rho_{bottom} := \frac{A_{s\_bottom}}{b \cdot d_{pos}} = 7.19 \cdot 10^{-3}$$

Young's Modulus of Steel

$$E_s := 29000\ ksi$$

Unit Weight of Concrete

$$w_c := 0.155\ \frac{kip}{ft^3}$$

Correction Factor for Source of Aggregate (5.4.2.4)

$$K_1 := 1$$

Young's Modulus of Concrete (5.4.2.4)

$$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} \cdot ksi = \left(4.9 \cdot 10^3\right)\ ksi$$

Modular Ratio

$$n := \frac{E_s}{E_c} = 5.91$$

$$k_{bottom} := \sqrt{2\ n \cdot \rho_{bottom} + \left(n \cdot \rho_{bottom}\right)^2} - n \cdot \rho_{bottom} = 0.25$$

$$j_{bottom} := 1 - \frac{k_{bottom}}{3} = 0.92$$

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss\_bottom} := \frac{M_{u\_service\_pos}}{A_{s\_bottom} \cdot j_{bottom} \cdot d_{pos}} = 14.51\ ksi$$

Maximum reinforcement spacing for positive moment

$$s_{max\_bottom} := \frac{700 \cdot \gamma_{e\_bot} \cdot in}{\beta_{s\_bottom} \cdot \frac{f_{ss\_bottom}}{ksi}} - 2 \cdot d_{c\_bottom} = 26.08\ in$$

Service limit state check

$$\text{if}\left(s < s_{max\_bottom}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$	$\frac{s}{s_{max\_bottom}} = 0.23$

Check cracking at the top of deck

Exposure factor Class 2 Exposure (AASHTO 5.6.7)	$\gamma_{e\_top} := 0.75$
Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto	$d_{c\_top} := c_{top} + \frac{1}{2} \cdot d_{bar} = 2.81\ in$
Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (EQ. 5.6.7-2)	$\beta_{s\_top} := 1 + \frac{d_{c\_top}}{0.7 \cdot (h_{top\_neg} - d_{c\_top})} = 1.34$
Tension Reinforcement Ratio	$\rho_{top} := \frac{A_{s\_top}}{b \cdot d_{neg}} = 4.42 \cdot 10^{-3}$
	$k_{top} := \sqrt{2\ n \cdot \rho_{top} + (n \cdot \rho_{top})^2} - n \cdot \rho_{top} = 0.2$
	$j_{top} := 1 - \frac{k_{top}}{3} = 0.93$
Tensile stress in nonprestressed reinforcement at SLS	$f_{ss\_top} := \frac{-M_{u\_service\_neg}}{A_{s\_top} \cdot j_{top} \cdot d_{neg}} = 20.23\ ksi$
Maximum allowable spacing for negative moment	$s_{max\_top} := \frac{700 \cdot \gamma_{e\_top} \cdot in}{\beta_{s\_top} \cdot \frac{f_{ss\_top}}{ksi}} - 2 \cdot d_{c\_top} = 13.69\ in$
Service limit state check	$\text{if}(s < s_{max\_top}, \text{“pass”}, \text{“fail”}) = \text{“pass”}$
$\frac{D}{C}$	$\frac{s}{s_{max\_top}} = 0.44$

Check Distribution of Reinforcement (AASHTO 9.7.3.2)

Mid cell C-C width	$b_{midcell} := 7\ ft + 7.5\ in + 1\ ft + 0.5\ in = 8.67\ ft$	Note: dimensioned in as-built framing plan for west span
Web thickness	$t_{web} := 9\ in$	
Fillet width	$t_{fillet} := 6\ in$	
Effective span length (BDM Figure 5.3.1-1)	$s_{eff} := b_{midcell} - t_{web} - t_{fillet} = 7.42\ ft$	
Amount of reinforcement required in secondary direction in the bottom of the slab (AASHTO 9.7.3.2)	$percent_{req} := min\left(67, \frac{220}{\sqrt{\frac{s_{eff}}{ft}}}\right) = 67$	

Number of long. reinforcement in bottom mat	$N_{long} := 41$
Long. bar area	$A_{long} := 0.6\ in^2$
Deck Width	$B := 36.1\ ft$
Long. reinforcement density	$\rho_{long} := N_{long} \cdot \frac{A_{long}}{B} = 0.68\ \frac{in^2}{ft}$
Transverse bar area	$A_{trans} := 0.31\ in^2$
Transverse bar spacing	$s = 6\ in$
Transverse reinforcement density	$\rho_{trans} := \frac{A_{trans}}{s} = 0.62\ \frac{in^2}{ft}$
Ratio between long. and trans. bar	$percent := \frac{\rho_{long}}{\rho_{trans}} \cdot 100 = 109.91$
$\frac{D}{C}$	$\frac{percent_{req}}{percent} = 0.61$

Note: to be conservative, the deck width is taken as the width between the edges of overhang for the calculation of longitudinal reinforcement density

Summary

#5 @6 inches for both top and bottom reinforcements

	Demand	Capacity	D/C Ratio
Ultimate Capacity	$M_{u\_pos} = 8.45\ kip \cdot ft$	$\varphi \cdot M_{n\_pos} = 19.04\ kip \cdot ft$	$\frac{M_{u\_pos}}{\varphi \cdot M_{n\_pos}} = 0.44$
	$M_{u\_neg} = -18.38\ kip \cdot ft$	$\varphi \cdot M_{n\_neg} = -31.59\ kip \cdot ft$	$\frac{M_{u\_neg}}{\varphi \cdot M_{n\_neg}} = 0.58$
Minimum Reinforcement	$M_{cr\_pos} = 6.93\ kip \cdot ft$	$\varphi \cdot M_{n\_pos} = 19.04\ kip \cdot ft$	$\frac{M_{cr\_pos}}{\varphi \cdot M_{n\_pos}} = 0.36$
	$M_{cr\_neg} = -20.16\ kip \cdot ft$	$\varphi \cdot M_{n\_neg} = -31.59\ kip \cdot ft$	$\frac{M_{cr\_neg}}{\varphi \cdot M_{n\_neg}} = 0.64$

Maximum Spacing at SLS

	Maximum Spacing	Actual Spacing	D/C Ratio
Top Layer	$s_{max\_top} = 13.69\ in$	$s = 6\ in$	$\frac{s}{s_{max\_top}} = 0.44$
Bottom	$s_{max\_bottom} = 26.08\ in$	$s = 6\ in$	$\frac{s}{s_{max\_bottom}} = 0.23$

Distribution of Reinforcement

	Demand	Capacity	D/C Ratio
Bottom	$percent_{req} = 67$	$percent = 109.91$	$\frac{percent_{req}}{percent} = 0.61$

Transverse Bottom Slab Design

Purpose and Assumptions

1. Calculate the capacity of the box girder bottom flange in resisting transverse bending moments resulting from the permanent loads, which include self weight, barriers, sidewalks, and utilities. Live load moment demands are negligible in the bottom slab.
2. The demands and capacities are calculated based on AASHTO LRFD Bridge Design Specification using the Approximate Elastic Method, or "Equivalent Strip Method" (AASHTO 4.6.2.1), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.
3. As per design criteria, the dimensions of the new portion of the box girder shall match the existing box girder

Material and Section Properties

Concrete strength	$f'_c := 5\ ksi$
Steel yield strength	$f_y := 60\ ksi$
Concrete top and bottom cover in bottom slab	$c_{bottom} := 1\ in$
Bottom Deck Thickness	$h_{bottom} := 6\ in$
Slab Strip Width	$b := 12\ in$

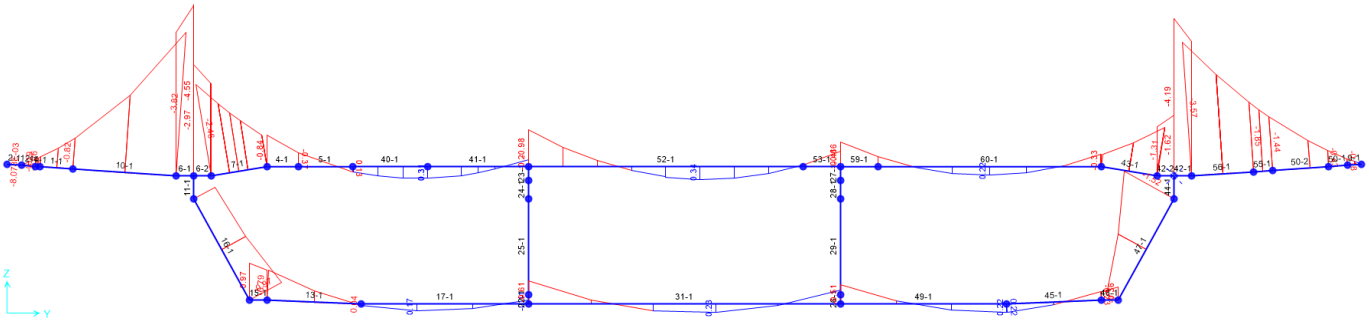
Unfactored Dead Loads

Dead Loads moments are calculated for a 1.00 ft. wide section of the bridge deck using approved structural analysis methods, which include continuous beam equations, moment distribution, simple beam equations, or finite element analysis. Dead Load bending moments shown below are produced by a 2D transverse model in CSiBridge using a uniform load for sidewalk and two point loads for barriers. The self weight of the cross-section is considered by defining the material density. The concrete material density is increased from 0.155 kcf to 0.166 kcf to account for the self weight of utilities, tendons, and grout.

$$M_{DL\_neg} := \frac{-0.97}{\cos(22\ deg)}\ kip \cdot ft$$

$$M_{DL\_pos} := \frac{0.23}{\cos(22\ deg)}\ kip \cdot ft$$

The demands are increased by the consine factor to account for the bridge skew and transverse reinforcement orientation



Factored Design Loads

Strength I load Combinations

$$M_{u\_pos} := 1.25 \cdot M_{DL\_pos} = 0.31\ kip \cdot ft$$
$$M_{u\_neg} := 1.25 \cdot M_{DL\_neg} = -1.308\ kip \cdot ft$$

Deck Strength Design

Positive Moment Capacity (AASHTO 2017 5.6.3.2)

Note: Assume #4 bars at 12 inches for bottom reinforcement

$s_{bottom} := 12\ in$

Area of Tension Transverse Reinforcement

$$A_{s\_bottom} := \frac{0.2\ in^2}{\left(\frac{s_{bottom}}{\frac{in}{12}}\right)} = 0.2\ in^2$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber

$$d_{pos} := h_{bottom} - c_{bottom} - 0.5\ \frac{in}{2} = 4.75\ in$$

Compression Block Depth

$$a_{pos} := \frac{A_{s\_bottom} \cdot f_y}{0.85\ f'_c \cdot b} = 0.235\ in$$

Unfactored Resistance

$$M_{n\_pos} := A_{s\_bottom} \cdot f_y \cdot \left(d_{pos} - \frac{a_{pos}}{2}\right) = 4.632\ kip \cdot ft$$

Resistance Factor

$$\varphi := 0.9$$

Factored Resistance

$$\varphi \cdot M_{n\_pos} = 4.169\ kip \cdot ft$$

D/C ratio

$$\frac{M_{u\_pos}}{\varphi \cdot M_{n\_pos}} = 0.074$$

Negative Moment Capacity (AASHTO 2017 5.6.3.2)

Note: Assume #4 bars at 12 inches for top reinforcement

$s_{top} := 12\ in$

Area of Tension Transverse Reinforcement

$$A_{s\_top} := \frac{\left(0.2\ in^2\right)}{\frac{s_{top}}{\frac{in}{12}}} = 0.2\ in^2$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber

$$d_{neg} := h_{bottom} - c_{bottom} - 0.5\ \frac{in}{2} = 4.75\ in$$

Compression Block Depth

$$a_{neg} := \frac{A_{s\_top} \cdot f_y}{0.85\ f'_c \cdot b} = 0.235\ in$$

Unfactored Resistance

$$M_{n\_neg} := -A_{s\_top} \cdot f_y \cdot \left(d_{neg} - \frac{a_{neg}}{2}\right) = -4.632\ kip \cdot ft$$

Resistance Factor

$$\varphi := 0.9$$

Factored Resistance

$$\varphi \cdot M_{n\_neg} = -4.169\ kip \cdot ft$$

D/C ratio

$$\frac{M_{u\_neg}}{\varphi \cdot M_{n\_neg}} = 0.314$$

Minimum Reinforcement (AASHTO 2017 5.6.3.3)

Unless otherwise specified, the amount of prestressed and non-prestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, Mr = φMn, at least equal to the lesser of:

- 1.33 times the factored ultimate moment
- Cracking moment

Flexural cracking variability factor

$$\gamma_1 := 1.6$$

Ratio of specified min. yield strength to ultimate tensile strength for AASHTO M31 (ASTM A615) Grade 60 reinforcement

$\gamma_3 := 0.67$

Concrete density modification factor AASHTO 5.4.2.6

$\lambda := 1$

Modulus of rupture of concrete AASHTO 5.4.2.6

$f_r := 0.24\ ksi\ \lambda \sqrt{\frac{f'_c}{ksi}} = 0.537\ ksi$

Section modulus of design section

$S_c := \frac{b \cdot h_{bottom}^{\ 2}}{6} = 72\ in^3$

Positive cracking moment AASHTO 5.6.3.3

$M_{cr\_pos} := \gamma_3 \cdot (\gamma_1 \cdot f_r) \cdot S_c = 3.452\ kip \cdot ft$

Note: $1.33\ M_{u\_pos}$ governs

1.33 times the positive factored ultimate moment

$1.33\ M_{u\_pos} = 0.412\ kip \cdot ft$

D/C ratio

$\frac{1.33\ M_{u\_pos}}{\varphi \cdot M_{n\_pos}} = 0.099$

Negative cracking moment AASHTO 5.6.3.3

$M_{cr\_neg} := -\gamma_3 \cdot (\gamma_1 \cdot f_r) \cdot S_c = -3.452\ kip \cdot ft$

Note: $1.33\ M_{u\_neg}$ governs

1.33 times the positive factored ultimate moment

$1.33\ M_{u\_neg} = -1.739\ kip \cdot ft$

D/C ratio

$\frac{1.33\ M_{u\_neg}}{\varphi \cdot M_{n\_neg}} = 0.417$

Minimum Reinforcement for Shrinkage and Temperature (AASHTO 2017 5.10.6)

Minimum reinforcement is required in all directions to accommodate shrinkage and temperature changes near the surface of the slab. Longitudinal reinforcement on each face shall meet the following

$A_S \geq \frac{1.3b\ t_{Deck}}{2(b + t_{Deck}) f_y}$ AASHTO 5.10.6-1

$0.11 \leq A_S \leq 0.60$ AASHTO 5.10.6-2

For bottom slab:

$$A_{smin1} := min\left(0.6\ in^2, \frac{1.3\ b \cdot h_{bottom}}{2 \cdot \frac{(b + h_{bottom})}{in} \cdot \frac{f_y}{ksi}}\right) = 0.043\ in^2$$

$$A_{smin2} := \max\left(0.11\ in^2, \frac{1.3\ b \cdot h_{bottom}}{2 \cdot \frac{(b + h_{bottom})}{in} \cdot \frac{f_y}{ksi}}\right) = 0.11\ in^2$$

D/C ratio

$\frac{min(A_{smin1}, A_{smin2})}{A_{s\_top}} = 0.217$

Therefore, the bottom slab has minimum reinforcement for shrinkage and temperature

Control of Cracking at Service Limit State (AASHTO 5.6.7)

$M_{u\_service\_pos} := 1 \quad M_{DL\_pos} = 0.248\ kip \cdot ft$

$M_{u\_service\_neg} := 1 \quad M_{DL\_neg} = -1.046\ kip \cdot ft$

Check cracking at the bottom of bottom slab

Exposure factor for Class 2 Exposure Condition (AASHTO 5.6.7)

$\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto

$d_{c\_bottom} := c_{bottom} + \frac{1}{2} \cdot 0.5\ in = 1.25\ in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7)

$\beta_{s\_bottom} := 1 + \frac{d_{c\_bottom}}{0.7 \cdot \left(h_{bottom} - d_{c\_bottom}\right)} = 1.376$

Tension Reinforcement Ratio

$\rho_{bottom} := \frac{A_{s\_bottom}}{b \cdot d_{pos}} = 0.004$

Young's Modulus of Steel

$E_s := 29000\ ksi$

Unit Weight of Concrete

$w_c := 0.155\ \frac{kip}{ft^3}$

Correction Factor for Source of Aggregate (5.4.2.4)

$K_1 := 1$

Young's Modulus of Concrete (5.4.2.4)

$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} \cdot ksi = \left(4.903 \cdot 10^3\right)\ ksi$

Modular Ratio

$n := \frac{E_s}{E_c} = 5.914$

$k_{bottom} := \sqrt{2\ n \cdot \rho_{bottom} + \left(n \cdot \rho_{bottom}\right)^2} - n \cdot \rho_{bottom} = 0.184 \qquad j_{bottom} := 1 - \frac{k_{bottom}}{3} = 0.939$

Tensile stress in nonprestressed reinforcement at SLS

$f_{ss\_bottom} := \frac{M_{u\_service\_pos}}{A_{s\_bottom} \cdot j_{bottom} \cdot d_{pos}} = 3.338\ ksi$

Maximum reinforcement spacing for positive moment

$s_{max\_bottom} := \frac{700 \cdot \gamma_e \cdot in}{\beta_{s\_bottom} \cdot \frac{f_{ss\_bottom}}{ksi}} - 2 \cdot d_{c\_bottom} = 111.8\ in$

Service limit state check

$$\text{if}\left(s_{bottom} < s_{max\_bottom}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

D/C ratio

$$\frac{s_{bottom}}{s_{max\_bottom}} = 0.107$$

Check cracking at the top of bottom slab

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto

$$d_{c\_top} := c_{bottom} + \frac{1}{2} \cdot 0.5\ in = 1.25\ in$$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (EQ 5.6.7-2)

$$\beta_{s\_top} := 1 + \frac{d_{c\_top}}{0.7 \cdot \left(h_{bottom} - d_{c\_top}\right)} = 1.376$$

Tension Reinforcement Ratio

$$\rho_{top} := \frac{A_{s\_top}}{b \cdot d_{neg}} = 0.004$$

$$k_{top} := \sqrt{2\ n \cdot \rho_{top} + \left(n \cdot \rho_{top}\right)^2} - n \cdot \rho_{top} = 0.184 \qquad j_{top} := 1 - \frac{k_{top}}{3} = 0.939$$

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss\_top} := \frac{M_{u\_service\_pos}}{A_{s\_top} \cdot j_{top} \cdot d_{neg}} = 3.338\ ksi$$

Maximum reinforcement spacing for negative moment

$$s_{max\_top} := \frac{700 \cdot \gamma_e \cdot in}{\beta_{s\_top} \cdot \frac{f_{ss\_top}}{ksi}} - 2 \cdot d_{c\_top} = 111.8\ in$$

Service limit state check

$$\text{if}\left(s_{top} < s_{max\_top}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

D/C ratio

$$\frac{s_{top}}{s_{max\_top}} = 0.107$$

Bottom Slab Reinforcement in Cast-in Place Box girders (AASHTO 2017 5.12.3.5.2b)

Slab area

$$A_{slab} := b \cdot h_{bottom} = 0.5\ ft^2$$

Minimum bottom slab transverse reinforcement (5.12.3.5.2b)

$$A_{smin'} := 0.005 \cdot A_{slab} = 0.36\ in^2$$

D/C ratio for top layer

$$\frac{\frac{A_{smin'}}{2}}{A_{s\_top}} = 0.9$$

D/C ratio for bottom layer

$$\frac{\frac{A_{smin'}}{2}}{A_{s\_bottom}} = 0.9$$

Summary

#4 bars at 12 inches for both top and bottom reinforcements

	Demand	Capacity	D/C Ratio
Ultimate Capacity	$M_{u\_pos} = 0.31\ kip \cdot ft$	$\varphi \cdot M_{n\_pos} = 4.169\ kip \cdot ft$	$\frac{M_{u\_pos}}{\varphi \cdot M_{n\_pos}} = 0.074$
	$M_{u\_neg} = -1.308\ kip \cdot ft$	$\varphi \cdot M_{n\_neg} = -4.169\ kip \cdot ft$	$\frac{M_{u\_neg}}{\varphi \cdot M_{n\_neg}} = 0.314$
Minimum Reinforcement	$1.33\ M_{u\_pos} = 0.412\ kip \cdot ft$	$\varphi \cdot M_{n\_pos} = 4.169\ kip \cdot ft$	$\frac{1.33\ M_{u\_pos}}{\varphi \cdot M_{n\_pos}} = 0.099$
	$1.33\ M_{u\_neg} = -1.739\ kip \cdot ft$	$\varphi \cdot M_{n\_neg} = -4.169\ kip \cdot ft$	$\frac{1.33\ M_{u\_neg}}{\varphi \cdot M_{n\_neg}} = 0.417$
Minimum Reinforcement for Shrinkage and Temperature	$min\left(A_{smin1}, A_{smin2}\right) = 0.043\ in^2$	$A_{s\_top} = 0.2\ in^2$	$\frac{min\left(A_{smin1}, A_{smin2}\right)}{A_{s\_top}} = 0.217$
Control of Cracking for Bottom of Bottom Slab	$s_{bottom} = 12\ in$	$s_{max\_bottom} = 111.8\ in$	$\frac{s_{bottom}}{s_{max\_bottom}} = 0.107$
Control of Cracking for Top of Bottom Slab	$s_{top} = 12\ in$	$s_{max\_top} = 111.8\ in$	$\frac{s_{top}}{s_{max\_top}} = 0.107$
Reinforcement Area-Top Layer (AASHTO 5.12.3.5.2b)	$\frac{A_{smin'}}{2} = 0.18\ in^2$	$A_{s\_top} = 0.2\ in^2$	$\frac{\frac{A_{smin'}}{2}}{A_{s\_top}} = 0.9$
Reinforcement Area-Bottom Layer (AASHTO 5.12.3.5.2b)	$\frac{A_{smin'}}{2} = 0.18\ in^2$	$A_{s\_bottom} = 0.2\ in^2$	$\frac{\frac{A_{smin'}}{2}}{A_{s\_bottom}} = 0.9$

Transverse Overhang Design - Barrier Type F 32" - End Region

Purpose and Assumptions

1. Calculate the capacity of the top flange overhang of a concrete box girder in resisting transverse bending moments resulting from both permanent and live loads. The permanent loads include self weight, barriers, sidewalks, and utilities.
2. The capacities are calculated based on AASHTO LRFD Bridge Design Specification using the Approximate Elastic Method, or "Equivalent Strip Method" (AASHTO 4.6.2.1), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.
3. Bridge deck overhang is designed for three separate design cases (AASHTO A13.4.1):
   - Case 1 - Horizontal forces from vehicle collision load (Extreme Event II limit state)
   - Case 2 - Vertical force from vehicle collision load (Extreme Event II limit state)
   - Case 3 - Vertical Dead and Live Load at the overhang section (Strength I limit state)
4. For each case, the design is conducted for two sections (AASHTO A13.4.1):
   - Section 1 - At the interior face of the barrier
   - Section 2 - At the exterior face of the web (Overhang critical section - AASHTO BDM 4.6.2.1-6)

As per design criteria, a barrier of test level TL-4 is used. The design forces for traffic barriers are obtained from Table A13.2-1 in AASHTO. As per BDM Table 10.2.4-1, the three moment capacities needed in the following calculations for Type F 32 in TL-4 are given as:

Table 10.2.4-1

Barrier Impact Design Forces on Traffic Barrier & Deck Overhang

Parameters		Type F 32 in. (TL-4)		Single Slope 34 in. (TL-4)		Type F 42 in. (TL-4)		Single Slope 42 in. (TL-4)		Type F 42 in. (TL-5)		Single Slope 42 in. (TL-5)	
		Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*
Traffic Barrier Design	Average M_c (ft-kips/ft)	20.55	20.55	19.33	19.33	25.93	25.93	22.42	22.42	25.93	25.93	22.42	36.04
	M_c at Base (ft-kips/ft)	27.15	27.15	26.03	26.03	32.87	32.87	30.66	30.66	32.87	32.87	30.66	49.52
	M_w (ft-kips)	42.47	46.04	46.01	43.16	72.54	71.72	60.66	57.26	98.23	96.93	83.85	79.12
	L_c (ft)	8.62	4.76	9.30	4.81	10.77	5.32	10.63	5.21	15.05	9.39	14.99	8.87
	R_w (kips)	132.82	73.31	126.92	65.69	159.62	78.83	136.17	66.81	223.00	139.20	192.02	182.61
	F_t (kips)	54.00	54.00	54.00	54.00	54.00	54.00	54.00	54.00	124.00	124.00	124.00	124.00
Deck Overhang Design	1.2*F_t (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	148.80	148.80	148.80
	Design R_w (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	139.20	148.80	148.80
	R_w*H/(L_c+aH) (ft-kips/ft)**	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	37.79	23.69	42.11
	Design M_s (ft-kips/ft)	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	32.87	23.69	42.11
	Design T (kips/ft)	4.65	8.73	4.33	8.47	3.65	7.35	3.68	7.44	6.75	10.80	6.77	12.03
Deck to Barrier Reinforcement	A_s required (in²/ft)	0.29	0.57	0.29	0.59	0.17	0.35	0.20	0.41	0.43	0.60	0.49	0.91
	A_s provided (in²/ft)	0.41	0.62	0.41	0.62	0.41	0.62	0.41	0.62	0.59	0.89	0.59	0.97
	S_1 Bars	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#6 @ 9 in	#6 @ 6 in	#6 @ 8 in	#6 @ 5.5 in

Horizontal moment capacity (adjusted for bridge skew and transverse reinforcement orientation)

$$M_s := \frac{23.28}{\cos(22\ deg)}\ kip \cdot ft$$

Lever arm for vertical collision

$$l_a := 10.5\ in = 0.88\ ft$$

Longitudinal distribution of vertical force (AASHTO Table A13.2-1)

$$L_v := 18\ ft$$

Vertical design force (AASHTO Table A13.2-1)

$$F_v := 18\ kip$$

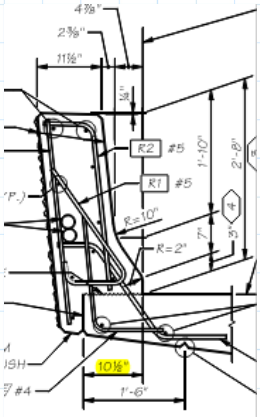
Tensile force from the barrier (adjusted for bridge skew)

$$T_1 := \frac{8.73}{\cos(22\ deg)}\ kip$$

Critical length of yield line

$$L_c := 4.76\ ft$$

Note: la is the horizontal distance between the interior face of the barrier to the centroid of the barrier. The centroid is approximately located at the edge of deck (see section below)



Material and Section Properties

Concrete strength

$$f'_c := 5\ ksi$$

Steel yield strength

$$f_y := 60\ ksi$$

Concrete top cover

$$c_{top} := 2.5\ in$$

Concrete bottom cover in deck and top and bottom covers in bottom slab

$$c_{bottom} := 1\ in$$

Overhang thickness at end	$t_{overhang} := 7\ in$
Top Deck Thickness	$t_{top} := 8.5\ in$
Overhang length	$L_{overhang} := 4.5\ ft$
Horizontal distance between the edge of deck and the inside face of the barrier	$L_{barrier} := 10.5\ in$
Top Deck Thickness at Fillet	$t_{top\_fillet} := 6\ in + t_{top} = 14.5\ in$
Overhang thickness at face of barrier	$t_{overhang\_barrier} := t_{overhang} + \left(\frac{t_{top\_fillet} - t_{overhang}}{L_{overhang}}\right) \cdot \left(L_{barrier}\right) = 8.46\ in$
Overhang thickness at outside face of web	$t_{overhang\_web} := t_{top\_fillet} = 14.5\ in$
Deck Width	$b := 12\ in$

Section 1: Overhang design at the interior face of the barrier

Design Case A: Horizontal Vehicular Collision Load

The deck overhang is designed to resist an axial tension force from vehicular collision acting simultaneously with the Dead Load moment.

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

$$M_{slab} := \frac{-0.04}{\cos(22\ deg)}\ kip \cdot ft = -0.04\ kip \cdot ft$$

$$M_{barrier} := \frac{-0.36}{\cos(22\ deg)}\ kip \cdot ft = -0.39\ kip \cdot ft$$

$$M_{DC} := M_{slab} + M_{barrier} = -0.43\ kip \cdot ft$$

$$M_{u1A} := -M_s + 1.0\ M_{DC} = -25.54\ kip \cdot ft$$

Note: Assume #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #6 @6"
For #5@ 6" (typ. top transverse reinforcement):

$A_{s\_top1} := 0.31\ in^2$ $s_{top1} := 6\ in$ $d_{bar\_top1} := 0.625\ in$

For additional short hooked #6@ 6" :

$A_{s\_top2} := 0.44\ in^2$ $s_{top2} := 6\ in$ $d_{bar\_top2} := 0.75\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5	$d_{1\_\#5} := t_{overhang\_barrier} - c_{top} - \frac{d_{bar\_top1}}{2} = 5.65\ in$
Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #6	$d_{1\_\#6} := t_{overhang\_barrier} - c_{top} - \frac{d_{bar\_top2}}{2} = 5.58\ in$
Resistance Factor for Extreme Load Combination	$\varphi_{ext} := 1$
Area of reinforcement per ft	$A_{s\_top1'} := \frac{A_{s\_top1}}{\frac{s_{top1}}{in}} = 0.62\ in^2$ $A_{s\_top2'} := \frac{A_{s\_top2}}{\frac{s_{top2}}{in}} = 0.88\ in^2$

The development length of #5 is 10.2". The development length of #6 is 12.2". The calculations are at spreadsheet "development length"

Development length of #5 $l_{d\_5} := 10.2\ in$

Development length of #6 $l_{d\_6} := 12.2\ in$

Note: the development length is restrained by the distance from the deck edge to the interior surface of the barrier and concrete cover

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #5 $f_{y\_M5} := f_y \cdot \frac{min\left(l_t, l_{d\_5}\right)}{l_{d\_5}} = 50\ ksi$

Modified stress of #6 $f_{y\_M6} := f_y \cdot \frac{min\left(l_t, l_{d\_6}\right)}{l_{d\_6}} = 41.8\ ksi$

Compression Block Depth $a_1 := \frac{\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5} + \left(A_{s\_top2'}\right) \cdot f_{y\_M6}}{0.85\ f'_c \cdot b} = 1.24\ in$

Unfactored resistance for top slab

$$M_{n\_top1} := -\left(\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5}\right) \cdot \left(d_{1\_\#5} - \frac{a_1}{2}\right) - \left(\left(A_{s\_top2'}\right) \cdot f_{y\_M6}\right) \cdot \left(d_{1\_\#6} - \frac{a_1}{2}\right) = -26.24\ kip \cdot ft$$

Factored Resistance $\varphi_{ext} \cdot M_{n\_top1} = -26.24\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(\left|M_{u1A}\right| < \left|\varphi_{ext} \cdot M_{n\_top1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.97$

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement :

Note: Assume #4 bars at 6 inches for bottom reinforcement

$A_{s\_bottom} := 0.2 \cdot in^2$ $s_{bottom} := 6\ in$ $d_{bar\_bottom} := 0.5\ in$

Development length of #4 $l_{d\_4} := 20\ in$

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #4 $f_{y\_M4} := f_y \cdot \frac{min\left(l_{d\_4}, l_t\right)}{l_{d\_4}} = 25.5\ ksi$

Area of Tension Transverse Reinforcement $A_{s\_bottom'} := \frac{\left(A_{s\_bottom}\right)}{\left(\frac{s_{bottom}}{in}\right)} = 0.4\ in^2$

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom1} := A_{s\_bottom'} \cdot f_{y\_M4} = 10.2\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$

Check for reinforcement in top slab $\text{if}\left(\left|\frac{T_1}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

D/C ratio $\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.46$

Design Case B: Vertical Vehicular Collision Load

Moment due to vertical vehicular collision load $M_v := \frac{F_v \cdot \frac{l_a}{L_v} \cdot 1\ ft}{\cos(22\ deg)} = 0.94\ ft \cdot kip$

$$M_{u1B} := M_{DC} + -M_v = -1.38\ kip \cdot ft$$

Note: Since Mu1B<Mu1A, design case A controls.

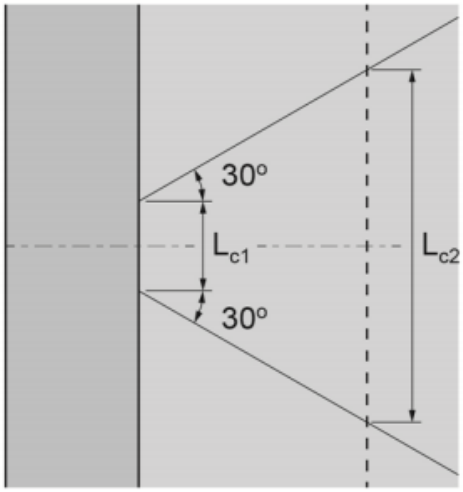
Design Case C: Dead and Live Loads at Strength Limit State

$$M_{u1C} := 1.25\ M_{DC} = -0.54\ kip \cdot ft$$

Note: Since $\frac{Mu1c}{\varphi_{str}} < \frac{Mu1a}{\varphi_{ext}}$, design case A controls.

Section 2: Overhang design at the Outside Face of the Web

Design Case A: Horizontal Vehicular Collision Load



As depicted in the figure above, loads distribute over larger lengths the farther the section is from the load application. Assume the distribution length, Lc, increases at a 30 degrees angle

Barrier height $H_{barrier} := 32\ in$

$$M_{c2} := \frac{M_s \cdot L_c}{L_c + 2 \cdot (L_{overhang} - L_{barrier}) \cdot \tan(30\ deg)} = 13.36\ ft \cdot kip$$

$$T_2 := \frac{T_1 \cdot L_c}{L_c + 2 \cdot (L_{overhang} - L_{barrier}) \cdot \tan(30\ deg) + 2 \cdot H_{barrier}} = 3.14\ kip$$

The moment due to the dead loads are:

$$M_{slab2} := \frac{-1.9}{\cos(22\ deg)}\ kip \cdot ft = -2.05\ kip \cdot ft$$

$$M_{barrier2} := \frac{-3.3}{\cos(22\ deg)}\ kip \cdot ft = -3.56\ kip \cdot ft$$

Extreme event II load combination:

$$M_{u2} := -M_{c2} + 1.0\ (M_{slab2} + M_{barrier2}) = -18.97\ kip \cdot ft$$

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5

$$d_{2\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #6

$$d_{2\_\#6} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top2}}{2} = 11.63\ in$$

Compression Block Depth

$$a_2 := \frac{\left(\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y + \left(A_{s\_top2'}\right) \cdot f_y\right)}{0.85\ f'_c \cdot b} = 1.73\ in$$

Unfactored resistance for top slab

$$M_{n\_top2} := -\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_2}{2}\right) - \left(A_{s\_top2'}\right) \cdot f_y \cdot \left(d_{2\_\#6} - \frac{a_2}{2}\right) = -79.46\ kip \cdot ft$$

Factored Resistance

$$\varphi_{ext} \cdot M_{n\_top2} = -79.46\ kip \cdot ft$$

Check for reinforcement in top slab

$$\text{if}\left(|M_{u2}| < |\varphi_{ext} \cdot M_{n\_top2}|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$

$$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.24$$

Bottom Transverse Reinforcement Tensile Strength Check

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement :

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom2} := A_{s\_bottom'} \cdot f_y = 24\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$

Check for reinforcement in bottom slab $\text{if}\left(\left|\frac{T_2}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.07$

Design Case B: Vertical Vehicular Collision Load

As per BDM Appendix 5-B6, for concrete parapets, the case of vertical collision never controls.

Design Case C: Dead and Live Loads at Strength Limit State

There is no moment due to the live loads at this location because of the sidewalk:

Overhang length $overhang := 4.5\ ft$

Exterior web thickness $t_w := 11.18\ in$

Multiple presence factor for 1 truck (AASHTO 3.6.1.1.2) $MPF := 1.2$

Live load per ft strip AASHTO 3.6.1.3.4 $LL := 1\ kip$

The moment due to the live loads are:

$$M_{LL} := -MPF \cdot \left(-\left(L_{barrier} + 1\ ft\right) + \left(overhang + \frac{t_w}{2}\right)\right) \cdot \frac{LL}{\cos\left(22\ deg\right)} = -4\ kip \cdot ft$$

Strength I load combination:

$$M_{u2\_strengthI} := 1.25\ \left(M_{slab2} + M_{barrier2}\right) + 1.75\ M_{LL} = -14.01\ kip \cdot ft$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5 $d_{2\_strengthI\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #6 $d_{2\_strengthI\_\#6} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top2}}{2} = 11.63\ in$

Compression Block Depth $a_{2\_strenghtI} := \frac{A_{s\_top1'} \cdot f_y + A_{s\_top2'} \cdot f_y}{0.85\ f'_c \cdot b} = 1.76\ in$

Unfactored resistance for top slab

$$M_{n\_top2\_strengthI} := -A_{s\_top1'} \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_{2\_strenghtI}}{2}\right) - A_{s\_top2'} \cdot f_y \cdot \left(d_{2\_\#6} - \frac{a_{2\_strenghtI}}{2}\right) = -80.76\ kip \cdot ft$$

Resistance Factor $\varphi_{str} := 0.9$

Factored Resistance $\varphi_{str} \cdot M_{n\_top2} = -71.52\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(|M_{u2}| < |\varphi_{ext} \cdot M_{n\_top2}|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.19$

Control of Cracking at Service Limit State

Service Limit State Demand at Interior Face of Barrier

$M_{u1\_service} := 1.0\ M_{DC} = -0.43\ kip \cdot ft$

Check cracking at the top of deck

Class 2 Exposure factor (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top1} := c_{top} + \frac{1}{2} \cdot \frac{A_{s\_top2'} \cdot d_{1\_\#6} + A_{s\_top1'} \cdot d_{1\_\#5}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)} = 5.3\ in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top1} := 1 + \frac{d_{c\_top1}}{0.7 \cdot \left(t_{overhang\_barrier} - d_{c\_top1}\right)} = 3.4$

Tension Reinforcement Ratio $\rho_{top1} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b \cdot \left(\frac{A_{s\_top2'} \cdot d_{1\_\#6} + A_{s\_top1'} \cdot d_{1\_\#5}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}\right)} = 0.02$

Young's Modulus of Steel $E_s := 29000\ ksi$

Unit Weight of Concrete $w_c := 0.155\ \frac{kip}{ft^3}$

Correction Factor for Source of Aggregate (5.4.2.4) $K_1 := 1$

Young's Modulus of Concrete (5.4.2.4) $E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} \cdot ksi = \left(4.9 \cdot 10^3\right)\ ksi$

$n := \frac{E_s}{E_c} = 5.91$

$k_{top1} := \sqrt{2\ n \cdot \rho_{top1} + \left(n \cdot \rho_{top1}\right)^2} - n \cdot \rho_{top1} = 0.4$ $j_{top1} := 1 - \frac{k_{top1}}{3} = 0.87$

Tensile stress in nonprestressed reinforcement at SLS $f_{ss\_top1} := \frac{-M_{u1\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right) \cdot j_{top1} \cdot \frac{A_{s\_top2'} \cdot d_{1\_\#6} + A_{s\_top1'} \cdot d_{1\_\#5}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}} = 0.71\ ksi$

Maximum reinforcement spacing for negative moment $s_{max\_top1} := \frac{700 \cdot \gamma_e \cdot in}{\beta_{s\_top1} \cdot \frac{f_{ss\_top1}}{ksi}} - 2 \cdot d_{c\_top1} = 206.85\ in$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top1}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top1}} = 0.03$

Service Limit State Demand at Exterior Face of Web

$M_{u2\_service} := \left(M_{slab2} + M_{barrier2}\right) + M_{LL} = -9.61\ \textit{kip} \cdot \textit{ft}$

Check cracking at the top of deck

Exposure factor for Class 2 Exposure Condition (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top2} := c_{top} + \frac{1}{2} \cdot \frac{\left(A_{s\_top1'} \cdot d_{bar\_top1} + A_{s\_top2'} \cdot d_{bar\_top2}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)} = 2.85\ \textit{in}$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top2} := 1 + \frac{d_{c\_top2}}{0.7 \cdot \left(t_{top\_fillet} - d_{c\_top2}\right)} = 1.35$

Tension Reinforcement Ratio $\rho_{top2} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b \cdot \frac{\left(A_{s\_top2'} \cdot d_{2\_strengthI\_\#6} + A_{s\_top1'} \cdot d_{2\_strengthI\_\#5}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}} = 0.01$

$k_{top2} := \sqrt{2\ n \cdot \rho_{top2} + \left(n \cdot \rho_{top2}\right)^2} - n \cdot \rho_{top2} = 0.3$ $j_{top2} := 1 - \frac{k_{top2}}{3} = 0.9$

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss\_top2} := \frac{-M_{u2\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right) \cdot j_{top2} \cdot \frac{\left(A_{s\_top2'} \cdot d_{2\_strengthI\_\#6} + A_{s\_top1'} \cdot d_{2\_strengthI\_\#5}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}} = 7.33\ \textit{ksi}$$

Maximum reinforcement spacing for negative moment $s_{max\_top2} := \frac{700 \cdot \gamma_e \cdot \textit{in}}{\beta_{s\_top2} \cdot \frac{f_{ss\_top2}}{\textit{ksi}}} - 2 \cdot d_{c\_top2} = 47.41\ \textit{in}$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top2}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top2}} = 0.13$

Summary

Top reinforcement: #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #6 @6"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$M_{u1A} = -25.54\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top1} = -26.24\ kip \cdot ft$	$\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.97$
@ Outside Face of the Web	$M_{u2} = -18.968\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top2} = -79.464\ kip \cdot ft$	$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.24$
@ Surface of Exterior Web	$M_{u2\_strengthI} = -14.01\ kip \cdot ft$	$\varphi_{str} \cdot M_{n\_top2\_strengthI} = -72.69\ kip \cdot ft$	$\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.19$

Maximum Spacing at SLS

	Maximum Spacing	Actual Spacing	D/C Ratio
@ Surface of Barrier	$s_{max\_top1} = 206.85\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top1}} = 0.03$
@ Surface of Exterior Web	$s_{max\_top2} = 47.41\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top2}} = 0.13$

Bottom reinforcement:#4 at 6"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$\frac{T_1}{2} = 4.71\ kip$	$\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$	$\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.46$
@ Surface of Exterior Web	$\frac{T_2}{2} = 1.57\ kip$	$\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$	$\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.07$

Transverse Overhang Design - Barrier Type F 32" - Interior Region

Purpose and Assumptions

1. Calculate the capacity of the top flange overhang of a concrete box girder in resisting transverse bending moments resulting from both permanent and live loads. The permanent loads include self weight, barriers, sidewalks, and utilities.
2. The capacities are calculated based on AASHTO LRFD Bridge Design Specification using the Approximate Elastic Method, or "Equivalent Strip Method" (AASHTO 4.6.2.1), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.
3. Bridge deck overhang is designed for three separate design cases (AASHTO A13.4.1):
   - Case 1 - Horizontal forces from vehicle collision load (Extreme Event II limit state)
   - Case 2 - Vertical force from vehicle collision load (Extreme Event II limit state)
   - Case 3 - Vertical Dead and Live Load at the overhang section (Strength I limit state)
4. For each case, the design is conducted for two sections (AASHTO A13.4.1):
   - Section 1 - At the interior face of the barrier
   - Section 2 - At the exterior face of the web (Overhang critical section - AASHTO BDM 4.6.2.1-6)

As per design criteria, a barrier of test level TL-4 is used. The design forces for traffic barriers are obtained from Table A13.2-1 in AASHTO. As per BDM Table 10.2.4-1, the three moment capacities needed in the following calculations for Type F 32 in TL-4 are given as:

Table 10.2.4-1

Barrier Impact Design Forces on Traffic Barrier & Deck Overhang

Parameters		Type F 32 in. (TL-4)		Single Slope 34 in. (TL-4)		Type F 42 in. (TL-4)		Single Slope 42 in. (TL-4)		Type F 42 in. (TL-5)		Single Slope 42 in. (TL-5)	
		Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*
Traffic Barrier Design	Average M_c (ft-kips/ft)	20.55	20.55	19.33	19.33	25.93	25.93	22.42	22.42	25.93	25.93	22.42	36.04
	M_c at Base (ft-kips/ft)	27.15	27.15	26.03	26.03	32.87	32.87	30.66	30.66	32.87	32.87	30.66	49.52
	M_w (ft-kips)	42.47	46.04	46.01	43.16	72.54	71.72	60.66	57.26	98.23	96.93	83.85	79.12
	L_c (ft)	8.62	4.76	9.30	4.81	10.77	5.32	10.63	5.21	15.05	9.39	14.99	8.87
	R_w (kips)	132.82	73.31	126.92	65.69	159.62	78.83	136.17	66.81	223.00	139.20	192.02	182.61
	F_t (kips)	54.00	54.00	54.00	54.00	54.00	54.00	54.00	54.00	124.00	124.00	124.00	124.00
Deck Overhang Design	1.2*F_t (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	148.80	148.80	148.80
	Design R_w (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	139.20	148.80	148.80
	R_w*H/(L_c+aH) (ft-kips/ft)**	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	37.79	23.69	42.11
	Design M_s (ft-kips/ft)	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	32.87	23.69	42.11
	Design T (kips/ft)	4.65	8.73	4.33	8.47	3.65	7.35	3.68	7.44	6.75	10.80	6.77	12.03
Deck to Barrier Reinforcement	A_s required (in^2/ft)	0.29	0.57	0.29	0.59	0.17	0.35	0.20	0.41	0.43	0.60	0.49	0.91
	A_s provided (in^2/ft)	0.41	0.62	0.41	0.62	0.41	0.62	0.41	0.62	0.59	0.89	0.59	0.97
	S_1 Bars	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#6 @ 9 in	#6 @ 6 in	#6 @ 8 in	#6 @ 5.5 in

Horizontal moment capacity (adjusted for bridge skew and transverse reinforcement orientation)
$$M_s := \frac{12.39}{\cos(22\ deg)}\ kip \cdot ft$$

Lever arm for vertical collision
$$l_a := 10.5\ in = 0.875\ ft$$

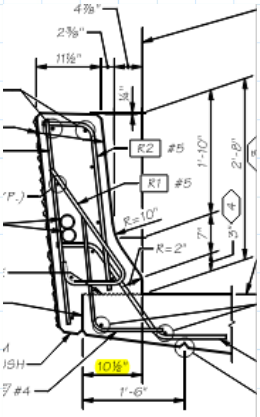
Longitudinal distribution of vertical force (AASHTO Table A13.2-1)
$$L_v := 18\ ft$$

Vertical design force (AASHTO Table A13.2-1)
$$F_v := 18\ kip$$

Tensile force from the barrier
$$T_1 := \frac{4.65}{\cos(22\ deg)}\ kip$$

Critical length of yield line
$$L_c := 8.62\ ft$$

Note: l_a is the horizontal distance between the interior face of the barrier to the centroid of the barrier. The centroid is approximately located at the edge of deck (see section below)



Material and Section Properties

Concrete strength
$$f'_c := 5\ ksi$$

Steel yield strength
$$f_y := 60\ ksi$$

Concrete top cover
$$c_{top} := 2.5\ in$$

Concrete bottom cover in deck and top and bottom covers in bottom slab
$$c_{bottom} := 1\ in$$

Overhang thickness at end	$t_{overhang} := 7\ in$
Top Deck Thickness	$t_{top} := 8.5\ in$
Overhang length	$L_{overhang} := 4.5\ ft$
Horizontal distance between the edge of deck and the inside face of the barrier	$L_{barrier} := 10.5\ in$
Top Deck Thickness at Fillet	$t_{top\_fillet} := 6\ in + t_{top} = 14.5\ in$
Overhang thickness at face of barrier	$t_{overhang\_barrier} := t_{overhang} + \left(\frac{t_{top\_fillet} - t_{overhang}}{L_{overhang}}\right) \cdot \left(L_{barrier}\right) = 8.46\ in$
Overhang thickness at outside face of web	$t_{overhang\_web} := t_{top\_fillet} = 14.5\ in$
Deck Width	$b := 12\ in$

Section 1: Overhang design at the interior face of the barrier

Design Case A: Horizontal Vehicular Collision Load

The deck overhang is designed to resist an axial tension force from vehicular collision acting simultaneously with the Dead Load moment.

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

$$M_{slab} := \frac{-0.04}{\cos(22\ deg)}\ kip \cdot ft = -0.04\ kip \cdot ft$$

$$M_{barrier} := \frac{-0.36}{\cos(22\ deg)}\ kip \cdot ft = -0.39\ kip \cdot ft$$

$$M_{DC} := M_{slab} + M_{barrier} = -0.43\ kip \cdot ft$$

$$M_{u1A} := -M_s + 1.0\ M_{DC} = -13.79\ kip \cdot ft$$

Note: Assume #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #5 @12"

For #5@ 6" (typ. top transverse reinforcement):

$A_{s\_top1} := 0.31\ in^2$ $s_{top1} := 6\ in$ $d_{bar\_top1} := 0.625\ in$

For additional short hooked #5@ 12" :

$A_{s\_top2} := 0.31\ in^2$ $s_{top2} := 12\ in$ $d_{bar\_top2} := 0.625\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5	$d_{1\_\#5} := t_{overhang\_barrier} - c_{top} - \frac{d_{bar\_top1}}{2} = 5.65\ in$
Resistance Factor for Extreme Load Combination	$\varphi_{ext} := 1$
Area of reinforcement per ft	$A_{s\_top1'} := \frac{A_{s\_top1}}{\frac{s_{top1}}{in}} = 0.62\ in^2$ $A_{s\_top2'} := \frac{A_{s\_top2}}{\frac{s_{top2}}{in}} = 0.31\ in^2$

The development length of #5 is 10.2". The calculations are at spreadsheet "development length"

Development length of #5 $l_{d\_5} := 10.2\ in$

Note: the development length is restrained by the distance from the deck edge to the interior surface of the barrier and concrete cover

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #5 $f_{y\_M5} := f_y \cdot \frac{min\left(l_t, l_{d\_5}\right)}{l_{d\_5}} = 50\ ksi$

Compression Block Depth $a_1 := \frac{\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5} + \left(A_{s\_top2'}\right) \cdot f_{y\_M5}}{0.85\ f'_c \cdot b} = 0.86\ in$

Unfactored resistance for top slab

$$M_{n\_top1} := -\left(\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5}\right) \cdot \left(d_{1\_\#5} - \frac{a_1}{2}\right) - \left(\left(A_{s\_top2'}\right) \cdot f_{y\_M5}\right) \cdot \left(d_{1\_\#5} - \frac{a_1}{2}\right) = -19.12\ kip \cdot ft$$

Factored Resistance $\varphi_{ext} \cdot M_{n\_top1} = -19.12\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(\left|M_{u1A}\right| < \left|\varphi_{ext} \cdot M_{n\_top1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.72$

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement :

Note: Assume #4 bars at 6 inches for bottom reinforcement

$A_{s\_bottom} := 0.2 \cdot in^2$ $s_{bottom} := 6\ in$ $d_{bar\_bottom} := 0.5\ in$

Development length of #4 $l_{d\_4} := 20\ in$

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #4 $f_{y\_M4} := f_y \cdot \frac{min\left(l_{d\_4}, l_t\right)}{l_{d\_4}} = 25.5\ ksi$

Area of Tension Transverse Reinforcement $A_{s\_bottom'} := \frac{\left(A_{s\_bottom}\right)}{\left(\frac{s_{bottom}}{\frac{in}{12}}\right)} = 0.4\ in^2$

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom1} := A_{s\_bottom'} \cdot f_{y\_M4} = 10.2\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$

Check for reinforcement in top slab $\text{if}\left(\left|\frac{T_1}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

D/C ratio $\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.25$

Design Case B: Vertical Vehicular Collision Load

Moment due to vertical vehicular collision load $M_v := \frac{F_v \cdot \frac{l_a}{L_v} \cdot 1\ ft}{\cos(22\ deg)} = 0.94\ ft \cdot kip$

$$M_{u1B} := M_{DC} + -M_v = -1.38\ kip \cdot ft$$

Note: Since Mu1B<Mu1A, design case A controls.

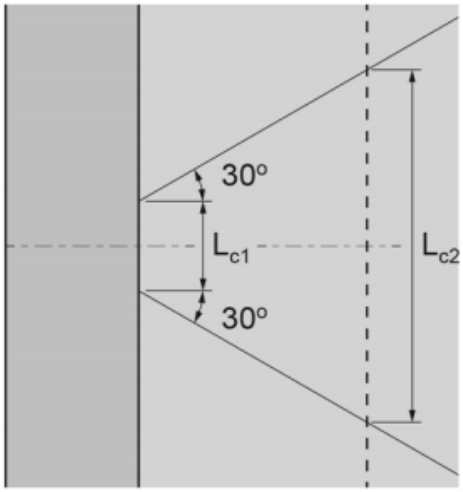
Design Case C: Dead and Live Loads at Strength Limit State

$$M_{u1C} := 1.25\ M_{DC} = -0.54\ kip \cdot ft$$

Note: Since $\frac{Mu1c}{\varphi_{str}} < \frac{Mu1a}{\varphi_{ext}}$, design case A controls.

Section 2: Overhang design at the Outside Face of the Web

Design Case A: Horizontal Vehicular Collision Load



As depicted in the figure above, loads distribute over larger lengths the farther the section is from the load application. Assume the distribution length, Lc, increases at a 30 degrees angle

Barrier height $H_{barrier} := 32\ in$

$$M_{c2} := \frac{M_s \cdot L_c}{L_c + 2 \cdot (L_{overhang} - L_{barrier}) \cdot \tan(30\ deg)} = 9\ ft \cdot kip$$

$$T_2 := \frac{T_1 \cdot L_c}{L_c + 2 \cdot (L_{overhang} - L_{barrier}) \cdot \tan(30\ deg) + 2 \cdot H_{barrier}} = 2.38\ kip$$

The moment due to the dead loads are:

$$M_{slab2} := \frac{-1.9}{\cos(22\ deg)}\ kip \cdot ft = -2.05\ kip \cdot ft$$

$$M_{barrier2} := \frac{-3.3}{\cos(22\ deg)}\ kip \cdot ft = -3.56\ kip \cdot ft$$

Extreme event II load combination:

$$M_{u2} := -M_{c2} + 1.0\ (M_{slab2} + M_{barrier2}) = -14.6\ kip \cdot ft$$

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5

$$d_{2\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$$

Compression Block Depth

$$a_2 := \frac{\left(\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y + \left(A_{s\_top2'}\right) \cdot f_y\right)}{0.85\ f'_c \cdot b} = 1.07\ in$$

Unfactored resistance for top slab

$$M_{n\_top2} := -\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_2}{2}\right) - \left(A_{s\_top2'}\right) \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_2}{2}\right) = -50.75\ kip \cdot ft$$

Factored Resistance

$$\varphi_{ext} \cdot M_{n\_top2} = -50.75\ kip \cdot ft$$

Check for reinforcement in top slab

$$\text{if}\left(|M_{u2}| < |\varphi_{ext} \cdot M_{n\_top2}|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$

$$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.29$$

Bottom Transverse Reinforcement Tensile Strength Check

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement :

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom2} := A_{s\_bottom'} \cdot f_y = 24\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$

Check for reinforcement in bottom slab $\text{if}\left(\left|\frac{T_2}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.05$

Design Case B: Vertical Vehicular Collision Load

As per BDM Appendix 5-B6, for concrete parapets, the case of vertical collision never controls.

Design Case C: Dead and Live Loads at Strength Limit State

There is no moment due to the live loads at this location because of the sidewalk:

Overhang length $overhang := 4.5\ ft$

Exterior web thickness $t_w := 11.18\ in$

Multiple presence factor for 1 truck (AASHTO 3.6.1.1.2) $MPF := 1.2$

Live load per ft strip AASHTO 3.6.1.3.4 $LL := 1\ kip$

The moment due to the live loads are:

$$M_{LL} := -MPF \cdot \left(-\left(L_{barrier} + 1\ ft\right) + \left(overhang + \frac{t_w}{2}\right)\right) \cdot \frac{LL}{\cos\left(22\ deg\right)} = -4\ kip \cdot ft$$

Strength I load combination:

$$M_{u2\_strengthI} := 1.25\ \left(M_{slab2} + M_{barrier2}\right) + 1.75\ M_{LL} = -14.01\ kip \cdot ft$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5 $d_{2\_strengthI\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$

Compression Block Depth $a_{2\_strenghtI} := \frac{A_{s\_top1'} \cdot f_y + A_{s\_top2'} \cdot f_y}{0.85\ f'_c \cdot b} = 1.09\ in$

Unfactored resistance for top slab

$$M_{n\_top2\_strengthI} := -A_{s\_top1'} \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_{2\_strenghtI}}{2}\right) - A_{s\_top2'} \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_{2\_strenghtI}}{2}\right) = -51.8\ kip \cdot ft$$

Resistance Factor $\varphi_{str} := 0.9$

Factored Resistance $\varphi_{str} \cdot M_{n\_top2} = -45.67\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(|M_{u2}| < |\varphi_{ext} \cdot M_{n\_top2}|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.3$

Control of Cracking at Service Limit State

Service Limit State Demand at Interior Face of Barrier

$M_{u1\_service} := 1.0\ M_{DC} = -0.43\ kip \cdot ft$

Check cracking at the top of deck

Class 2 Exposure factor (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top1} := c_{top} + \frac{1}{2} \cdot \left(d_{1\_\#5}\right) = 5.32\ in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top1} := 1 + \frac{d_{c\_top1}}{0.7 \cdot \left(t_{overhang\_barrier} - d_{c\_top1}\right)} = 3.43$

Tension Reinforcement Ratio $\rho_{top1} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b \cdot \left(d_{1\_\#5}\right)} = 0.01$

Young's Modulus of Steel $E_s := 29000\ ksi$

Unit Weight of Concrete $w_c := 0.155\ \frac{kip}{ft^3}$

Correction Factor for Source of Aggregate (5.4.2.4) $K_1 := 1$

Young's Modulus of Concrete (5.4.2.4) $E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} \cdot ksi = \left(4.9 \cdot 10^3\right)\ ksi$

$n := \frac{E_s}{E_c} = 5.91$

$k_{top1} := \sqrt{2\ n \cdot \rho_{top1} + \left(n \cdot \rho_{top1}\right)^2} - n \cdot \rho_{top1} = 0.33$ $j_{top1} := 1 - \frac{k_{top1}}{3} = 0.89$

Tensile stress in nonprestressed reinforcement at SLS $f_{ss\_top1} := \frac{-M_{u1\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right) \cdot j_{top1} \cdot d_{1\_\#5}} = 1.11\ ksi$

Maximum reinforcement spacing for negative moment $s_{max\_top1} := \frac{700 \cdot \gamma_e \cdot in}{\beta_{s\_top1} \cdot \frac{f_{ss\_top1}}{ksi}} - 2 \cdot d_{c\_top1} = 127.72\ in$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top1}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top1}} = 0.05$

Service Limit State Demand at Exterior Face of Web

$M_{u2\_service} := \left(M_{slab2} + M_{barrier2}\right) + M_{LL} = -9.61\ \textit{kip}\cdot\textit{ft}$

Check cracking at the top of deck

Exposure factor for Class 2 Exposure Condition (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top2} := c_{top} + \frac{1}{2}\cdot\frac{\left(A_{s\_top1'}\cdot d_{bar\_top1} + A_{s\_top2'}\cdot d_{bar\_top2}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)} = 2.81\ \textit{in}$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top2} := 1 + \frac{d_{c\_top2}}{0.7\cdot\left(t_{top\_fillet} - d_{c\_top2}\right)} = 1.34$

Tension Reinforcement Ratio $\rho_{top2} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b\cdot d_{2\_strengthI\_\#5}} = 6.63\cdot 10^{-3}$

$k_{top2} := \sqrt{2\ n\cdot\rho_{top2} + \left(n\cdot\rho_{top2}\right)^2} - n\cdot\rho_{top2} = 0.24$ $j_{top2} := 1 - \frac{k_{top2}}{3} = 0.92$

Tensile stress in nonprestressed reinforcement at SLS

$f_{ss\_top2} := \frac{-M_{u2\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)\cdot j_{top2}\cdot d_{2\_strengthI\_\#5}} = 11.55\ \textit{ksi}$

Maximum reinforcement spacing for negative moment $s_{max\_top2} := \frac{700\cdot\gamma_e\cdot\textit{in}}{\beta_{s\_top2}\cdot\frac{f_{ss\_top2}}{\textit{ksi}}} - 2\cdot d_{c\_top2} = 28.21\ \textit{in}$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top2}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top2}} = 0.21$

Summary

Top reinforcement: #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #5 @12"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$M_{u1A} = -13.79\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top1} = -19.12\ kip \cdot ft$	$\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.72$
@ Outside Face of the Web	$M_{u2} = -14.603\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top2} = -50.75\ kip \cdot ft$	$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.29$
@ Surface of Exterior Web	$M_{u2\_strengthI} = -14.01\ kip \cdot ft$	$\varphi_{str} \cdot M_{n\_top2\_strengthI} = -46.62\ kip \cdot ft$	$\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.3$

Maximum Spacing at SLS

	Maximum Spacing	Actual Spacing	D/C Ratio
@ Surface of Barrier	$s_{max\_top1} = 127.72\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top1}} = 0.05$
@ Surface of Exterior Web	$s_{max\_top2} = 28.21\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top2}} = 0.21$

Bottom reinforcement:#4 at 6"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$\frac{T_1}{2} = 2.51\ kip$	$\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$	$\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.25$
@ Surface of Exterior Web	$\frac{T_2}{2} = 1.19\ kip$	$\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$	$\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.05$

Transverse Overhang Design - Pedestrian Barrier (End Region)

Purpose and Assumptions

1. Calculate the capacity of the top flange overhang of a concrete box girder in resisting transverse bending moments resulting from both permanent and live loads. The permanent loads include self weight, barriers, sidewalks, and utilities.
2. The capacities are calculated based on AASHTO LRFD Bridge Design Specification using the Approximate Elastic Method, or "Equivalent Strip Method" (AASHTO 4.6.2.1), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.
3. Bridge deck overhang is designed for three separate design cases (AASHTO A13.4.1):
   - Case 1 - Horizontal forces from vehicle collision load (Extreme Event II limit state)
   - Case 2 - Vertical force from vehicle collision load (Extreme Event II limit state)
   - Case 3 - Vertical Dead and Live Load at the overhang section (Strength I limit state)
4. For each case, the design is conducted for two sections (AASHTO A13.4.1):
   - Section 1 - At the interior face of the barrier
   - Section 2 - At the exterior face of the web (Overhang critical section - AASHTO BDM 4.6.2.1-6)

As per design criteria, the design forces are obtained from BDM Table 10.2.4-1. However, since pedestrian barriers are not covered, the parameters for a similar barrier of test level TL-4 are used instead. The single slope 34"traffic barrier has similar shape to the 32" pedestrian barrier used in the project, but it is heavier and taller than the 32" pedestrian barrier. Therefore it is conservative to design the overhang based on the parameters for the 34" single slope traffic barrier. The three moment capacities needed in the following calculations are given as:

Table 10.2.4-1

Barrier Impact Design Forces on Traffic Barrier & Deck Overhang

Parameters		Type F 32 in. (TL-4)		Single Slope 34 in. (TL-4)		Type F 42 in. (TL-4)		Single Slope 42 in. (TL-4)		Type F 42 in. (TL-5)		Single Slope 42 in. (TL-5)	
		Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*
Traffic Barrier Design	Average M_c (ft-kips/ft)	20.55	20.55	19.33	19.33	25.93	25.93	22.42	22.42	25.93	25.93	22.42	36.04
	M_c at Base (ft-kips/ft)	27.15	27.15	26.03	26.03	32.87	32.87	30.66	30.66	32.87	32.87	30.66	49.52
	M_w (ft-kips)	42.47	46.04	46.01	43.16	72.54	71.72	60.66	57.26	98.23	96.93	83.85	79.12
	L_c (ft)	8.62	4.76	9.30	4.81	10.77	5.32	10.63	5.21	15.05	9.39	14.99	8.87
	R_w (kips)	132.82	73.31	126.92	65.69	159.62	78.83	136.17	66.81	223.00	139.20	192.02	182.61
	F_t (kips)	54.00	54.00	54.00	54.00	54.00	54.00	54.00	54.00	124.00	124.00	124.00	124.00
Deck Overhang Design	1.2*F_t (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	148.80	148.80	148.80
	Design R_w (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	139.20	148.80	148.80
	R_w*H/(L_c+aH) (ft-kips/ft)**	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	37.79	23.69	42.11
	Design M_s (ft-kips/ft)	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	32.87	23.69	42.11
	Design T (kips/ft)	4.65	8.73	4.33	8.47	3.65	7.35	3.68	7.44	6.75	10.80	6.77	12.03
Deck to Barrier Reinforcement	A_s required (in^2/ft)	0.29	0.57	0.29	0.59	0.17	0.35	0.20	0.41	0.43	0.60	0.49	0.91
	A_s provided (in^2/ft)	0.41	0.62	0.41	0.62	0.41	0.62	0.41	0.62	0.59	0.89	0.59	0.97
	S_1 Bars	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#6 @ 9 in	#6 @ 6 in	#6 @ 8 in	#6 @ 5.5 in

Horizontal moment capacity (adjusted for bridge skew and transverse reinforcement orientation)

$$M_s := \frac{24.01}{\cos(22\ deg)}\ kip \cdot ft = 25.9\ kip \cdot ft$$

Lever arm for vertical collision

$$l_a := 10.5\ in - 4\ in = 0.54\ ft$$

Longitudinal distribution of vertical force (AASHTO Table A13.2-1)

$$L_v := 18\ ft$$

Vertical design force (AASHTO Table A13.2-1)

$$F_v := 18\ kip$$

Tensile force from the barrier

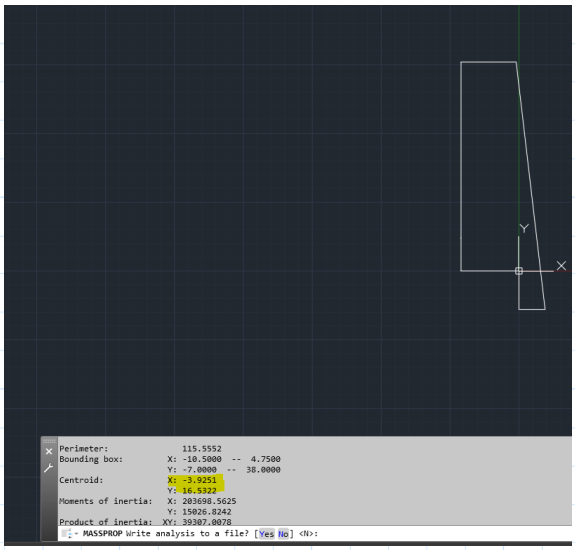
$$T_1 := \frac{8.47}{\cos(22\ deg)}\ kip = 9.14\ kip$$

Critical length of yield line

$$L_c := 4.81\ ft$$

Note: l_a is the horizontal distance between the interior face of the barrier to the centroid of the barrier. The centroid is located at approximately 4" from edge of deck (measured using AutoCad, see screenshot on next sheet)

Material and Section Properties



Concrete strength $f'_c := 5\ ksi$

Steel yield strength $f_y := 60\ ksi$

Concrete top cover $c_{top} := 2.5\ in$

Concrete bottom cover in deck and top and bottom covers in bottom slab $c_{bottom} := 1\ in$

Overhang thickness at end $t_{overhang} := 7\ in$

Top Deck Thickness $t_{top} := 8.5\ in$

Overhang length $L_{overhang} := 4.5\ ft$

Horizontal distance between the edge of deck and the inside face of the barrier $L_{barrier} := 10.5\ in$

Top Deck Thickness at Fillet $t_{top\_fillet} := 6\ in + t_{top} = 14.5\ in$

Overhang thickness at face of barrier $t_{overhang\_barrier} := t_{overhang} + \left(\frac{t_{top\_fillet} - t_{overhang}}{L_{overhang}}\right) \cdot \left(L_{barrier}\right) = 8.46\ in$

Overhang thickness at outside face of web $t_{overhang\_web} := t_{top\_fillet} = 14.5\ in$

Deck Width $b := 12\ in$

Section 1: Overhang design at the interior face of the barrier

Design Case A: Horizontal Vehicular Collision Load

The deck overhang is designed to resist an axial tension force from vehicular collision acting simultaneously with the Dead Load moment.

$$M_{slab} := \frac{-0.01}{\cos(22\ deg)}\ kip \cdot ft$$

$$M_{barrier} := \frac{-0.17}{\cos(22\ deg)}\ kip \cdot ft$$

$$M_{DC} := M_{slab} + M_{barrier} = -0.19\ kip \cdot ft$$

$$M_{u1A} := -M_s + 1.0\ M_{DC} = -26.09\ kip \cdot ft$$

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

Note: Assume #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #6 @ 6"
For #5 @ 6" (typ. top transverse reinforcement):

$A_{s\_top1} := 0.31\ in^2$ $s_{top1} := 6\ in$ $d_{bar\_top1} := 0.625\ in$

For additional short hooked #6 @6":

$A_{s\_top2} := 0.44\ in^2$ $s_{top2} := 6\ in$ $d_{bar\_top2} := 0.75\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5 $d_{1\_\#5} := t_{overhang\_barrier} - c_{top} - \frac{d_{bar\_top1}}{2} = 5.65\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #6

$$d_{1\_\#6} := t_{overhang\_barrier} - c_{top} - \frac{d_{bar\_top2}}{2} = 5.58\ in$$

Resistance Factor for Extreme Load Combination

$$\varphi_{ext} := 1$$

Area of reinforcement per ft

$$A_{s\_top1'} := \frac{A_{s\_top1}}{\frac{s_{top1}}{12\ in}} = 0.62\ in^2 \qquad A_{s\_top2'} := \frac{A_{s\_top2}}{\frac{s_{top2}}{12\ in}} = 0.88\ in^2$$

The development length of #5 is 10.2". The development length of #6 is 12.2". The calculations are at spreadsheet "development length"

Development length of #5

$$l_{d\_5} := 10.2\ in$$

Development length of #6

$$l_{d\_6} := 12.2\ in$$

Note: the development length is restrained by the distance from the deck edge to the interior surface of the barrier and concrete cover

Maximum available development length

$$l_t := L_{barrier} - 2\ in = 8.5\ in$$

Modified stress of #5

$$f_{y\_M5} := f_y \cdot \frac{min(l_t, l_{d\_5})}{l_{d\_5}} = 50\ ksi$$

Modified stress of #6

$$f_{y\_M6} := f_y \cdot \frac{min(l_t, l_{d\_6})}{l_{d\_6}} = 41.8\ ksi$$

Compression Block Depth

$$a_1 := \frac{\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5} + \left(A_{s\_top2'}\right) \cdot f_{y\_M6}}{0.85\ f'_c \cdot b} = 1.24\ in$$

Unfactored resistance for top slab

$$M_{n\_top1} := -\left(\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5}\right) \cdot \left(d_{1\_\#5} - \frac{a_1}{2}\right) - \left(\left(A_{s\_top2'}\right) \cdot f_{y\_M6}\right) \cdot \left(d_{1\_\#6} - \frac{a_1}{2}\right) = -26.29\ kip \cdot ft$$

Factored Resistance

$$\varphi_{ext} \cdot M_{n\_top1} = -26.29\ kip \cdot ft$$

Check for reinforcement in top slab

$$\text{if}\left(\left|M_{u1A}\right| < \left|\varphi_{ext} \cdot M_{n\_top1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$

$$\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.99$$

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement:

Note: Assume #4 bars at 6 inches for bottom reinforcement

$A_{s\_bottom} := 0.2 \cdot in^2$ $s_{bottom} := 6\ in$ $d_{bar\_bottom} := 0.5\ in$

Development length of #4 $l_{d\_4} := 20\ in$

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #4 $f_{y\_M4} := f_y \cdot \frac{min\left(l_t, l_{d\_4}\right)}{l_{d\_4}} = 25.5\ ksi$

Area of Tension Transverse Reinforcement $A_{s\_bottom'} := \frac{\left(A_{s\_bottom}\right)}{\left(\frac{s_{bottom}}{\frac{in}{12}}\right)} = 0.4\ in^2$

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom1} := A_{s\_bottom'} \cdot f_{y\_M4} = 10.2\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$

Check for reinforcement in top slab $\text{if}\left(\left|\frac{T_1}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

D/C ratio $\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.448$

Design Case B: Vertical Vehicular Collision Load

Moment due to vertical vehicular collision load $M_v := \frac{F_v \cdot \frac{l_a}{L_v} \cdot 1\ ft}{\cos\left(22\ deg\right)} = 0.58\ ft \cdot kip$

$M_{u1B} := M_{DC} + -M_v = -0.778\ kip \cdot ft$

Note: Since Mu1B<Mu1A, design case A controls.

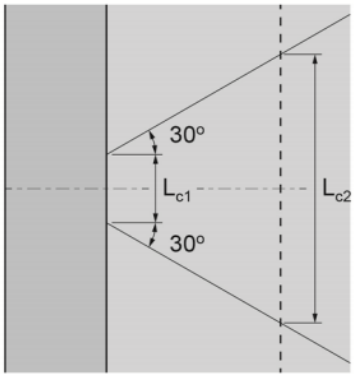
Design Case C: Dead and Live Loads at Strength Limit State

$M_{u1C} := 1.25\ M_{DC} = -0.24\ kip \cdot ft$

Note: Since $\frac{Mu1c}{\varphi_{str}} < \frac{Mu1a}{\varphi_{ext}}$, design case A controls.

Section 2: Overhang design at the Outside Face of the Web

Design Case A: Horizontal Vehicular Collision Load



As depicted in the figure above, loads distribute over larger lengths the farther the section is from the load application. Assume the distribution length, Lc, increases at a 30 degrees angle

Barrier height $H_{barrier} := 32\ in$

$$M_{c2} := \frac{M_s \cdot L_c}{L_c + 2 \cdot \left(L_{overhang} - L_{barrier}\right) \cdot \tan\left(30\ deg\right)} = 13.85\ ft \cdot kip$$

$$T_2 := \frac{T_1 \cdot L_c}{L_c + 2 \cdot \left(L_{overhang} - L_{barrier}\right) \cdot \tan\left(30\ deg\right) + 2 \cdot H_{barrier}} = 3.07\ kip$$

The moment due to the dead loads are:

$$M_{slab2} := \frac{-1.9}{\cos\left(22\ deg\right)}\ kip \cdot ft = -2.05\ kip \cdot ft$$

$$M_{barrier2} := \frac{-3.3}{\cos\left(22\ deg\right)}\ kip \cdot ft = -3.56\ kip \cdot ft$$

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

Extreme event II load combination:

$$M_{u2} := -M_{c2} + 1.0\ \left(M_{slab2} + M_{barrier2}\right) = -19.45\ kip \cdot ft$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5

$$d_{2\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #6

$$d_{2\_\#6} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top2}}{2} = 11.63\ in$$

Compression Block Depth

$$a_2 := \frac{\left(\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y + \left(A_{s\_top2'}\right) \cdot f_y\right)}{0.85\ f'_c \cdot b} = 1.73\ in$$

Unfactored resistance for top slab

$$M_{n\_top2} := -\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_2}{2}\right) - \left(A_{s\_top2'}\right) \cdot f_y \cdot \left(d_{2\_\#6} - \frac{a_2}{2}\right) = -79.49\ kip \cdot ft$$

Factored Resistance $\varphi_{ext} \cdot M_{n\_top2} = -79.49\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(\left|M_{u2}\right| < \left|\varphi_{ext} \cdot M_{n\_top2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.24$

Bottom Transverse Reinforcement Tensile Strength Check

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement :

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom2} := A_{s\_bottom'} \cdot f_y = 24\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$

Check for reinforcement in bottom slab $\text{if}\left(\left|\frac{T_2}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.064$

Design Case B: Vertical Vehicular Collision Load

As per BDM Appendix 5-B6, for concrete parapets, the case of vertical collision never controls.

Design Case C: Dead and Live Loads at Strength Limit State

There is no moment due to the live loads at this location because of the sidewalk:

$M_{LL} := 0\ kip \cdot ft$

Strength I load combination:

$$M_{u2\_strengthI} := 1.25\ \left(M_{slab2} + M_{barrier2}\right) + 1.75\ M_{LL} = -7.01\ kip \cdot ft$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5 $d_{2\_strengthI\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #6 $d_{2\_strengthI\_\#6} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top2}}{2} = 11.63\ in$

Compression Block Depth $a_{2\_strenghtI} := \frac{A_{s\_top1'} \cdot f_y + A_{s\_top2'} \cdot f_y}{0.85\ f'_c \cdot b} = 1.76\ in$

Unfactored resistance for top slab

$$M_{n\_top2\_strengthI} := -A_{s\_top1'} \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_{2\_strenghtI}}{2}\right) - A_{s\_top2'} \cdot f_y \cdot \left(d_{2\_\#6} - \frac{a_{2\_strenghtI}}{2}\right) = -80.76\ kip \cdot ft$$

Resistance Factor $\varphi_{str} := 0.9$

Factored Resistance $\varphi_{str} \cdot M_{n\_top2} = -71.54\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(|M_{u2}| < |\varphi_{ext} \cdot M_{n\_top2}|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.096$

Control of Cracking at Service Limit State

Service Limit State Demand at Interior Face of Barrier

$M_{u1\_service} := 1.0\ M_{DC} = -0.19\ kip \cdot ft$

Check cracking at the top of deck

Class 2 Exposure factor (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top1} := c_{top} + \frac{1}{2} \cdot \frac{A_{s\_top2'} \cdot d_{1\_\#6} + A_{s\_top1'} \cdot d_{1\_\#5}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)} = 5.3\ in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top1} := 1 + \frac{d_{c\_top1}}{0.7 \cdot \left(t_{overhang\_barrier} - d_{c\_top1}\right)} = 3.4$

Tension Reinforcement Ratio $\rho_{top1} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b \cdot \left(\frac{A_{s\_top2'} \cdot d_{1\_\#6} + A_{s\_top1'} \cdot d_{1\_\#5}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}\right)} = 0.02$

Young's Modulus of Steel $E_s := 29000\ ksi$

Unit Weight of Concrete $w_c := 0.155\ \frac{kip}{ft^3}$

Correction Factor for Source of Aggregate (5.4.2.4) $K_1 := 1$

Young's Modulus of Concrete (5.4.2.4) $E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f'_c}{ksi}\right)^{0.33} \cdot ksi = \left(4.9 \cdot 10^3\right)\ ksi$

$n := \frac{E_s}{E_c} = 5.91$

$k_{top1} := \sqrt{2\ n \cdot \rho_{top1} + \left(n \cdot \rho_{top1}\right)^2} - n \cdot \rho_{top1} = 0.4$ $j_{top1} := 1 - \frac{k_{top1}}{3} = 0.87$

Tensile stress in nonprestressed reinforcement at SLS $f_{ss\_top1} := \frac{-M_{u1\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right) \cdot j_{top1} \cdot \frac{A_{s\_top2'} \cdot d_{1\_\#6} + A_{s\_top1'} \cdot d_{1\_\#5}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}} = 0.32\ ksi$

Maximum reinforcement spacing for negative moment $s_{max\_top1} := \frac{700 \cdot \gamma_e \cdot in}{\beta_{s\_top1} \cdot \frac{f_{ss\_top1}}{ksi}} - 2 \cdot d_{c\_top1} = 472.63\ in$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top1}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top1}} = 0.013$

Service Limit State Demand at Exterior Face of Web

$M_{u2\_service} := \left(M_{slab2} + M_{barrier2}\right) + M_{LL} = -5.608\ \textit{kip} \cdot \textit{ft}$

Check cracking at the top of deck

Exposure factor for Class 2 Exposure Condition (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top2} := c_{top} + \frac{1}{2} \cdot \frac{\left(A_{s\_top1'} \cdot d_{bar\_top1} + A_{s\_top2'} \cdot d_{bar\_top2}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)} = 2.85\ \textit{in}$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top2} := 1 + \frac{d_{c\_top2}}{0.7 \cdot \left(t_{top\_fillet} - d_{c\_top2}\right)} = 1.35$

Tension Reinforcement Ratio $\rho_{top2} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b \cdot \frac{\left(A_{s\_top1'} \cdot d_{2\_strengthI\_\#5} + A_{s\_top2'} \cdot d_{2\_strengthI\_\#6}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}} = 0.01$

$k_{top2} := \sqrt{2\ n \cdot \rho_{top2} + \left(n \cdot \rho_{top2}\right)^2} - n \cdot \rho_{top2} = 0.298$ $j_{top2} := 1 - \frac{k_{top2}}{3} = 0.9$

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss\_top2} := \frac{-M_{u2\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right) \cdot j_{top2} \cdot \frac{\left(A_{s\_top1'} \cdot d_{2\_strengthI\_\#5} + A_{s\_top2'} \cdot d_{2\_strengthI\_\#6}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)}} = 4.28\ \textit{ksi}$$

Maximum reinforcement spacing for negative moment $s_{max\_top2} := \frac{700 \cdot \gamma_e \cdot \textit{in}}{\beta_{s\_top2} \cdot \frac{f_{ss\_top2}}{\textit{ksi}}} - 2 \cdot d_{c\_top2} = 85.29\ \textit{in}$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top2}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top2}} = 0.07$

Summary

Top reinforcement: #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #6 @6"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$M_{u1A} = -26.09\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top1} = -26.29\ kip \cdot ft$	$\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.99$
@ Outside Face of the Web	$M_{u2} = -19.45\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top2} = -79.49\ kip \cdot ft$	$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.24$
@ Surface of Exterior Web	$M_{u2\_strengthI} = -7.01\ kip \cdot ft$	$\varphi_{str} \cdot M_{n\_top2\_strengthI} = -72.69\ kip \cdot ft$	$\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.1$

Maximum Spacing at SLS

	Maximum Spacing	Actual Spacing	D/C Ratio
@ Surface of Barrier	$s_{max\_top1} = 472.63\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top1}} = 0.01$
@ Surface of Exterior Web	$s_{max\_top2} = 85.29\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top2}} = 0.07$

Bottom reinforcement:#4 at 6"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$\frac{T_1}{2} = 4.57\ kip$	$\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$	$\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.45$
@ Surface of Exterior Web	$\frac{T_2}{2} = 1.53\ kip$	$\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$	$\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.06$

Transverse Overhang Design - Pedestrian Barrier (Interior Region)

Purpose and Assumptions

1. Calculate the capacity of the top flange overhang of a concrete box girder in resisting transverse bending moments resulting from both permanent and live loads. The permanent loads include self weight, barriers, sidewalks, and utilities.
2. The capacities are calculated based on AASHTO LRFD Bridge Design Specification using the Approximate Elastic Method, or "Equivalent Strip Method" (AASHTO 4.6.2.1), in which the deck is divided into transverse strips, assumed to be supported on rigid supports at the center of the girders.
3. Bridge deck overhang is designed for three separate design cases (AASHTO A13.4.1):
   - Case 1 - Horizontal forces from vehicle collision load (Extreme Event II limit state)
   - Case 2 - Vertical force from vehicle collision load (Extreme Event II limit state)
   - Case 3 - Vertical Dead and Live Load at the overhang section (Strength I limit state)
4. For each case, the design is conducted for two sections (AASHTO A13.4.1):
   - Section 1 - At the interior face of the barrier
   - Section 2 - At the exterior face of the web (Overhang critical section - AASHTO BDM 4.6.2.1-6)

As per design criteria, the design forces are obtained from BDM Table 10.2.4-1. However, since pedestrian barriers are not covered, the parameters for a similar barrier of test level TL-4 are used instead. The single slope 34"traffic barrier has similar shape to the 32" pedestrian barrier used in the project, but it is heavier and taller than the 32" pedestrian barrier. Therefore it is conservative to design the overhang based on the parameters for the 34" single slope traffic barrier. The three moment capacities needed in the following calculations are given as:

Table 10.2.4-1

Barrier Impact Design Forces on Traffic Barrier & Deck Overhang

Parameters		Type F 32 in. (TL-4)		Single Slope 34 in. (TL-4)		Type F 42 in. (TL-4)		Single Slope 42 in. (TL-4)		Type F 42 in. (TL-5)		Single Slope 42 in. (TL-5)	
		Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*	Interior	End*
Traffic Barrier Design	Average M_c (ft-kips/ft)	20.55	20.55	19.33	19.33	25.93	25.93	22.42	22.42	25.93	25.93	22.42	36.04
	M_c at Base (ft-kips/ft)	27.15	27.15	26.03	26.03	32.87	32.87	30.66	30.66	32.87	32.87	30.66	49.52
	M_w (ft-kips)	42.47	46.04	46.01	43.16	72.54	71.72	60.66	57.26	98.23	96.93	83.85	79.12
	L_c (ft)	8.62	4.76	9.30	4.81	10.77	5.32	10.63	5.21	15.05	9.39	14.99	8.87
	R_w (kips)	132.82	73.31	126.92	65.69	159.62	78.83	136.17	66.81	223.00	139.20	192.02	182.61
	F_t (kips)	54.00	54.00	54.00	54.00	54.00	54.00	54.00	54.00	124.00	124.00	124.00	124.00
Deck Overhang Design	1.2*F_t (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	148.80	148.80	148.80
	Design R_w (kips)	64.80	64.80	64.80	64.80	64.80	64.80	64.80	64.80	148.80	139.20	148.80	148.80
	R_w*H/(L_c+aH) (ft-kips/ft)**	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	37.79	23.69	42.11
	Design M_s (ft-kips/ft)	12.39	23.28	12.27	24.01	9.72	19.59	9.80	19.83	23.62	32.87	23.69	42.11
	Design T (kips/ft)	4.65	8.73	4.33	8.47	3.65	7.35	3.68	7.44	6.75	10.80	6.77	12.03
Deck to Barrier Reinforcement	A_s required (in^2/ft)	0.29	0.57	0.29	0.59	0.17	0.35	0.20	0.41	0.43	0.60	0.49	0.91
	A_s provided (in^2/ft)	0.41	0.62	0.41	0.62	0.41	0.62	0.41	0.62	0.59	0.89	0.59	0.97
	S_1 Bars	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#5 @ 9 in	#5 @ 6 in	#6 @ 9 in	#6 @ 6 in	#6 @ 8 in	#6 @ 5.5 in

Horizontal moment capacity (adjusted for bridge skew and transverse reinforcement orientation)

$$M_s := \frac{12.27}{\cos(22\ deg)}\ kip \cdot ft = 13.23\ kip \cdot ft$$

Lever arm for vertical collision

$$l_a := 10.5\ in - 4\ in = 0.54\ ft$$

Longitudinal distribution of vertical force (AASHTO Table A13.2-1)

$$L_v := 18\ ft$$

Vertical design force (AASHTO Table A13.2-1)

$$F_v := 18\ kip$$

Tensile force from the barrier

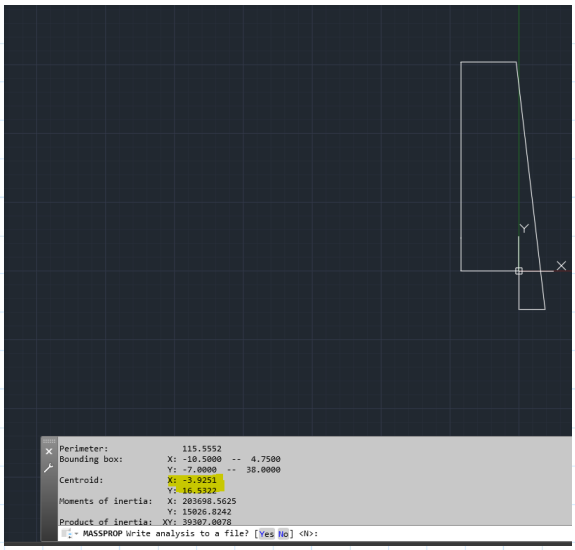
$$T_1 := \frac{4.33}{\cos(22\ deg)}\ kip = 4.67\ kip$$

Critical length of yield line

$$L_c := 9.30\ ft$$

Note: l_a is the horizontal distance between the interior face of the barrier to the centroid of the barrier. The centroid is located at approximately 4" from edge of deck (measured using AutoCad, see screenshot on next sheet)

Material and Section Properties



Concrete strength $f'_c := 5\ ksi$

Steel yield strength $f_y := 60\ ksi$

Concrete top cover $c_{top} := 2.5\ in$

Concrete bottom cover in deck and top and bottom covers in bottom slab $c_{bottom} := 1\ in$

Overhang thickness at end $t_{overhang} := 7\ in$

Top Deck Thickness $t_{top} := 8.5\ in$

Overhang length $L_{overhang} := 4.5\ ft$

Horizontal distance between the edge of deck and the inside face of the barrier $L_{barrier} := 10.5\ in$

Top Deck Thickness at Fillet $t_{top\_fillet} := 6\ in + t_{top} = 14.5\ in$

Overhang thickness at face of barrier $t_{overhang\_barrier} := t_{overhang} + \left(\frac{t_{top\_fillet} - t_{overhang}}{L_{overhang}}\right) \cdot \left(L_{barrier}\right) = 8.46\ in$

Overhang thickness at outside face of web $t_{overhang\_web} := t_{top\_fillet} = 14.5\ in$

Deck Width $b := 12\ in$

Section 1: Overhang design at the interior face of the barrier

Design Case A: Horizontal Vehicular Collision Load

The deck overhang is designed to resist an axial tension force from vehicular collision acting simultaneously with the Dead Load moment.

$$M_{slab} := \frac{-0.01}{\cos(22\ deg)}\ kip \cdot ft$$

$$M_{barrier} := \frac{-0.17}{\cos(22\ deg)}\ kip \cdot ft$$

$$M_{DC} := M_{slab} + M_{barrier} = -0.19\ kip \cdot ft$$

$$M_{u1A} := -M_s + 1.0\ M_{DC} = -13.43\ kip \cdot ft$$

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

Note: Assume #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #5 @ 12"
For #5 @ 6" (typ. top transverse reinforcement):

$A_{s\_top1} := 0.31\ in^2$ $s_{top1} := 6\ in$ $d_{bar\_top1} := 0.625\ in$

For additional short hooked #5 @ 12"
:

$A_{s\_top2} := 0.31\ in^2$ $s_{top2} := 12\ in$ $d_{bar\_top2} := 0.625\ in$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5 $d_{1\_\#5} := t_{overhang\_barrier} - c_{top} - \frac{d_{bar\_top1}}{2} = 5.646\ in$

Resistance Factor for Extreme Load Combination $\varphi_{ext} := 1$

Area of reinforcement per ft $A_{s\_top1'} := \frac{A_{s\_top1}}{\frac{s_{top1}}{12\ in}} = 0.62\ in^2$ $A_{s\_top2'} := \frac{A_{s\_top2}}{\frac{s_{top2}}{12\ in}} = 0.31\ in^2$

The development length of #5 is 10.2". The calculations are at spreadsheet "development length"

Development length of #5 $l_{d\_5} := 10.2\ in$

Note: the development length is restrained by the distance from the deck edge to the interior surface of the barrier and concrete cover

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #5 $f_{y\_M5} := f_y \cdot \frac{min(l_t, l_{d\_5})}{l_{d\_5}} = 50\ ksi$

Compression Block Depth $a_1 := \frac{\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5} + (A_{s\_top2'}) \cdot f_{y\_M5}}{0.85\ f'_c \cdot b} = 0.87\ in$

Unfactored resistance for top slab

$$M_{n\_top1} := -\left(\left(A_{s\_top1'} - \frac{T_1}{2 \cdot f_{y\_M5}}\right) \cdot f_{y\_M5}\right) \cdot \left(d_{1\_\#5} - \frac{a_1}{2}\right) - \left((A_{s\_top2'}) \cdot f_{y\_M5}\right) \cdot \left(d_{1\_\#5} - \frac{a_1}{2}\right) = -19.19\ kip \cdot ft$$

Factored Resistance $\varphi_{ext} \cdot M_{n\_top1} = -19.19\ kip \cdot ft$

Check for reinforcement in top slab $\text{if}\left(|M_{u1A}| < |\varphi_{ext} \cdot M_{n\_top1}|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.7$

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement:

Note: Assume #4 bars at 6 inches for bottom reinforcement

$A_{s\_bottom} := 0.2 \cdot in^2$ $s_{bottom} := 6\ in$ $d_{bar\_bottom} := 0.5\ in$

Development length of #4 $l_{d\_4} := 20\ in$

Maximum available development length $l_t := L_{barrier} - 2\ in = 8.5\ in$

Modified stress of #4 $f_{y\_M4} := f_y \cdot \frac{min\left(l_t, l_{d\_4}\right)}{l_{d\_4}} = 25.5\ ksi$

Area of Tension Transverse Reinforcement $A_{s\_bottom'} := \frac{\left(A_{s\_bottom}\right)}{\left(\frac{s_{bottom}}{\frac{in}{12}}\right)} = 0.4\ in^2$

Unfactored Tensile Resistance for bottom slab $T_{n\_bottom1} := A_{s\_bottom'} \cdot f_{y\_M4} = 10.2\ kip$

Factored Resistance $\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$

Check for reinforcement in top slab $\text{if}\left(\left|\frac{T_1}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom1}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

D/C ratio $\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.229$

Design Case B: Vertical Vehicular Collision Load

Moment due to vertical vehicular collision load $M_v := \frac{F_v \cdot \frac{l_a}{L_v} \cdot 1\ ft}{\cos\left(22\ deg\right)} = 0.58\ ft \cdot kip$

$$M_{u1B} := M_{DC} + -M_v = -0.78\ kip \cdot ft$$

Note: Since Mu1B<Mu1A, design case A controls.

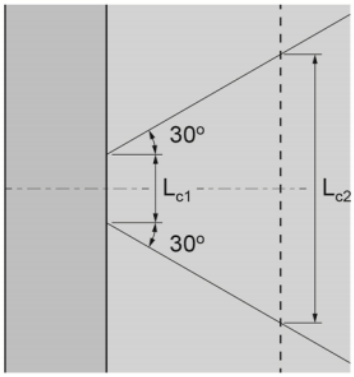
Design Case C: Dead and Live Loads at Strength Limit State

$$M_{u1C} := 1.25\ M_{DC} = -0.24\ kip \cdot ft$$

Note: Since $\frac{Mu1c}{\varphi_{str}} < \frac{Mu1a}{\varphi_{ext}}$, design case A controls.

Section 2: Overhang design at the Outside Face of the Web

Design Case A: Horizontal Vehicular Collision Load



As depicted in the figure above, loads distribute over larger lengths the farther the section is from the load application. Assume the distribution length, Lc, increases at a 30 degrees angle

Barrier height $H_{barrier} := 32\ in$

$$M_{c2} := \frac{M_s \cdot L_c}{L_c + 2 \cdot (L_{overhang} - L_{barrier}) \cdot \tan(30\ deg)} = 9.13\ ft \cdot kip$$

$$T_2 := \frac{T_1 \cdot L_c}{L_c + 2 \cdot (L_{overhang} - L_{barrier}) \cdot \tan(30\ deg) + 2 \cdot H_{barrier}} = 2.31\ kip$$

The moment due to the dead loads are:

$$M_{slab2} := \frac{-1.9}{\cos(22\ deg)}\ kip \cdot ft = -2.05\ kip \cdot ft$$

$$M_{barrier2} := \frac{-3.3}{\cos(22\ deg)}\ kip \cdot ft = -3.56\ kip \cdot ft$$

Note: results obtained from CSiBridge model. Moment due to sw of slab is from load case "DEAD" and the moment due to sw of barrier is from load case " BARRIER"

Extreme event II load combination:

$$M_{u2} := -M_{c2} + 1.0\ (M_{slab2} + M_{barrier2}) = -14.73\ kip \cdot ft$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5

$$d_{2\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69\ in$$

Compression Block Depth

$$a_2 := \frac{\left(\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y + (A_{s\_top2'}) \cdot f_y\right)}{0.85\ f'_c \cdot b} = 1.07\ in$$

Unfactored resistance for top slab

$$M_{n\_top2} := -\left(A_{s\_top1'} - \frac{T_2}{2 \cdot f_y}\right) \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_2}{2}\right) - (A_{s\_top2'}) \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_2}{2}\right) = -50.78\ kip \cdot ft$$

Factored Resistance $\varphi_{ext} \cdot M_{n\_top2} = -50.78\ kip \cdot ft$

Check for reinforcement in top slab

$$\text{if}\left(\left|M_{u2}\right| < \left|\varphi_{ext} \cdot M_{n\_top2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$

$$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.29$$

Bottom Transverse Reinforcement Tensile Strength Check

Check if bottom reinforcement in the deck has enough reinforcement to resist the tension due to collision. Assuming half of the the tension force is carried by the bottom layer of reinforcement :

Unfactored Tensile Resistance for bottom slab

$$T_{n\_bottom2} := A_{s\_bottom'} \cdot f_y = 24 \ \textit{kip}$$

Factored Resistance

$$\varphi_{ext} \cdot T_{n\_bottom2} = 24 \ \textit{kip}$$

Check for reinforcement in bottom slab

$$\text{if}\left(\left|\frac{T_2}{2}\right| < \left|\varphi_{ext} \cdot T_{n\_bottom2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$

$$\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.048$$

Design Case B: Vertical Vehicular Collision Load

As per BDM Appendix 5-B6, for concrete parapets, the case of vertical collision never controls.

Design Case C: Dead and Live Loads at Strength Limit State

There is no moment due to the live loads at this location because of the sidewalk:

$$M_{LL} := 0 \ \textit{kip} \cdot \textit{ft}$$

Strength I load combination:

$$M_{u2\_strengthI} := 1.25 \left(M_{slab2} + M_{barrier2}\right) + 1.75 \ M_{LL} = -7.01 \ \textit{kip} \cdot \textit{ft}$$

Vertical Distance from Tension Reinforcement to Extreme Compression Fiber for #5

$$d_{2\_strengthI\_\#5} := t_{top\_fillet} - c_{top} - \frac{d_{bar\_top1}}{2} = 11.69 \ \textit{in}$$

Compression Block Depth

$$a_{2\_strenghtI} := \frac{A_{s\_top1'} \cdot f_y + A_{s\_top2'} \cdot f_y}{0.85 \ f'_c \cdot b} = 1.09 \ \textit{in}$$

Unfactored resistance for top slab

$$M_{n\_top2\_strengthI} := -A_{s\_top1'} \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_{2\_strenghtI}}{2}\right) - A_{s\_top2'} \cdot f_y \cdot \left(d_{2\_\#5} - \frac{a_{2\_strenghtI}}{2}\right) = -51.8 \ \textit{kip} \cdot \textit{ft}$$

Resistance Factor

$$\varphi_{str} := 0.9$$

Factored Resistance

$$\varphi_{str} \cdot M_{n\_top2} = -45.705 \ \textit{kip} \cdot \textit{ft}$$

Check for reinforcement in top slab

$$\text{if}\left(\left|M_{u2}\right| < \left|\varphi_{ext}\cdot M_{n\_top2}\right|, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$$

$\frac{D}{C}$

$$\frac{M_{u2\_strengthI}}{\varphi_{str}\cdot M_{n\_top2\_strengthI}} = 0.15$$

Control of Cracking at Service Limit State

Service Limit State Demand at Interior Face of Barrier

$$M_{u1\_service} := 1.0 \quad M_{DC} = -0.194 \ \textit{kip}\cdot\textit{ft}$$

Check cracking at the top of deck

Class 2 Exposure factor (AASHTO 5.6.7)

$$\gamma_e := 0.75$$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto

$$d_{c\_top1} := c_{top} + \frac{1}{2}\cdot d_{1\_\#5} = 5.32 \ \textit{in}$$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7)

$$\beta_{s\_top1} := 1 + \frac{d_{c\_top1}}{0.7\cdot\left(t_{overhang\_barrier} - d_{c\_top1}\right)} = 3.43$$

Tension Reinforcement Ratio

$$\rho_{top1} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b\cdot\left(d_{1\_\#5}\right)} = 0.01$$

Young's Modulus of Steel

$$E_s := 29000 \ \textit{ksi}$$

Unit Weight of Concrete

$$w_c := 0.155 \ \frac{\textit{kip}}{\textit{ft}^3}$$

Correction Factor for Source of Aggregate (5.4.2.4)

$$K_1 := 1$$

Young's Modulus of Concrete (5.4.2.4)

$$E_c := 120000\cdot\left(\frac{w_c}{\frac{\textit{kip}}{\textit{ft}^3}}\right)^2\cdot K_1\cdot\left(\frac{f'_c}{\textit{ksi}}\right)^{0.33}\cdot\textit{ksi} = \left(4.9\cdot 10^3\right)\ \textit{ksi}$$

$$n := \frac{E_s}{E_c} = 5.914$$

$$k_{top1} := \sqrt{2\ n\cdot\rho_{top1} + \left(n\cdot\rho_{top1}\right)^2} - n\cdot\rho_{top1} = 0.33 \qquad j_{top1} := 1 - \frac{k_{top1}}{3} = 0.89$$

Tensile stress in nonprestressed reinforcement at SLS

$$f_{ss\_top1} := \frac{-M_{u1\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)\cdot j_{top1}\cdot d_{1\_\#5}} = 0.5 \ \textit{ksi}$$

Maximum reinforcement spacing for negative moment

$$s_{max\_top1} := \frac{700\cdot\gamma_e\cdot\textit{in}}{\beta_{s\_top1}\cdot\frac{f_{ss\_top1}}{\textit{ksi}}} - 2\cdot d_{c\_top1} = 296.82 \ \textit{in}$$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top1}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top1}} = 0.02$

Service Limit State Demand at Exterior Face of Web

$M_{u2\_service} := \left(M_{slab2} + M_{barrier2}\right) + M_{LL} = -5.608\ \textit{kip}\cdot\textit{ft}$

Check cracking at the top of deck

Exposure factor for Class 2 Exposure Condition (AASHTO 5.6.7) $\gamma_e := 0.75$

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_{c\_top2} := c_{top} + \frac{1}{2}\cdot\frac{\left(A_{s\_top1'}\cdot d_{bar\_top1} + A_{s\_top2'}\cdot d_{bar\_top2}\right)}{\left(A_{s\_top1'} + A_{s\_top2'}\right)} = 2.81\ \textit{in}$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_{s\_top2} := 1 + \frac{d_{c\_top2}}{0.7\cdot\left(t_{top\_fillet} - d_{c\_top2}\right)} = 1.34$

Tension Reinforcement Ratio $\rho_{top2} := \frac{A_{s\_top1'} + A_{s\_top2'}}{b\cdot d_{2\_strengthI\_\#5}} = 0.007$

$k_{top2} := \sqrt{2\ n\cdot\rho_{top2} + \left(n\cdot\rho_{top2}\right)^2} - n\cdot\rho_{top2} = 0.244$ $j_{top2} := 1 - \frac{k_{top2}}{3} = 0.919$

Tensile stress in nonprestressed reinforcement at SLS $f_{ss\_top2} := \frac{-M_{u2\_service}}{\left(A_{s\_top1'} + A_{s\_top2'}\right)\cdot j_{top2}\cdot d_{2\_strengthI\_\#5}} = 6.74\ \textit{ksi}$

Maximum reinforcement spacing for negative moment $s_{max\_top2} := \frac{700\cdot\gamma_e\cdot\textit{in}}{\beta_{s\_top2}\cdot\frac{f_{ss\_top2}}{\textit{ksi}}} - 2\cdot d_{c\_top2} = 52.35\ \textit{in}$

Service limit state check $\text{if}\left(s_{top1} < s_{max\_top2}, \text{“pass”}, \text{“fail”}\right) = \text{“pass”}$

$\frac{D}{C}$ $\frac{s_{top1}}{s_{max\_top2}} = 0.115$

Summary

Top reinforcement: #5 @ 6" (typ. top transverse reinforcement) and additional short hooked #5 @12"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$M_{u1A} = -13.43\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top1} = -19.19\ kip \cdot ft$	$\frac{M_{u1A}}{\varphi_{ext} \cdot M_{n\_top1}} = 0.7$
@ Outside Face of the Web	$M_{u2} = -14.73\ kip \cdot ft$	$\varphi_{ext} \cdot M_{n\_top2} = -50.78\ kip \cdot ft$	$\frac{M_{u2}}{\varphi_{ext} \cdot M_{n\_top2}} = 0.29$
@ Surface of Exterior Web	$M_{u2\_strengthI} = -7.01\ kip \cdot ft$	$\varphi_{str} \cdot M_{n\_top2\_strengthI} = -46.62\ kip \cdot ft$	$\frac{M_{u2\_strengthI}}{\varphi_{str} \cdot M_{n\_top2\_strengthI}} = 0.15$

Maximum Spacing at SLS

	Maximum Spacing	Actual Spacing	D/C Ratio
@ Surface of Barrier	$s_{max\_top1} = 296.82\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top1}} = 0.02$
@ Surface of Exterior Web	$s_{max\_top2} = 52.35\ in$	$s_{top1} = 6\ in$	$\frac{s_{top1}}{s_{max\_top2}} = 0.115$

Bottom reinforcement:#4 at 6"

Ultimate Capacity

	Demand	Capacity	D/C Ratio
@ Surface of Barrier	$\frac{T_1}{2} = 2.34\ kip$	$\varphi_{ext} \cdot T_{n\_bottom1} = 10.2\ kip$	$\frac{\frac{T_1}{2}}{\varphi_{ext} \cdot T_{n\_bottom1}} = 0.23$
@ Surface of Exterior Web	$\frac{T_2}{2} = 1.15\ kip$	$\varphi_{ext} \cdot T_{n\_bottom2} = 24\ kip$	$\frac{\frac{T_2}{2}}{\varphi_{ext} \cdot T_{n\_bottom2}} = 0.05$

COWI

PROJECT	I-405 R2B: Bridge 17.7				CONT A207833
SUBJECT	Superstructure Transverse Reinforcement Design - Development Lengths				PAGE
CALCS BY	LDLI	DATE 2021-10-29		CHECK BY FCCA	DATE 2021-10-29

Calculation Description: The calculation below is done to determine the development lengths and hook development lengths

Summary:

- Calculate development and splice lengths of reinforcing bars in tension
- Based on AASHTO LRFD Bridge Design Specification, 8th Edition, 2017
- Minimum Splice 12.0" length, AASHTO BDS APPENDIX 5.1-A4

Input:

User input=			
f'c =	5.0	ksi	concrete 28-day compressive strength
λ =	1.0		Normal weight concrete AASHTO BDS 5.4.2.8
A_s=	0.0		Transverse reinforcement is ignored as per C5.10.8.2.1.c
n=	2.0		AASHTO BDS Fig.C5.10.8.2.1c-1
K_s=	0.0		AASHTO BDS Eq.5.10.8.2.1c-3. This is conservatively taken as zero
Splice Class=	B		
Splice factor=	1.3		AASHTO BDS Eq.5.10.8.4.3a

DEVELOPMENT AND SPLICE LENGTH FOR FLEXURAL REINFORCEMENT

References(AASHTO BDS 2017)								Eq.5.10.8.2.1a-2	5.10.8.2.1b	Fig.C5.10.8.2.1c-1	Eq.5.10.8.2.1c-1 Eq.5.10.8.2.1c-2	5.10.8.2.1b					5.10.8.2.3	5.10.8.2.4b	Eq.5.10.8.2.1a-1
Location		Size	Spacing [in]	Epoxy Coated?	Clear Cover [in]	Yield strength of reinforcement, fy (Ksi)	Bar Dia, d_b [in]	Basic development Length, l_{db} [in]	λ_{rl}	c_b [in]	λ_{rc}	Clear Cover< 3 d_b?	Clear Spacing< 6 d_b?	λ_{cf}	$\lambda_{rl} \times \lambda_{cf} <= 1.7$	No.of Bundled Bars	Bundled bars Factor	λ_{cr}	Tension development l_d [in]
Deck	Transverse-Bot Rebar	#5	6	YES	1	60	0.63	40.2	1	1.31	0.48	YES	NO	1.50	1.50	0	1.0	0.65	19.0
Overhang	Transverse-Bot Rebar_Long and Short Bar	#4	6	YES	1	60	0.50	32.2	1	1.25	0.40	YES	NO	1.50	1.50	0	1.0	1.0	20.0
Bottom Slab	Transverse-Top Rebar	#4	12	NO	1	60	0.50	32.2	1	1.25	0.40	YES	NO	1.00	1.00	0	1.0	1.0	13.0
(Typ.)	Transverse-Bot Rebar	#4	12	NO	1	60	0.50	32.2	1	1.25	0.40	YES	NO	1.00	1.00	0	1.0	1.0	13.0

BASIC HOOK DEVELOPMENT LENGTH FOR FLEXURAL REINFORCEMENT

References(AASHTO BDS 2017)										Eq. 5.10.8.2.4a-2	5.10.8.2.4b	5.10.8.2.4b	5.10.8.2.4b	5.10.8.2.4a C5.10.8.2.4a
Location		Size	Spacing [in]	Epoxy Coated?	Hook Type	Yield strength of reinforcement, fy (Ksi)	End Cover [in]	Side Cover [in]	Bar Dia, d_b [in]	Basic development Length, l_{hb} [in]	λ_{cw}	λ_{rc}*	λ_{cr}	l_{dh} (in)
Overhang and Deck	Transverse-Top_Interior+End Zone_Long Bar	#5	6	YES	180	60	2	2.5	0.63	10.6	1.2	0.80	1	10.2
Overhang	Transverse-Top_Interior Zone_Short Bar	#5	12	YES	180	60	2	2.5	0.63	10.6	1.2	0.80	1	10.2
	Transverse-Top_End Zone_Short Bar	#6	6	YES	180	60	2	2.5	0.75	12.7	1.2	0.80	1	12.2

*Note the equation of λrc doesn't govern all the equations in AASHTO and fig shown right . Look into AASHTO Section 5.10.8.2.4b for further info.

COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I5 CALC BOOK COMPLETION DATE: **2022 Nov 25**

Title: Existing and Proposed Forces and Stresses Comparison

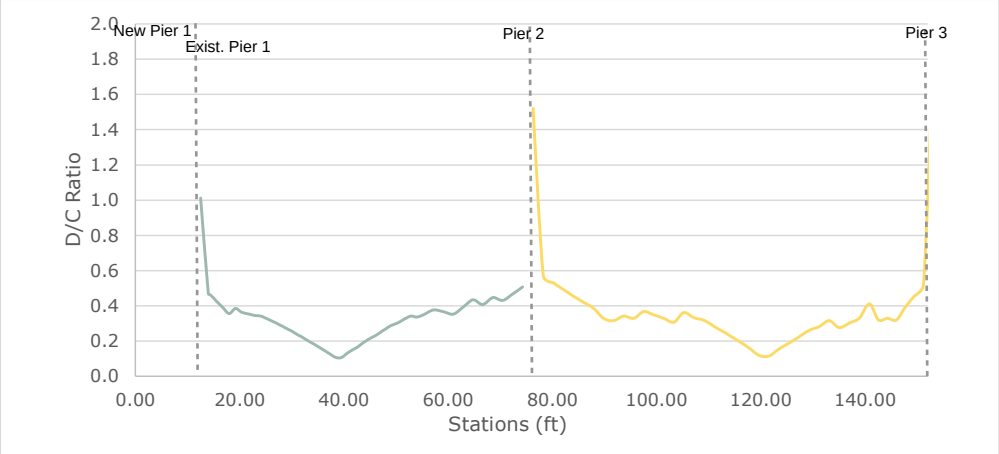
Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I5-0	-	I5-0	Calc Register (this sheet)	N/A	N/A	
2	I5-1	-	I5-1	1. Existing Structure Shear D/C	LDLI	CUPG	
3	I5-2	-	I5-2	2. Existing Structure Flexure D/C	LDLI	CUPG	
4	I5-3	-	I5-3	3. Existing and Proposed Structure Serviceability D/C	LDLI	CUPG	
5	I5-3	-	I5-3	4. Existing and Proposed Structure Shear, Flexure, and Serviceability D/C	LDLI	CUPG	
6	I5-4	-	I5-4	5. Existing Structure Forces	LDLI	CUPG	
7	I5-5	-	I5-5	6. Proposed Structure Forces	LDLI	CUPG	
8	I5-6	-	I5-6	7. Existing and Proposed Structure Force Comparison	LDLI	CUPG	
9	I5-7	-	I5-7	8. Existing Structure Concrete Stresses for Uncracked Cross-section	LDLI	CUPG	
10	I5-8	-	I5-8	9. Proposed Structure Concrete Stresses for Uncracked Cross-section	LDLI	BOHU	
11	I5-9	-	I5-9	10. Proposed and Existing Structure Concrete Stresses for Uncracked Cross-section Comparison	LDLI	BOHU	
12	I5-10	-	I5-10	11. Proposed and Existing Structure Reinforcement Tensile Stresses for Cracked Cross-section	LDLI	BOHU	
13	I5-11	-	I5-21	Appendix 1 - Existing Structure Shear and Flexure D/C Calculations	LDLI	CUPG	
14	I5-22	-	I5-43	Appendix 2 - Serviceability D/C Calculations	LDLI	CUPG	
15	I5-44	-	I5-57	Appendix 3 - Cracked Section Reinforcement Tensile Stress Calculations	LDLI	BOHU	

1. Existing Structure Shear D/C

COWI

Project I-405 Bridge 17.7 **Project No** A207833
Title Existing Structure Shear Capacity D/C
PREPARED BY LDLI **Date** 11/25/2021
CHECKED BY CUPG **Date** 11/25/2021

Description:
This sheet summarized the outputs of the shear capacity check for Bridge 17.7 existing superstructure.



see appendix 1 in this calc book for detailed calculations

	Station (ft)	Min Reinf.?	Max Spacing?	Long. Reinf.?	Governed D/C
Pier 3	152.93	yes	no	no	1.7
	151.20	yes	yes	yes	0.5
	149.48	yes	yes	yes	0.5
	147.75	yes	yes	yes	0.4
	146.02	yes	yes	yes	0.3
	144.29	yes	yes	yes	0.3
	142.57	yes	yes	yes	0.3
	140.84	yes	yes	yes	0.4
	138.91	yes	yes	yes	0.3
	136.98	yes	yes	yes	0.3
	135.05	yes	yes	yes	0.3
	133.11	yes	yes	yes	0.3
	131.18	yes	yes	yes	0.3
	129.25	yes	yes	yes	0.3
	127.32	yes	yes	yes	0.2
	125.39	yes	yes	yes	0.2
	123.46	yes	yes	yes	0.2
	121.53	yes	yes	yes	0.1
	119.59	yes	yes	yes	0.1
	117.66	yes	yes	yes	0.2
East Spa	115.73	yes	yes	yes	0.2
	113.80	yes	yes	yes	0.2
	112.93	yes	yes	yes	0.2
	111.01	yes	yes	yes	0.3
	109.08	yes	yes	yes	0.3
	107.16	yes	yes	yes	0.3
	105.23	yes	yes	yes	0.4
	103.31	yes	yes	yes	0.3
	101.39	yes	yes	yes	0.3
	99.46	yes	yes	yes	0.3
	97.54	yes	yes	yes	0.4
	95.62	yes	yes	yes	0.3
	93.69	yes	yes	yes	0.3
	91.77	yes	yes	yes	0.3
	89.84	yes	yes	yes	0.3
	87.92	yes	yes	yes	0.4
	86.00	yes	yes	yes	0.4
	84.07	yes	yes	yes	0.5
	82.15	yes	yes	yes	0.5
	80.22	yes	yes	yes	0.5
	78.30	yes	yes	yes	0.6
Pier 2	76.30	yes	no	yes	1.5

Station (ft)	Min Reinf.?	Max Spacing	Long. Reinf.	Governed D/C	
74.30	yes	yes	yes	0.5	
72.39	yes	yes	yes	0.5	
70.49	yes	yes	yes	0.4	
68.58	yes	yes	yes	0.4	
66.68	yes	yes	yes	0.4	
64.77	yes	yes	yes	0.4	
62.86	yes	yes	yes	0.4	
60.96	yes	yes	yes	0.4	
59.05	yes	yes	yes	0.4	
57.15	yes	yes	yes	0.4	
55.24	yes	yes	yes	0.3	
54.04	yes	yes	yes	0.3	
52.84	yes	yes	yes	0.3	
51.64	yes	yes	yes	0.3	
50.44	yes	yes	yes	0.3	
48.87	yes	yes	yes	0.3	
47.30	yes	yes	yes	0.3	
45.85	yes	yes	yes	0.2	
44.40	yes	yes	yes	0.2	
42.62	yes	yes	yes	0.2	
40.83	yes	yes	yes	0.1	
39.63	yes	yes	yes	0.1	West Span
38.43	yes	yes	yes	0.1	
37.23	yes	yes	yes	0.1	
36.03	yes	yes	yes	0.1	
34.83	yes	yes	yes	0.2	
33.63	yes	yes	yes	0.2	
32.43	yes	yes	yes	0.2	
31.22	yes	yes	yes	0.2	
30.02	yes	yes	yes	0.3	
28.82	yes	yes	yes	0.3	
27.62	yes	yes	yes	0.3	
26.42	yes	yes	yes	0.3	
25.22	yes	yes	yes	0.3	
24.02	yes	yes	yes	0.3	
22.82	yes	yes	yes	0.3	
21.62	yes	yes	yes	0.4	
20.42	yes	yes	yes	0.4	
19.22	yes	yes	yes	0.4	
18.02	yes	yes	yes	0.4	
16.82	yes	yes	yes	0.4	
15.61	yes	yes	yes	0.4	
14.41	yes	yes	yes	0.5	
14.00	yes	yes	yes	0.5	
12.51	yes	no	yes	1.0	Exist Pier 1

2. Existing Structure Flexure D/C

COWI

Project I-405 Bridge 17.7 **Project No** A207833

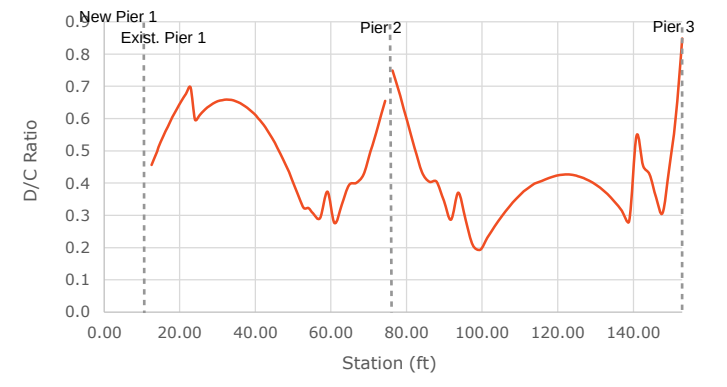
Title Existing Structure Flexural D/C

REPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

Description:
This sheet summarizes the outputs of the flexural capacity check for Bridge 17.7 existing superstructure.

see appendix 1 in this calc book for detailed calculations



Note: the longitudinal reinforcements in the flare region for stations between 113.8 and 140.84ft are ignored in the capacity calculation, so the actual D/C ratios in cross-sections within the flare region are less than shown in

	Station (ft)	D/C Ratio for Max. M	D/C Ratio for Min. M	Governed D/C
Pier 3	152.93	0.43	0.85	0.8
	151.20	0.17	0.60	0.6
	149.48	0.07	0.44	0.4
	147.75	0.22	0.31	0.3
	146.02	0.36	0.18	0.4
	144.29	0.43	0.07	0.4
	142.57	0.45	0.02	0.5
	140.84	0.55	0.07	0.5
	138.91	0.28	0.06	0.3
	136.98	0.31	0.08	0.3
	135.05	0.34	0.09	0.3
	133.11	0.37	0.11	0.4
	131.18	0.39	0.12	0.4
	129.25	0.40	0.13	0.4
	127.32	0.41	0.13	0.4
	125.39	0.42	0.14	0.4
	123.46	0.43	0.14	0.4
	121.53	0.43	0.13	0.4
	119.59	0.42	0.13	0.4
	117.66	0.42	0.12	0.4
East Span	115.73	0.41	0.11	0.4
	113.80	0.40	0.10	0.4
	112.93	0.39	0.09	0.4
	111.01	0.37	0.08	0.4
	109.08	0.35	0.06	0.4
	107.16	0.33	0.04	0.3
	105.23	0.30	0.02	0.3
	103.31	0.27	0.01	0.3
	101.39	0.23	0.07	0.2
	99.46	0.19	0.13	0.2
	97.54	0.19	0.21	0.2
	95.62	0.15	0.29	0.3
	93.69	0.09	0.37	0.4
	91.77	0.02	0.29	0.3
	89.84	0.06	0.35	0.3
	87.92	0.16	0.40	0.4
	86.00	0.24	0.40	0.4
	84.07	0.32	0.43	0.4
	82.15	0.40	0.51	0.5
	80.22	0.40	0.59	0.6
	78.30	0.42	0.67	0.7
Pier 2	76.30	0.46	0.75	0.7

	Station (ft)	D/C Ratio for Max. M	D/C Ratio for	Governed D/C
	74.30	0.40	0.65	0.7
	72.39	0.40	0.57	0.6
	70.49	0.39	0.50	0.5
	68.58	0.31	0.43	0.4
	66.68	0.24	0.40	0.4
	64.77	0.17	0.39	0.4
	62.86	0.08	0.33	0.3
	60.96	0.01	0.28	0.3
	59.05	0.14	0.37	0.4
	57.15	0.26	0.29	0.3
	55.24	0.31	0.22	0.3
	54.04	0.32	0.17	0.3
	52.84	0.32	0.13	0.3
	51.64	0.35	0.09	0.4
	50.44	0.39	0.05	0.4
	48.87	0.43	0.01	0.4
	47.30	0.47	0.02	0.5
	45.85	0.51	0.05	0.5
	44.40	0.54	0.07	0.5
	42.62	0.57	0.10	0.6
	40.83	0.60	0.12	0.6
	39.63	0.62	0.14	0.6
West Span	38.43	0.63	0.15	0.6
	37.23	0.64	0.16	0.6
	36.03	0.65	0.17	0.6
	34.83	0.65	0.17	0.7
	33.63	0.66	0.18	0.7
	32.43	0.66	0.18	0.7
	31.22	0.66	0.18	0.7
	30.02	0.65	0.18	0.7
	28.82	0.65	0.18	0.6
	27.62	0.64	0.18	0.6
	26.42	0.63	0.17	0.6
	25.22	0.61	0.16	0.6
	24.02	0.60	0.16	0.6
	22.82	0.70	0.18	0.7
	21.62	0.68	0.16	0.7
	20.42	0.65	0.15	0.7
	19.22	0.63	0.14	0.6
	18.02	0.60	0.12	0.6
	16.82	0.58	0.11	0.6
	15.61	0.54	0.09	0.5
	14.41	0.51	0.07	0.5
	14.00	0.50	0.06	0.5
Exist Pier 1	12.51	0.46	0.03	0.5

3. Existing and Proposed Structure Serviceability D/C

COWI

Project I-405 Bridge 17.7 **Project No** A207833
Title Serviceability D/C
PREPARED BY LDLI **Date** 11/25/2021
CHECKED BY CUPG **Date** 11/25/2021

Serviceability D/C

new

Location	Station	D/C	Governing Check
east	113.62	0.47	min spacing
pier	76.3	0.33	max spacing
west	38.43	0.61	deflection
interface	12.5	0.35	min spacing

see calc book I-2 for detailed calculations

Note: The stations start from the new Pier 1 hinge CL

existing

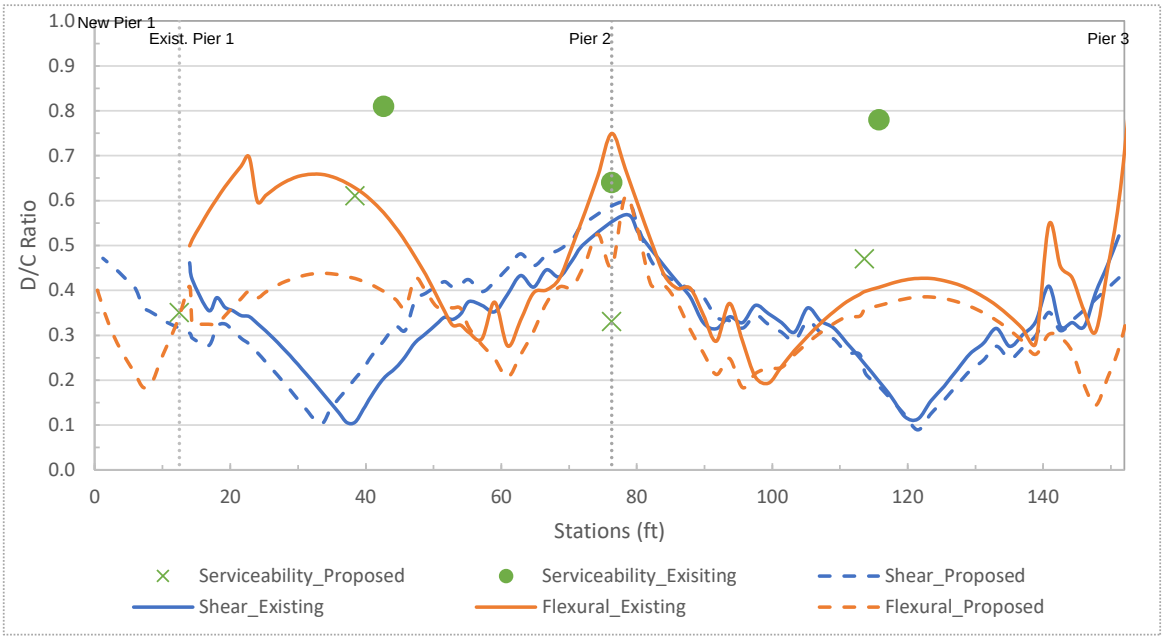
Location	Station	D/C	Governing Check
east	115.74	0.78	max spacing
east pier	76.3	0.64	max spacing
west	42.62	0.81	max spacing

see appendix 2 in this calc book for detailed calculations

Note: The stations start from the new Pier 1 hinge CL

4. Existing and Proposed Structure Shear, Flexure, and Serviceability D/C

Flexural, shear, and serviceability D/Cs



*Note: The calculations for the D/Cs for the proposed structure can be found in Calculation Book I2

5. Existing Structure Forces

COWI

Project I-405 Bridge 17.7
Title Existing Structure Forces
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

		Unfactored Dead Load		*Unfactored Live Load*				*Service III Load Combination*				*Strength I Load Combination*			
		DC		LL				SE3				ST1			
	Station (ft)	V (kip)	M (kip.ft)	Vmax (kip)	Vmin (kip)	MMAX (kip.ft)	MMIN (kip.ft)	Vmax (kip)	Vmin (kip)	MMAX (kip.ft)	MMIN (kip.ft)	Vmax (kip)	Vmin (kip)	MMAX (kip.ft)	MMIN (kip.ft)
Pier 3	152.93	-464	-380	10	-127	59	-194	-476	-616	-2281	-2683	-364	-786	-1947	-3568
	151.20	-437	420	10	-123	191	-61	-449	-584	-1308	-1706	-339	-744	-911	-2514
	149.48	-410	1172	10	-118	376	-61	-422	-553	-332	-910	-315	-702	289	-1866
	147.75	-384	1877	11	-114	552	-73	-395	-522	588	-174	-291	-661	1412	-1284
	146.02	-357	2534	11	-109	714	-88	-369	-491	1447	512	-267	-621	2453	-749
	144.29	-332	3145	12	-105	865	-104	-341	-461	2248	1151	-241	-581	3416	-257
	142.57	-306	3710	14	-101	1009	-119	-314	-432	2997	1744	-215	-543	4311	193
	140.84	-281	4350	17	-96	1176	-138	-286	-401	3787	2366	-187	-502	5264	628
	138.91	-255	4871	20	-92	1302	-155	-258	-371	4477	2917	-160	-463	6065	1034
	136.98	-229	5341	22	-83	1423	-173	-229	-337	5112	3418	-131	-416	6795	1394
	135.05	-204	5762	25	-79	1524	-190	-201	-307	5677	3869	-103	-376	7427	1709
	133.11	-179	6132	28	-75	1615	-208	-173	-278	6182	4270	-75	-337	7979	1979
	131.18	-154	6454	31	-70	1687	-225	-145	-248	6619	4622	-48	-298	8438	2205
	129.25	-129	6727	34	-66	1750	-243	-117	-220	6999	4926	-20	-260	8820	2387
	127.32	-105	6953	38	-62	1798	-261	-89	-191	7316	5181	8	-222	9116	2526
	125.39	-80	7131	41	-58	1838	-278	-61	-163	7578	5389	36	-185	9338	2622
	123.46	-57	7261	46	-56	1867	-296	-33	-137	7780	5550	65	-153	9481	2675
	121.53	-33	7346	50	-50	1880	-313	-5	-108	7920	5665	93	-112	9539	2687
	119.59	-10	7384	54	-50	1872	-331	22	-84	7994	5733	121	-83	9503	2658
East Span	117.66	13	7378	58	-46	1863	-349	49	-57	8021	5756	154	-52	9408	2587
	115.73	36	7326	62	-43	1856	-366	76	-31	8005	5734	190	-25	9261	2476
	113.80	59	7325	66	-38	1864	-382	102	-5	7999	5730	224	2	9152	2367
	112.93	68	7270	68	-35	1851	-390	113	8	7950	5689	240	16	9028	2291
	111.01	89	7118	73	-32	1811	-407	139	32	7804	5570	273	40	8703	2095
	109.08	110	6926	77	-29	1759	-424	164	56	7605	5410	307	64	8306	1862
	107.16	131	6694	81	-26	1694	-442	189	80	7353	5210	340	88	7835	1594
	105.23	152	6422	85	-23	1615	-460	214	103	7046	4967	374	112	7288	1286
	103.31	173	6109	89	-21	1531	-480	239	127	6695	4682	407	135	6684	939
	101.39	194	5756	93	-18	1436	-500	264	150	6297	4354	440	158	6013	555
	99.46	215	5362	97	-16	1335	-520	289	173	5852	3985	474	180	5279	134
	97.54	236	4928	101	-14	1224	-540	314	196	5356	3576	507	202	4497	-343
	95.62	257	4454	105	-13	1103	-560	339	219	4809	3127	540	224	3663	-871
	93.69	278	3940	109	-11	972	-580	364	241	4214	2637	574	246	2762	-1436
	91.77	299	3385	113	-9	833	-601	389	264	3569	2106	607	268	1796	-2038
	89.84	320	2790	117	-8	688	-624	414	286	2878	1533	640	290	767	-2682
	87.92	341	2154	121	-6	560	-658	438	309	2164	908	672	311	-280	-3379
	86.00	362	1478	125	-5	456	-700	463	331	1433	235	705	332	-1336	-4129
	84.07	383	762	128	-4	357	-751	488	353	667	-488	738	353	-2434	-4930
	82.15	404	6	132	-3	262	-810	513	376	-134	-1259	771	374	-3574	-5781
	80.22	425	-791	136	-2	173	-877	537	397	-972	-2077	803	395	-4480	-6958
	78.30	446	-1629	139	-1	105	-988	562	419	-1828	-2981	835	415	-5381	-8266
Pier 2	76.30	-389	-2507	1	-138	65	-1238	-361	-503	-2750	-4091	-364	-766	-6326	-9904
	74.30	-368	-1749	2	-131	122	-1150	-338	-474	-1991	-3296	-343	-727	-5512	-8728
	72.39	-347	-1056	3	-127	199	-1072	-316	-449	-1275	-2572	-322	-693	-4724	-7652
	70.49	-326	-403	5	-123	295	-1001	-294	-424	-579	-1897	-301	-660	-3938	-6640
	69.64	-306	209	6	-118	396	-938	-272	-399	81	-1270	-280	-627	-3106	-5765
	66.68	-285	781	8	-114	502	-884	-249	-374	706	-692	-258	-593	-2174	-5083
	64.77	-264	1313	10	-110	623	-840	-227	-349	1306	-163	-236	-560	-1268	-4454
	62.86	-244	1805	12	-105	756	-803	-204	-324	1877	318	-214	-526	-390	-3874
	60.96	-223	2257	14	-101	879	-769	-181	-299	2403	750	-191	-492	423	-3339
	59.05	-202	2668	16	-96	991	-736	-158	-273	2879	1141	-169	-458	1166	-2841
	57.15	-181	3039	19	-94	1096	-703	-135	-251	3306	1491	-146	-429	1847	-2380
	55.24	-161	3369	21	-88	1185	-670	-113	-225	3678	1800	-125	-394	2480	-1991
	54.04	-149	3558	23	-86	1247	-650	-98	-210	3900	1976	-110	-374	2869	-1767
	52.84	-136	3731	24	-83	1288	-629	-84	-193	4085	2136	-96	-351	3202	-1557
	51.64	-122	3888	26	-80	1341	-610	-69	-178	4266	2278	-80	-331	3537	-1365
	50.44	-109	4028	28	-76	1376	-594	-54	-160	4413	2402	-66	-307	3819	-1192
	48.87	-92	4188	31	-74	1427	-573	-34	-142	4585	2539	-45	-283	4167	-987
	47.30	-75	4320	34	-68	1465	-552	-14	-119	4718	2648	-25	-251	4459	-806
	45.85	-59	4421	36	-65	1493	-532	5	-99	4811	2728	-5	-224	4693	-655
	44.40	-43	4496	39	-62	1512	-511	24	-80	4870	2782	15	-200	4880	-527
	42.62	-22	4554	43	-55	1522	-486	48	-53	4893	2814	40	-163	5042	-400
West Span	40.83	-2	4574	47	-52	1539	-461	72	-30	4886	2808	65	-132	5170	-306
	39.63	11	4565	50	-49	1544	-444	89	-13	4853	2782	86	-114	5217	-263
	38.43	25	4539	53	-47	1543	-427	105	3	4797	2739	107	-97	5232	-235
	37.23	38	4495	55	-44	1535	-409	121	19	4716	2679	129	-80	5214	-222
	36.03	52	4434	58	-41	1521	-392	138	35	4612	2601	151	-63	5164	-225
	34.83	65	4355	61	-39	1501	-375	154	51	4484	2506	173	-47	5080	-243
	33.63	79	4259	64	-37	1474	-358	171	67	4332	2393	195	-31	4964	-276
	32.43	92	4146	67	-35	1440	-341	187	82	4156	2264	217	-15	4814	-325
	31.22	106	4015	70	-33	1404	-324	204	98	3961	2117	240	1	4639	-390
	30.02	119	3866	73	-31	1366	-307	220	114	3746	1952	262	17	4438	-470
	28.82	133	3700	76	-28	1321	-290	237	129	3507	1770	284	33	4203	-565
	27.62	146	3517	80	-26	1269	-272	254	145	3243	1571	307	48	3936	-676
	26.42	160	3316	83	-24	1211	-255	270	161	2957	1355	329	64	3635	-802
	25.22	174	3098	86	-22	1146	-238	287	176	2646	1121	352	80	3302	-944
	24.02	187	2863	89	-20	1075	-221	304	192	2311	870	375	96	2936	-1101
	22.82	201	2610	93	-18	998	-205	321	207	1954	602	397	112	2537	-1274
	21.62	214	2340	96	-18	916	-188	338	221	1574	316	420	124	2109	-1462
	20.42	228	2053	100	-18	829	-172	355	235	1172	13	443	137	1650	-1667
	19.22	241	1748	103	-17	734	-156	372	249	745	-307	466	149	1157	-1887
	18.02	255	1427	107	-17	632	-141	389	262	295	-646	489	162	630	-2124
	16.82	268	1088	110	-17	524	-127	406	276	-180	-1003	512	174	70	-2379
	15.61	282	731	114	-17	408	-118	423	290	-679	-1381	535	186	-524	-2657
	14.41	295	360	117	-17	286	-122	440	303	-1201	-1789	559	199	-1151	-2973
	14.00	300	224	117	-17	239	-137	445	308	-1390	-1949	565	203	-1381	-3110
Existing Pier 1	12.51	317	-263	122	-17	140	-244	467	325	-2017	-2587	594	218	-2013	-3802

6. Proposed Structure Forces

COWI

Project I-405 Bridge 17.7
Title Proposed Structure Forces
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

Forces

		Unfactored Dead Load		Unfactored Prestressing			Unfactore Live Load				Service III Load Combination				Strength I Load Combination			
		DC		PT			LL				SE3				ST1			
	Stations (ft	V (kip)	M (kip.ft)	V (kip)	P (kip)	M (kip.ft)	Vmax (kip)	Vmin (kip)	MMAX (kip.	MMIN (kip.	Vmax (kip)	Vmin (kip)	MMAX (kip.	MMIN (kip.	Vmax (kip)	Vmin (kip)	MMAX (kip.	MMIN (kip.ft
Pier 3	152.93	-438	-300	31	-1012	290	14	-127	69	-200	-369	-511	-1660	-2089	-341	-760	-1693	-3324
	151.20	-411	453	32	-1012	234	14	-123	198	-79	-342	-479	-874	-1313	-316	-718	-680	-2349
	149.48	-384	1158	32	-1012	177	14	-119	383	-85	-315	-448	-83	-711	-292	-676	472	-1743
	147.75	-357	1815	32	-1012	120	14	-114	557	-104	-288	-417	650	-170	-268	-635	1547	-1204
	146.02	-331	2426	32	-1012	64	14	-110	718	-125	-262	-386	1323	320	-244	-595	2540	-712
	144.29	-305	2989	32	-1012	7	15	-105	870	-147	-236	-356	1940	764	-220	-555	3460	-262
	142.57	-280	3507	32	-1012	-50	15	-102	1016	-170	-209	-327	2506	1161	-196	-517	4311	145
	140.84	-255	4085	31	-1040	-130	18	-96	1185	-192	-182	-297	3046	1525	-169	-476	5196	524
	138.91	-229	4554	31	-1040	-191	21	-92	1312	-217	-154	-267	3537	1864	-141	-437	5949	883
	136.98	-203	4973	31	-1040	-252	24	-84	1433	-242	-125	-233	3971	2152	-113	-390	6629	1196
	135.05	-177	5342	31	-1040	-313	26	-79	1536	-266	-97	-203	4336	2391	-85	-350	7215	1464
	133.11	-152	5661	31	-1040	-375	29	-75	1628	-291	-69	-173	4641	2579	-57	-311	7719	1688
	131.18	-127	5931	31	-1040	-436	32	-71	1702	-316	-41	-144	4878	2719	-30	-273	8130	1867
	129.25	-103	6153	31	-1040	-498	35	-67	1766	-341	-13	-115	5057	2809	-2	-234	8463	2003
	127.32	-78	6326	32	-1040	-559	38	-63	1816	-365	15	-87	5174	2852	25	-197	8712	2095
	125.39	-54	6452	32	-1040	-621	42	-59	1858	-390	43	-58	5235	2847	54	-160	8888	2144
	123.46	-30	6532	32	-1040	-683	46	-57	1888	-415	71	-33	5237	2795	82	-127	8983	2152
	121.53	-7	6565	32	-1040	-745	51	-51	1903	-440	99	-3	5178	2696	111	-86	8994	2117
	119.59	17	6551	32	-1040	-807	54	-51	1898	-465	126	21	5053	2552	145	-63	8913	2041
East Span	117.66	40	6493	32	-1040	-869	59	-47	1887	-489	153	48	4875	2361	181	-36	8764	1923
	115.73	63	6390	32	-1040	-931	63	-43	1882	-514	180	74	4659	2126	217	-9	8570	1766
	113.80	85	6330	30	-1060	-1016	67	-39	1892	-530	204	98	4404	1868	251	18	8387	1601
	112.93	95	6252	30	-1060	-1042	69	-35	1879	-541	216	111	4268	1734	267	32	8243	1503
	111.01	116	6050	-87	-1102	-1133	73	-32	1842	-565	137	20	3907	1390	310	61	7886	1277
	109.08	137	5807	-87	-1102	-966	77	-29	1792	-590	162	44	3713	1244	343	85	7424	989
	107.16	158	5525	-87	-1102	-799	81	-26	1729	-614	187	68	3466	1058	377	109	6887	666
	105.23	178	5201	-87	-1102	-632	85	-24	1651	-639	212	92	3164	831	410	133	6274	305
	103.31	199	4838	-87	-1102	-465	89	-21	1568	-663	237	116	2816	564	443	156	5601	-91
	101.39	220	4434	-87	-1102	-298	94	-19	1475	-688	263	139	2419	255	477	179	4862	-525
	99.46	241	3990	-87	-1102	-131	98	-17	1377	-712	288	162	1998	-115	510	202	4075	-1005
	97.54	262	3505	-87	-1102	36	102	-15	1267	-738	313	185	1525	-526	543	224	3236	-1544
	95.62	283	2980	-87	-1102	203	106	-13	1147	-765	338	207	1001	-979	577	246	2343	-2136
	93.69	304	2415	-87	-1102	370	110	-11	1017	-792	363	230	427	-1473	610	268	1383	-2765
	91.77	325	1809	-87	-1102	537	114	-10	878	-819	387	253	-197	-2007	643	290	356	-3430
	89.84	346	1163	-87	-1102	705	117	-8	731	-847	412	275	-869	-2582	676	311	-735	-4133
	87.92	367	477	-87	-1102	872	121	-6	585	-883	437	298	-1580	-3204	709	333	-1874	-4885
	86.00	388	-249	-87	-1102	1039	125	-5	476	-929	462	320	-2295	-3878	742	354	-2912	-5779
	84.07	409	-1016	-87	-1102	1206	129	-4	373	-984	487	343	-3044	-4602	775	375	-3810	-6907
	82.15	430	-1824	-87	-1102	1373	133	-2	274	-1048	511	365	-3829	-5374	807	396	-4736	-8100
	80.22	451	-2671	-87	-1102	1540	136	-2	180	-1120	536	387	-4650	-6194	840	417	-5690	-9358
	78.30	472	-3559	-87	-1102	1707	140	-1	104	-1199	560	408	-5493	-7062	872	437	-6649	-10678
Pier 2	76.30	-462	-4521	107	-1166	1873	1	-144	58	-1341	-385	-543	-6168	-7777	-435	-875	-7381	-11916
	74.30	-440	-3619	107	-1166	1658	2	-138	111	-1178	-362	-515	-5364	-6838	-415	-837	-6432	-10411
	72.39	-420	-2786	107	-1166	1451	3	-134	174	-1094	-341	-491	-4616	-6044	-394	-805	-5531	-9136
	70.49	-399	-1992	107	-1166	1243	4	-131	262	-1018	-319	-466	-3883	-5298	-373	-773	-4622	-7923
	69.64	-378	-1240	107	-1166	1035	5	-127	357	-949	-297	-442	-3183	-4599	-352	-740	-3736	-6774
	66.68	-357	-527	107	-1166	828	7	-123	457	-888	-274	-417	-2518	-3948	-331	-707	-2879	-5688
	64.77	-337	145	107	-1166	620	8	-119	560	-836	-252	-392	-1890	-3346	-310	-675	-2000	-4718
	62.86	-316	777	107	-1166	412	10	-115	678	-792	-230	-368	-1288	-2792	-288	-642	-962	-3984
	60.96	-295	1369	107	-1166	204	12	-111	817	-755	-207	-343	-705	-2287	-267	-609	61	-3299
	59.05	-274	1921	107	-1166	-3	13	-107	948	-724	-185	-318	-155	-1842	-245	-576	1030	-2668
	57.15	-254	2432	107	-1166	-211	15	-105	1073	-696	-162	-295	358	-1450	-223	-547	1941	-2082
	55.24	-234	2902	107	-1166	-416	17	-100	1183	-668	-140	-271	817	-1099	-202	-513	2774	-1536
	54.04	-221	3179	107	-1165	-545	19	-98	1260	-650	-126	-256	1097	-896	-188	-494	3289	-1209
	52.84	-208	3441	107	-1165	-673	20	-95	1317	-633	-112	-240	1341	-709	-174	-472	3770	-917
	51.64	-195	3686	107	-1164	-802	22	-93	1387	-617	-97	-224	1583	-538	-159	-451	4256	-642
	50.44	-182	3915	107	-1163	-930	23	-89	1440	-604	-83	-207	1791	-386	-145	-428	4690	-388
	48.87	-165	4191	107	-1162	-1097	25	-87	1514	-587	-63	-189	2044	-212	-125	-404	5237	-76
	47.30	-148	4438	16	-1139	-1218	28	-82	1572	-568	-131	-245	2298	-17	-100	-364	5717	212
	45.85	-131	4650	16	-1140	-1243	30	-79	1628	-552	-112	-226	2591	228	-81	-338	6136	456
	44.40	-115	4836	16	-1142	-1269	32	-77	1676	-535	-93	-207	2852	447	-62	-314	6510	677
	42.62	-95	5031	16	-1143	-1299	36	-71	1724	-515	-70	-181	3127	683	-39	-278	6907	918
West Span	40.83	-75	5187	16	-1144	-1330	39	-68	1761	-495	-46	-158	3354	882	-15	-248	7237	1126
	39.63	-61	5270	16	-1145	-1350	41	-65	1778	-481	-31	-142	3477	993	1	-226	7419	1246
	38.43	-48	5336	16	-1146	-1370	43	-62	1790	-467	-15	-125	3578	1088	17	-204	7570	1351
	37.23	-34	5384	16	-1146	-1390	46	-60	1799	-453	1	-109	3658	1166	34	-183	7693	1440
	36.03	-21	5414	16	-1147	-1410	48	-57	1812	-439	17	-93	3725	1226	50	-161	7803	1514
	34.83	-7	5427	16	-1147	-1430	50	-54	1819	-425	33	-77	3769	1269	66	-139	7880	1572
	33.63	6	5422	16	-1147	-1450	53	-51	1821	-412	49	-60	3791	1295	85	-120	7926	1614
	32.43	20	5400	16	-1147	-1470	55	-49	1817	-398	65	-44	3789	1303	106	-103	7941	1641
	31.22	34	5360	16	-1148	-1490	58	-46	1808	-384	81	-28	3765	1294	128	-86	7924	1652
	30.02	47	5303	16	-1148	-1509	61	-43	1793	-370	97	-12	3717	1268	149	-69	7875	1648
	28.82	61	5228	16	-1147	-1528	63	-41	1772	-356	113	4	3647	1225	171	-53	7794	1628
	27.62	74	5136	16	-1147	-1548	66	-38	1745	-342	130	20	3553	1165	192	-36	7681	1593
	26.42	88	5026	16	-1147	-1567	69	-36	1712	-328	146	36	3436	1087	214	-20	7536	1542
	25.22	101	4899	16	-1147	-1586	71	-35	1674	-314	162	51	3296	992	236	-5	7358	1475
	24.02	113	4781	16	-1146	-1602	74	-33	1641	-302	176	64	3168	901	254	8	7196	1409
	22.82	124	4652	16	-1146	-1617	76	-31	1607	-291	189	77	3026	799	272	21	7015	1333
	21.62	135	4511	16	-1146	-1633	78	-30	1569	-279	203	90	2869	685	291	34	6813	1246
	20.42	147	4358	16	-1145	-1648	81	-28	1526	-268	217	103	2696	560	309	47	6589	1149
	19.22	158	4195	16	-1145	-1663	83	-27	1480	-256	231	115	2509	424	327	59	6346	1042
	18.02	169	4020	16	-1144	-1678	85	-26	1432	-245	244	128	2307	277	344	70	6084	924
	16.82	181	3834	16	-1143	-1693	88	-24	1376	-233	258	142	2088	119	364	86	5795	797
	15.61	192	3636	16	-1142	-1708	90	-23	1319	-222	272	154	1856	-51	383	97	5489	659
	14.41	203	3428	16	-1142	-1724	93	-20	1258	-211	286	168	1608	-234	401	112	5163	510
	14.00	206	3348	16	-1141	-1728	93	-20	1227	-207	289	171	1508	-301	405	115	5024	454
Existing Pier 1	12.51	218	3063	-148	-1062	-1642	97	-18	1160	-195	157	32	1279	-461	442	141	4595	235
	11.51	228	2843	-148	-1061	-1496	99	-17	1094	-185	171	44	1160	-495	460	153	4233	70
West Span Extension	10.56	238	2625	-148	-1061	-1356	101	-16	1028	-175	184	56	1033	-537	477	166	3871	-95
	9.61	247	2401	-148	-1060	-1217	104	-15	959	-166	197	68	895	-588	494	177	3494	-269
	8.41	259	2102	-148	-1059	-1041	107	-15	867	-155	213	82	701	-667	516	190	2989	-502
	7.21	271	1791	-148	-1058	-865	109	-15	770	-144	230	95	488	-762	537	202	2458	-751
	6.01	282	1466	-148	-1057	-688	112	-14	666	-136	246	109	254	-873	559	215	1899	-1016
	4.81	294	1129	-148	-1057	-512	115	-14	558	-130	263	123	0	-1002	581	227	1313	-1299
	3.61	306	772	-148	-1055	-336	118	-14	445	-130	279	136	-286	-1162	603	240	687	-1616
	2.41	318	394	-148	-1054	-160	121	-14	327	-150	295	149	-605	-1372	623	252	18	-1996
	1.21	329	2	-148	-1053	16	124	-14	243	-224	310	162	-909	-1654	644	263	-612	-2486
Pier 1	0.00	340	-396	-186	-1053	977	127	-14	-322	-390	325	173	-1227	-1976	663	274	-1116	-3172

7. Existing and Proposed Structure Force Comparison

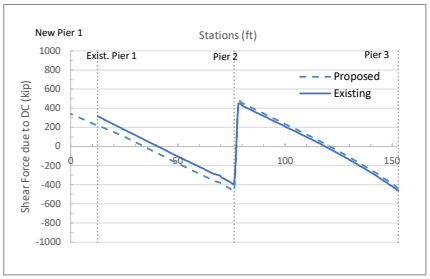
COWI

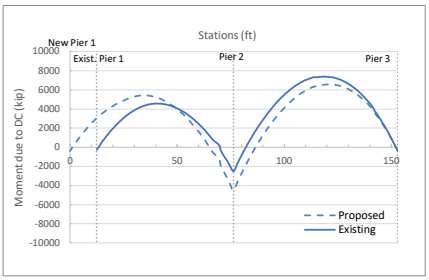
Project I-405 Bridge 17.7
Title Proposed Structure Forces
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

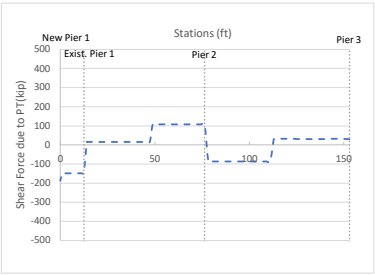
Force Diagrams

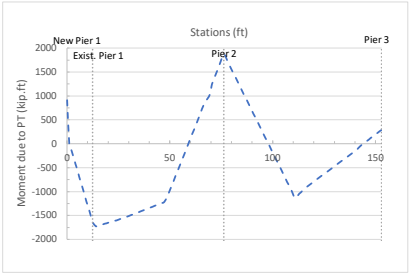
Unfactore Dead Load

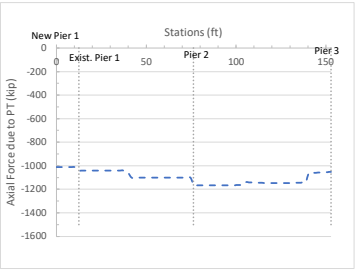




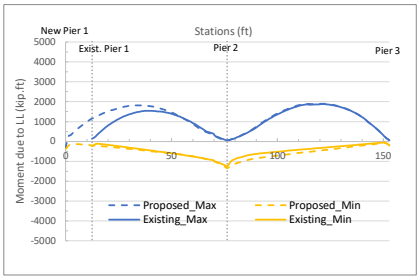
Unfactored Prestressing

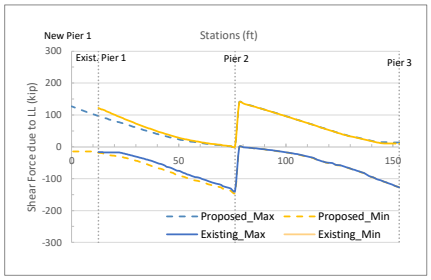




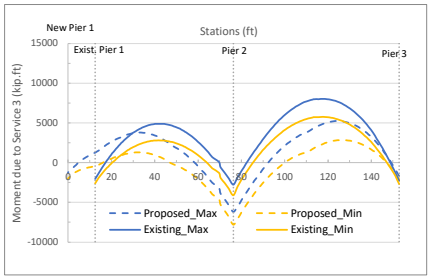


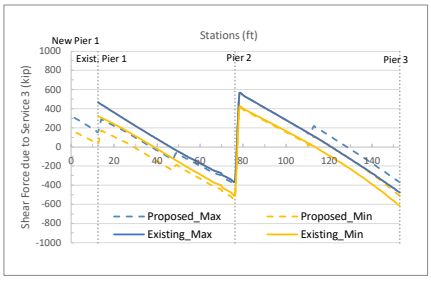
Unfactored Live Load



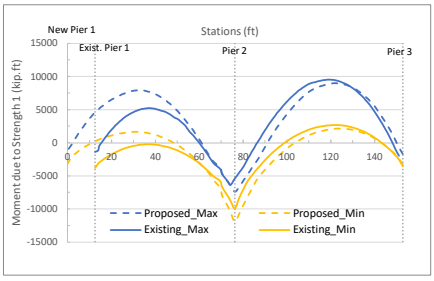


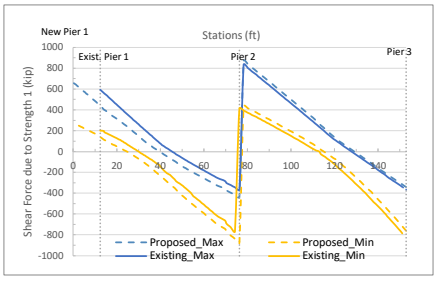
Service III Load Combination





Strength I Load Combination





8. Existing Structure Concrete Stresses for Uncracked Cross-section

COWI

Project I-405 Bridge 17.7 **Project No** A207833

Title Concrete Extreme Fiber Stresses for Proposed Structure Uncracked Section

PREPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

		Properties				Unfactored Dead Load: DL			Serviceability III Load Combination: SE3							Unfactored Live Load: LL					
	Station (ft)	y_top (ft)	y_bot (ft)	I(ft^4)	A(ft^2)	M	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	MMAX (kip.ft)	P	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	MMIN (kip.ft)	P	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	MMAX (kip.ft)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	MMIN (kip.ft)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)
Pier 3	152.93	1.65	2.60	128	53	-380	-0.05	0.03	-2281	-779	-0.43	0.10	-2683	-835	-0.49	0.13	59	0.01	-0.01	-194	-0.03	0.02
	151.20	1.65	2.60	128	53	420	0.06	-0.04	-1308	-779	-0.29	0.01	-1706	-835	-0.35	0.04	191	0.03	-0.02	-61	-0.01	0.01
	149.48	1.65	2.60	128	53	1172	0.17	-0.10	-332	-779	-0.15	-0.07	-910	-835	-0.24	-0.03	376	0.05	-0.03	-61	-0.01	0.01
	147.75	1.65	2.60	128	53	1877	0.27	-0.17	588	-779	-0.02	-0.16	-174	-835	-0.13	-0.09	552	0.08	-0.05	-73	-0.01	0.01
	146.02	1.65	2.60	128	53	2534	0.36	-0.23	1447	-779	0.10	-0.23	512	-835	-0.04	-0.16	714	0.10	-0.06	-88	-0.01	0.01
	144.29	1.65	2.60	128	53	3145	0.44	-0.28	2248	-779	0.22	-0.30	1151	-835	0.05	-0.21	865	0.12	-0.08	-104	-0.01	0.01
	142.57	1.65	2.60	128	53	3710	0.52	-0.33	2997	-779	0.32	-0.37	1744	-835	0.14	-0.27	1009	0.14	-0.09	-119	-0.02	0.01
	140.84	1.65	2.60	128	53	4350	0.62	-0.39	3787	-796	0.43	-0.44	2366	-851	0.22	-0.32	1176	0.17	-0.11	-138	-0.02	0.01
	138.91	1.65	2.60	128	53	4871	0.69	-0.44	4477	-796	0.53	-0.51	2917	-851	0.30	-0.37	1302	0.18	-0.12	-155	-0.02	0.01
	136.98	1.65	2.60	128	53	5341	0.76	-0.48	5112	-796	0.62	-0.56	3418	-851	0.37	-0.42	1423	0.20	-0.13	-173	-0.02	0.02
	135.05	1.65	2.60	128	53	5762	0.81	-0.52	5677	-796	0.70	-0.61	3869	-851	0.44	-0.46	1524	0.22	-0.14	-190	-0.03	0.02
	133.11	1.65	2.60	128	53	6132	0.87	-0.55	6182	-796	0.77	-0.66	4270	-851	0.49	-0.49	1615	0.23	-0.14	-208	-0.03	0.02
	131.18	1.65	2.60	128	53	6454	0.91	-0.58	6619	-796	0.83	-0.70	4622	-851	0.54	-0.53	1687	0.24	-0.15	-225	-0.03	0.02
	129.25	1.65	2.60	128	53	6727	0.95	-0.60	6999	-796	0.89	-0.73	4926	-851	0.58	-0.55	1750	0.25	-0.16	-243	-0.03	0.02
	127.32	1.65	2.60	128	53	6953	0.98	-0.62	7316	-796	0.93	-0.76	5181	-851	0.62	-0.58	1798	0.25	-0.16	-261	-0.04	0.02
	125.39	1.65	2.60	128	53	7131	1.01	-0.64	7578	-796	0.97	-0.78	5389	-851	0.65	-0.59	1838	0.26	-0.16	-278	-0.04	0.02
	123.46	1.65	2.60	128	53	7261	1.03	-0.65	7780	-796	1.00	-0.80	5550	-851	0.67	-0.61	1867	0.26	-0.17	-296	-0.04	0.03
	121.53	1.65	2.60	128	53	7346	1.04	-0.66	7920	-796	1.02	-0.81	5665	-851	0.69	-0.62	1880	0.27	-0.17	-313	-0.04	0.03
	119.59	1.65	2.60	128	53	7384	1.04	-0.66	7994	-796	1.03	-0.82	5733	-851	0.70	-0.63	1872	0.26	-0.17	-331	-0.05	0.03
East Span	117.66	1.65	2.60	128	53	7378	1.04	-0.66	8021	-796	1.03	-0.82	5756	-851	0.70	-0.63	1863	0.26	-0.17	-349	-0.05	0.03
	115.73	1.65	2.60	128	53	7326	1.04	-0.66	8005	-796	1.03	-0.82	5734	-851	0.70	-0.63	1856	0.26	-0.17	-366	-0.05	0.03
	113.80	1.65	2.60	128	53	7325	1.04	-0.66	7999	-804	1.03	-0.82	5730	-857	0.70	-0.63	1864	0.26	-0.17	-382	-0.05	0.03
	112.93	1.65	2.60	128	53	7270	1.03	-0.65	7950	-804	1.02	-0.82	5689	-857	0.69	-0.62	1851	0.26	-0.17	-390	-0.06	0.03
	111.01	1.65	2.60	128	53	7118	1.01	-0.64	7804	-804	1.00	-0.80	5570	-857	0.67	-0.61	1811	0.26	-0.16	-407	-0.06	0.04
	109.08	1.65	2.60	128	53	6926	0.98	-0.62	7605	-804	0.97	-0.79	5410	-857	0.65	-0.60	1759	0.25	-0.16	-424	-0.06	0.04
	107.16	1.65	2.60	128	53	6694	0.95	-0.60	7353	-804	0.93	-0.76	5210	-857	0.62	-0.58	1694	0.24	-0.15	-442	-0.06	0.04
	105.23	1.65	2.60	128	53	6422	0.91	-0.57	7046	-804	0.89	-0.74	4967	-857	0.59	-0.56	1615	0.23	-0.14	-460	-0.07	0.04
	103.31	1.65	2.60	128	53	6109	0.86	-0.55	6695	-804	0.84	-0.70	4682	-857	0.55	-0.53	1531	0.22	-0.14	-480	-0.07	0.04
	101.39	1.65	2.60	128	53	5756	0.81	-0.52	6297	-804	0.78	-0.67	4354	-857	0.50	-0.50	1436	0.20	-0.13	-500	-0.07	0.04
	99.46	1.65	2.60	128	53	5362	0.76	-0.48	5852	-804	0.72	-0.63	3985	-857	0.45	-0.47	1335	0.19	-0.12	-520	-0.07	0.05
	97.54	1.65	2.60	128	53	4928	0.70	-0.44	5356	-804	0.65	-0.59	3576	-857	0.39	-0.43	1224	0.17	-0.11	-540	-0.08	0.05
	95.62	1.65	2.60	128	53	4454	0.63	-0.40	4809	-804	0.57	-0.54	3127	-857	0.33	-0.39	1103	0.16	-0.10	-560	-0.08	0.05
	93.69	1.65	2.60	128	53	3940	0.56	-0.35	4214	-804	0.49	-0.48	2637	-857	0.26	-0.35	972	0.14	-0.09	-580	-0.08	0.05
	91.77	1.65	2.60	128	53	3385	0.48	-0.30	3569	-804	0.40	-0.43	2106	-857	0.19	-0.30	833	0.12	-0.07	-601	-0.08	0.05
	89.84	1.65	2.60	128	53	2790	0.39	-0.25	2878	-804	0.30	-0.36	1533	-857	0.10	-0.25	688	0.10	-0.06	-624	-0.09	0.06
	87.92	1.65	2.60	128	53	2154	0.30	-0.19	2164	-857	0.19	-0.31	908	-804	0.02	-0.19	560	0.08	-0.05	-658	-0.09	0.06
	86.00	1.65	2.60	128	53	1478	0.21	-0.13	1433	-857	0.09	-0.24	235	-804	-0.07	-0.13	456	0.06	-0.04	-700	-0.10	0.06
	84.07	1.65	2.60	128	53	762	0.11	-0.07	667	-857	-0.02	-0.17	-488	-804	-0.17	-0.06	357	0.05	-0.03	-751	-0.11	0.07
	82.15	1.65	2.60	128	53	6	0.00	0.00	-134	-857	-0.13	-0.10	-1259	-804	-0.28	0.01	262	0.04	-0.02	-810	-0.11	0.07
	80.22	1.65	2.60	128	53	-791	-0.11	0.07	-972	-857	-0.25	-0.03	-2077	-804	-0.40	0.08	173	0.02	-0.02	-877	-0.12	0.08
	78.30	1.65	2.60	128	53	-1629	-0.23	0.15	-1828	-857	-0.37	0.05	-2981	-804	-0.53	0.16	105	0.01	-0.01	-988	-0.14	0.09
Pier 2	76.30	1.65	2.60	128	53	-2507	-0.35	0.22	-2750	-836	-0.50	0.14	-4091	-807	-0.68	0.26	65	0.01	-0.01	-1238	-0.18	0.11
	74.30	1.61	2.64	124	52	-1749	-0.26	0.16	-1991	-836	-0.41	0.07	-3296	-807	-0.60	0.19	122	0.02	-0.01	-1150	-0.17	0.10
	72.39	1.61	2.64	124	52	-1056	-0.16	0.10	-1275	-836	-0.30	0.00	-2572	-807	-0.49	0.12	199	0.03	-0.02	-1072	-0.16	0.10
	70.49	1.61	2.64	124	52	-403	-0.06	0.04	-579	-836	-0.20	-0.06	-1897	-807	-0.39	0.06	295	0.04	-0.03	-1001	-0.15	0.09
	69.64	1.61	2.64	124	52	209	0.03	-0.02	81	-836	-0.10	-0.12	-1270	-807	-0.30	0.01	396	0.06	-0.04	-938	-0.14	0.08
	66.68	1.61	2.64	124	52	781	0.12	-0.07	706	-836	-0.01	-0.18	-692	-807	-0.21	-0.05	502	0.07	-0.05	-884	-0.13	0.08
	64.77	1.61	2.64	124	52	1313	0.19	-0.12	1306	-836	0.08	-0.23	-163	-807	-0.13	-0.09	623	0.09	-0.06	-840	-0.12	0.08
	62.86	1.61	2.64	124	52	1805	0.27	-0.16	1877	-836	0.17	-0.28	318	-807	-0.06	-0.14	756	0.11	-0.07	-803	-0.12	0.07
	60.96	1.61	2.64	124	52	2257	0.33	-0.20	2403	-836	0.24	-0.33	750	-807	0.00	-0.18	879	0.13	-0.08	-769	-0.11	0.07
	59.05	1.61	2.64	124	52	2668	0.40	-0.24	2879	-836	0.31	-0.37	1141	-807	0.06	-0.21	991	0.15	-0.09	-736	-0.11	0.07
	57.15	1.61	2.64	124	52	3039	0.45	-0.27	3306	-836	0.38	-0.41	1491	-807	0.11	-0.24	1096	0.16	-0.10	-703	-0.10	0.06
	55.24	1.61	2.64	124	52	3369	0.50	-0.30	3678	-836	0.43	-0.44	1800	-807	0.16	-0.27	1185	0.18	-0.11	-670	-0.10	0.06
	54.04	1.61	2.64	124	52	3558	0.53	-0.32	3900	-836	0.47	-0.46	1976	-807	0.18	-0.29	1247	0.18	-0.11	-650	-0.10	0.06
	52.84	1.61	2.64	124	52	3731	0.55	-0.34	4085	-837	0.49	-0.48	2136	-807	0.21	-0.30	1288	0.19	-0.12	-629	-0.09	0.06
	51.64	1.61	2.64	124	52	3888	0.58	-0.35	4266	-837	0.52	-0.50	2278	-807	0.23	-0.31	1341	0.20	-0.12	-610	-0.09	0.06
	50.44	1.61	2.64	124	52	4028	0.60	-0.36	4413	-836	0.54	-0.51	2402	-806	0.25	-0.32	1376	0.20	-0.12	-594	-0.09	0.05
	48.87	1.61	2.64	124	52	4188	0.62	-0.38	4585	-836	0.57	-0.53	2539	-806	0.27	-0.34	1427	0.21	-0.13	-573	-0.08	0.05
	47.30	1.61	2.64	124	52	4320	0.64	-0.39	4718	-836	0.59	-0.54	2648	-806	0.28	-0.35	1465	0.22	-0.13	-552	-0.08	0.05
	45.85	1.61	2.64	124	52	4421	0.65	-0.40	4811	-805	0.60	-0.54	2728	-836	0.29	-0.36	1493	0.22	-0.13	-532	-0.08	0.05
	44.40	1.61	2.64	124	52	4496	0.67	-0.41	4870	-804	0.61	-0.55	2782	-835	0.30	-0.36	1512	0.22	-0.14	-511	-0.08	0.05
	42.62	1.61	2.64	124	52	4554	0.67	-0.41	4893	-803	0.62	-0.55	2814	-834	0.30	-0.37	1522	0.23	-0.14	-486	-0.07	0.04
West Span	40.83	1.61	2.64	124	52	4574	0.68	-0.41	4886	-803	0.62	-0.55	2808	-834	0.30	-0.37	1539	0.23	-0.14	-461	-0.07	0.04
	39.63	1.61	2.64	124	52	4565	0.68	-0.41	4853	-802	0.61	-0.55	2782	-833	0.30	-0.36	1544	0.23	-0.14	-444	-0.07	0.04
	38.43	1.61	2.64	124	52	4539	0.67	-0.41	4797	-801	0.60	-0.54	2739	-832	0.29	-0.36	1543	0.23	-0.14	-427	-0.06	0.04
	37.23	1.61	2.64	124	52	4495	0.67	-0.41	4716	-800	0.59	-0.53	2679	-831	0.29	-0.35	1535	0.23	-0.14	-409	-0.06	0.04
	36.03	1.61	2.64	124	52	4434	0.66	-0.40	4612	-799	0.58	-0.52	2601	-831	0.27	-0.35	1521	0.23	-0.14	-392	-0.06	0.04
	34.83	1.61	2.64	124	52	4355	0.64	-0.39	4484	-798	0.56	-0.51	2506	-830	0.26	-0.34	1501	0.22	-0.14	-375	-0.06	0.03
	33.63	1.61	2.64	124	52	4259	0.63	-0.38	4332	-797	0.53	-0.50	2393	-829	0.24	-0.33	1474	0.22	-0.13	-358	-0.05	0.03
	32.43	1.61	2.64	124	52	4146	0.61	-0.37	4156	-796	0.51	-0.48	2264	-828	0.22	-0.32	1440	0.21	-0.13	-341	-0.05	0.03
	31.22	1.61	2.64	124	52	4015	0.59	-0.36	3961	-795	0.48	-0.46	2117	-827	0.20	-0.30	1404	0.21	-0.13	-324	-0.05	0.03
	30.02	1.61	2.64	124	52	3866	0.57	-0.35	3746	-794	0.45	-0.44	1952	-826	0.18	-0.29	1366	0.20	-0.12	-307	-0.05	0.03
	28.82	1.61	2.64	124	52	3700	0.55	-0.33	3507	-793	0.41	-0.42	1770	-824	0.15	-0.27	1321	0.20	-0.12	-290	-0.04	0.03
	27.62	1.61	2.64	124	52	3517	0.52	-0.32	3243	-791	0.37	-0.40	1571	-823	0.12	-0.25	1269	0.19	-0.11	-272	-0.04	0.02
	26.42	1.61	2.64	124	52	3316	0.49	-0.30	2957	-790	0.33	-0.37	1355	-822	0.09	-0.23	1211	0.18	-0.11	-255	-0.04	0.02
	25.22	1.61	2.64	124	52	3098	0.46	-0.28	2646	-788	0.29	-0.34	1121	-820	0.06	-0.21	1146	0.17	-0.10	-238	-0.04	0.02
	24.02	1.61	2.64	124	52	2863	0.42	-0.26	2311	-787	0.24	-0.31	870	-819	0.02	-0.19	1075	0.16	-0.10	-221	-0.03	0.02
	22.82	1.61	2.64	124	52	2610	0.39	-0.24	1954	-786	0.18	-0.28	602	-818	-0.02	-0.16	998	0.15	-0.09	-205	-0.03	0.02
	21.62	1.61	2.64	124	52	2340	0.35	-0.21	1574	-785	0.13	-0.25	316	-817	-0.06	-0.14	916	0.14	-0.08	-188	-0.03	0.02
	20.42	1.61	2.64	124	52	2053	0.30	-0.19	1172	-783	0.07	-0.21	13	-815	-0.11	-0.11	829	0.12	-0.07	-172	-0.03	0.02
	19.22	1.61	2.64	124	52	1748	0.26	-0.16	745	-782	0.01	-0.17	-307	-814	-0.15	-0.08	734	0.11	-0.07	-156	-0.02	0.01
	18.02	1.61	2.64	124	52	1427	0.21	-0.13	295	-781	-0.06	-0.13	-646	-813	-0.20	-0.05	632	0.09	-0.06	-141	-0.02	0.01
	16.82	1.61	2.64	124	52	1088	0.16	-0.10	-180	-779	-0.13	-0.09	-1003	-811	-0.26	-0.02	524	0.08	-0.05	-127	-0.02	0.01
	15.61	1.61	2.64	124	52	731	0.11	-0.07	-679	-778	-0.20	-0.04	-1381	-810	-0.31	0.02	408	0.06	-0.04	-118	-0.02	0.01
	14.41	1.61	2.64	124	52	360	0.05	-0.03	-1201	-776	-0.28	0.00	-1789	-809	-0.37	0.05	286	0.04	-0.03	-122	-0.02	0.01
	14.00	1.61	2.64	124	52	224	0.03	-0.02	-1390	-775	-0.31	0.02	-1949	-808	-0.40	0.07	239	0.04	-0.02	-137	-0.02	0.01
Existing Pier 1	12.51	1.61	2.64	124	52	-263	-0.04	0.02	-2017	-774	-0.40	0.08	-2587	-807	-0.49	0.13	140	0.02	-0.01	-244	-0.04	0.02

9. Proposed Structure Concrete Stresses for Uncracked Cross-section

COWI

Project I-405 Bridge 17.7
Title Uncracked Concrete Extreme Fiber Stresses for Proposed Structure
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

Stresses

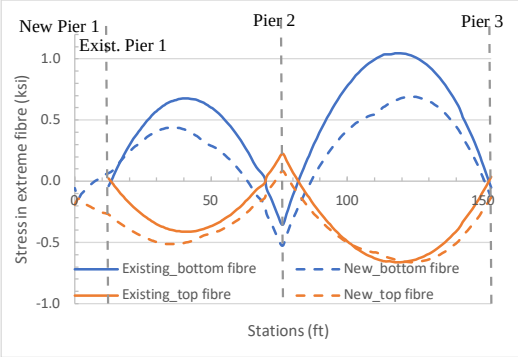
Unfactored Dead Load+Unfactored Prestressing | *Serviceability III Load Combination* | *Unfactored Live Load*

	Properties					DL+PT				SE3							LL					
	Station (ft)	y_top (ft)	y_bot (ft)	I(ft^4)	A(ft^2)	M (kip.ft)	P (kip)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	Mmax (kip.ft)	P (kip)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	Mmin (kip.ft)	P (kip)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	Mmax (kip.ft)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)	Mmin (kip.ft)	STRESS @ BOT FIBRE (ksi)	STRESS @ TOP FIBRE (ksi)
Pier 3	152.9	1.65	2.60	128	53	-10	-1012	-0.13	-0.13	-1660	-1757	-0.47	-0.08	-2089	-835	-0.41	0.08	69	0.01	-0.01	-200	-0.03	0.02
	151.2	1.65	2.60	128	53	687	-1012	-0.04	-0.19	-874	-1757	-0.35	-0.15	-1313	-835	-0.30	0.01	198	0.03	-0.02	-79	-0.01	0.01
	149.5	1.65	2.60	128	53	1335	-1012	0.06	-0.25	-83	-1756	-0.24	-0.22	-711	-835	-0.21	-0.05	383	0.05	-0.03	-85	-0.01	0.01
	147.7	1.65	2.60	128	53	1936	-1012	0.14	-0.31	650	-1756	-0.14	-0.29	-170	-835	-0.13	-0.09	557	0.08	-0.05	-104	-0.01	0.01
	146.0	1.65	2.60	128	53	2489	-1012	0.22	-0.36	1323	-1756	-0.04	-0.35	320	-835	-0.06	-0.14	718	0.10	-0.06	-125	-0.02	0.01
	144.3	1.65	2.60	128	53	2996	-1012	0.29	-0.40	1940	-1755	0.04	-0.40	764	-835	0.00	-0.18	870	0.12	-0.08	-147	-0.02	0.01
	142.6	1.65	2.60	128	53	3457	-1012	0.36	-0.44	2506	-1755	0.12	-0.46	1161	-835	0.05	-0.21	1016	0.14	-0.09	-170	-0.02	0.02
	140.8	1.65	2.60	128	53	3955	-1040	0.42	-0.49	3046	-1794	0.19	-0.51	1525	-851	0.10	-0.25	1185	0.17	-0.11	-192	-0.03	0.02
	138.9	1.65	2.60	128	53	4363	-1040	0.48	-0.53	3537	-1794	0.26	-0.55	1864	-851	0.15	-0.28	1312	0.19	-0.12	-217	-0.03	0.02
	137.0	1.65	2.60	128	53	4721	-1040	0.53	-0.56	3971	-1793	0.33	-0.59	2152	-851	0.19	-0.30	1433	0.20	-0.13	-242	-0.03	0.02
	135.0	1.65	2.60	128	53	5028	-1040	0.57	-0.59	4336	-1793	0.38	-0.62	2391	-851	0.23	-0.33	1536	0.22	-0.14	-266	-0.04	0.02
	133.1	1.65	2.60	128	53	5286	-1040	0.61	-0.61	4641	-1792	0.42	-0.65	2579	-851	0.25	-0.34	1628	0.23	-0.15	-291	-0.04	0.03
	131.2	1.65	2.60	128	53	5495	-1040	0.64	-0.63	4878	-1792	0.45	-0.67	2719	-851	0.27	-0.36	1702	0.24	-0.15	-316	-0.04	0.03
	129.3	1.65	2.60	128	53	5655	-1040	0.66	-0.64	5057	-1792	0.48	-0.69	2809	-851	0.29	-0.36	1766	0.25	-0.16	-341	-0.05	0.03
	127.3	1.65	2.60	128	53	5767	-1040	0.68	-0.65	5174	-1791	0.50	-0.70	2852	-851	0.29	-0.37	1816	0.26	-0.16	-365	-0.05	0.03
	125.4	1.65	2.60	128	53	5831	-1040	0.69	-0.66	5235	-1791	0.50	-0.70	2847	-851	0.29	-0.37	1858	0.26	-0.17	-390	-0.06	0.03
	123.5	1.65	2.60	128	53	5849	-1040	0.69	-0.66	5237	-1791	0.50	-0.70	2795	-851	0.28	-0.36	1888	0.27	-0.17	-415	-0.06	0.04
	121.5	1.65	2.60	128	53	5820	-1040	0.69	-0.66	5178	-1792	0.50	-0.70	2696	-851	0.27	-0.35	1903	0.27	-0.17	-440	-0.06	0.04
	119.6	1.65	2.60	128	53	5745	-1040	0.68	-0.65	5053	-1793	0.48	-0.69	2552	-851	0.25	-0.34	1898	0.27	-0.17	-465	-0.07	0.04
East Span	117.7	1.65	2.60	128	53	5624	-1040	0.66	-0.64	4875	-1794	0.45	-0.67	2361	-851	0.22	-0.32	1887	0.27	-0.17	-489	-0.07	0.04
	115.7	1.65	2.60	128	53	5459	-1040	0.64	-0.63	4659	-1796	0.42	-0.65	2126	-851	0.19	-0.30	1882	0.27	-0.17	-514	-0.07	0.05
	113.8	1.65	2.60	128	53	5314	-1060	0.61	-0.62	4404	-1820	0.38	-0.63	1868	-857	0.15	-0.28	1892	0.27	-0.17	-530	-0.07	0.05
	112.9	1.65	2.60	128	53	5210	-1060	0.60	-0.61	4268	-1821	0.36	-0.62	1734	-857	0.13	-0.27	1879	0.27	-0.17	-541	-0.08	0.05
	111.0	1.65	2.60	128	53	4917	-1102	0.55	-0.59	3907	-1853	0.31	-0.59	1390	-857	0.08	-0.24	1842	0.26	-0.16	-565	-0.08	0.05
	109.1	1.65	2.60	128	53	4841	-1102	0.54	-0.58	3713	-1854	0.28	-0.58	1244	-857	0.06	-0.22	1792	0.25	-0.16	-590	-0.08	0.05
	107.2	1.65	2.60	128	53	4725	-1102	0.52	-0.57	3466	-1855	0.25	-0.55	1058	-857	0.04	-0.21	1729	0.24	-0.15	-614	-0.09	0.05
	105.2	1.65	2.60	128	53	4569	-1102	0.50	-0.55	3164	-1857	0.20	-0.53	831	-857	0.00	-0.19	1651	0.23	-0.15	-639	-0.09	0.06
	103.3	1.65	2.60	128	53	4373	-1102	0.47	-0.54	2816	-1858	0.15	-0.50	564	-857	-0.03	-0.16	1568	0.22	-0.14	-663	-0.09	0.06
	101.4	1.65	2.60	128	53	4136	-1102	0.44	-0.52	2419	-1859	0.10	-0.46	255	-857	-0.08	-0.14	1475	0.21	-0.13	-688	-0.10	0.06
	99.5	1.65	2.60	128	53	3859	-1102	0.40	-0.49	1998	-1860	0.04	-0.42	-115	-857	-0.13	-0.10	1377	0.19	-0.12	-712	-0.10	0.06
	97.5	1.65	2.60	128	53	3541	-1102	0.36	-0.46	1525	-1861	-0.03	-0.38	-526	-857	-0.19	-0.07	1267	0.18	-0.11	-738	-0.10	0.07
	95.6	1.65	2.60	128	53	3183	-1102	0.31	-0.43	1001	-1863	-0.10	-0.33	-979	-857	-0.25	-0.03	1147	0.16	-0.10	-765	-0.11	0.07
	93.7	1.65	2.60	128	53	2785	-1102	0.25	-0.39	427	-1864	-0.19	-0.28	-1473	-857	-0.32	0.02	1017	0.14	-0.09	-792	-0.11	0.07
	91.8	1.65	2.60	128	53	2347	-1102	0.19	-0.36	-197	-1865	-0.27	-0.23	-2007	-857	-0.40	0.07	878	0.12	-0.08	-819	-0.12	0.07
	89.8	1.65	2.60	128	53	1868	-1102	0.12	-0.31	-869	-1866	-0.37	-0.17	-2582	-857	-0.48	0.12	731	0.10	-0.07	-847	-0.12	0.08
	87.9	1.65	2.60	128	53	1349	-1102	0.05	-0.27	-1580	-1867	-0.47	-0.10	-3204	-804	-0.56	0.18	585	0.08	-0.05	-883	-0.12	0.08
	86.0	1.65	2.60	128	53	789	-1102	-0.03	-0.22	-2295	-1868	-0.57	-0.04	-3878	-804	-0.65	0.24	476	0.07	-0.04	-929	-0.13	0.08
	84.1	1.65	2.60	128	53	189	-1102	-0.12	-0.16	-3044	-1870	-0.68	0.03	-4602	-804	-0.76	0.31	373	0.05	-0.03	-984	-0.14	0.09
	82.1	1.65	2.60	128	53	-451	-1102	-0.21	-0.10	-3829	-1871	-0.79	0.10	-5374	-804	-0.87	0.37	274	0.04	-0.02	-1048	-0.15	0.09
	80.2	1.65	2.60	128	53	-1131	-1102	-0.31	-0.04	-4650	-1872	-0.90	0.17	-6194	-804	-0.98	0.45	180	0.03	-0.02	-1120	-0.16	0.10
	78.3	1.65	2.60	128	53	-1852	-1102	-0.41	0.02	-5493	-1873	-1.02	0.24	-7062	-804	-1.10	0.53	104	0.01	-0.01	-1199	-0.17	0.11
Pier 2	76.3	1.65	2.60	128	53	-2648	-1166	-0.53	0.08	-6168	-1792	-1.11	0.32	-7777	-807	-1.21	0.59	58	0.01	-0.01	-1341	-0.19	0.12
	74.3	1.61	2.64	124	52	-1960	-1166	-0.45	0.02	-5364	-1790	-1.03	0.24	-6838	-807	-1.12	0.51	111	0.02	-0.01	-1178	-0.17	0.11
	72.4	1.61	2.64	124	52	-1335	-1166	-0.35	-0.04	-4616	-1790	-0.92	0.18	-6044	-807	-1.00	0.44	174	0.03	-0.02	-1094	-0.16	0.10
	70.5	1.61	2.64	124	52	-749	-1166	-0.27	-0.09	-3883	-1789	-0.81	0.11	-5298	-807	-0.89	0.37	262	0.04	-0.02	-1018	-0.15	0.09
	69.6	1.61	2.64	124	52	-204	-1166	-0.19	-0.14	-3183	-1788	-0.71	0.05	-4599	-807	-0.79	0.31	357	0.05	-0.03	-949	-0.14	0.09
	66.7	1.61	2.64	124	52	300	-1166	-0.11	-0.18	-2518	-1788	-0.61	-0.01	-3948	-807	-0.69	0.25	457	0.07	-0.04	-888	-0.13	0.08
	64.8	1.61	2.64	124	52	765	-1166	-0.04	-0.23	-1890	-1789	-0.52	-0.07	-3346	-807	-0.60	0.19	560	0.08	-0.05	-836	-0.12	0.08
	62.9	1.61	2.64	124	52	1189	-1166	0.02	-0.26	-1288	-1790	-0.43	-0.12	-2792	-807	-0.52	0.14	678	0.10	-0.06	-792	-0.12	0.07
	61.0	1.61	2.64	124	52	1573	-1166	0.08	-0.30	-705	-1790	-0.34	-0.18	-2287	-807	-0.45	0.10	817	0.12	-0.07	-755	-0.11	0.07
	59.1	1.61	2.64	124	52	1917	-1166	0.13	-0.33	-155	-1791	-0.26	-0.23	-1842	-807	-0.38	0.06	948	0.14	-0.09	-724	-0.11	0.07
	57.1	1.61	2.64	124	52	2221	-1166	0.17	-0.36	358	-1792	-0.19	-0.27	-1450	-807	-0.32	0.02	1073	0.16	-0.10	-696	-0.10	0.06
	55.2	1.61	2.64	124	52	2485	-1166	0.21	-0.38	817	-1793	-0.12	-0.31	-1099	-807	-0.27	-0.01	1183	0.18	-0.11	-668	-0.10	0.06
	54.0	1.61	2.64	124	52	2634	-1165	0.23	-0.39	1097	-1794	-0.08	-0.34	-896	-807	-0.24	-0.03	1260	0.19	-0.11	-650	-0.10	0.06
	52.8	1.61	2.64	124	52	2767	-1165	0.25	-0.41	1341	-1794	-0.04	-0.36	-709	-807	-0.21	-0.04	1317	0.19	-0.12	-633	-0.09	0.06
	51.6	1.61	2.64	124	52	2885	-1164	0.27	-0.42	1583	-1794	0.01	-0.38	-538	-807	-0.19	-0.06	1387	0.21	-0.13	-617	-0.09	0.06
	50.4	1.61	2.64	124	52	2986	-1163	0.29	-0.43	1791	-1794	0.02	-0.40	-386	-806	-0.17	-0.07	1440	0.21	-0.13	-604	-0.09	0.05
	48.9	1.61	2.64	124	52	3094	-1162	0.30	-0.44	2044	-1794	0.06	-0.42	-212	-806	-0.14	-0.09	1514	0.22	-0.14	-587	-0.09	0.05
	47.3	1.61	2.64	124	52	3220	-1139	0.32	-0.44	2298	-1770	0.10	-0.44	-17	-806	-0.11	-0.11	1572	0.23	-0.14	-568	-0.08	0.05
	45.9	1.61	2.64	124	52	3407	-1140	0.35	-0.46	2591	-1773	0.15	-0.47	228	-836	-0.08	-0.13	1628	0.24	-0.15	-552	-0.08	0.05
	44.4	1.61	2.64	124	52	3568	-1142	0.38	-0.48	2852	-1775	0.18	-0.50	447	-835	-0.05	-0.15	1676	0.25	-0.15	-535	-0.08	0.05
	42.6	1.61	2.64	124	52	3732	-1143	0.40	-0.49	3127	-1777	0.22	-0.52	683	-834	-0.01	-0.17	1724	0.26	-0.16	-515	-0.08	0.05
West Span	40.8	1.61	2.64	124	52	3857	-1144	0.42	-0.50	3354	-1779	0.26	-0.54	882	-834	0.02	-0.19	1761	0.26	-0.16	-495	-0.07	0.04
	39.6	1.61	2.64	124	52	3920	-1145	0.43	-0.51	3477	-1780	0.28	-0.55	993	-833	0.04	-0.20	1778	0.26	-0.16	-481	-0.07	0.04
	38.4	1.61	2.64	124	52	3965	-1146	0.43	-0.51	3578	-1781	0.29	-0.56	1088	-832	0.05	-0.21	1790	0.27	-0.16	-467	-0.07	0.04
	37.2	1.61	2.64	124	52	3993	-1146	0.44	-0.51	3658	-1782	0.30	-0.57	1166	-831	0.06	-0.22	1799	0.27	-0.16	-453	-0.07	0.04
	36.0	1.61	2.64	124	52	4004	-1147	0.44	-0.52	3725	-1783	0.31	-0.58	1226	-831	0.07	-0.22	1812	0.27	-0.16	-439	-0.07	0.04
	34.8	1.61	2.64	124	52	3997	-1147	0.44	-0.51	3769	-1784	0.32	-0.58	1269	-830	0.08	-0.23	1819	0.27	-0.16	-425	-0.06	0.04
	33.6	1.61	2.64	124	52	3972	-1147	0.43	-0.51	3791	-1784	0.32	-0.58	1295	-829	0.08	-0.23	1821	0.27	-0.16	-412	-0.06	0.04
	32.4	1.61	2.64	124	52	3930	-1147	0.43	-0.51	3789	-1785	0.32	-0.58	1303	-828	0.08	-0.23	1817	0.27	-0.16	-398	-0.06	0.04
	31.2	1.61	2.64	124	52	3871	-1148	0.42	-0.50	3765	-1785	0.32	-0.58	1294	-827	0.08	-0.23	1808	0.27	-0.16	-384	-0.06	0.03
	30.0	1.61	2.64	124	52	3794	-1148	0.41	-0.50	3717	-1785	0.31	-0.57	1268	-826	0.08	-0.23	1793	0.27	-0.16	-370	-0.05	0.03
	28.8	1.61	2.64	124	52	3700	-1147	0.39	-0.49	3647	-1785	0.30	-0.57	1225	-824	0.07	-0.22	1772	0.26	-0.16	-356	-0.05	0.03
	27.6	1.61	2.64	124	52	3588	-1147	0.38	-0.48	3553	-1785	0.29	-0.56	1165	-823	0.06	-0.22	1745	0.26	-0.16	-342	-0.05	0.03
	26.4	1.61	2.64	124	52	3459	-1147	0.36	-0.47	3436	-1784	0.27	-0.55	1087	-822	0.05	-0.21	1712	0.25	-0.15	-328	-0.05	0.03
	25.2	1.61	2.64	124	52	3313	-1147	0.34	-0.45	3296	-1784	0.25	-0.54	992	-820	0.04	-0.20	1674	0.25	-0.15	-314	-0.05	0.03
	24.0	1.61	2.64	124	52	3180	-1146	0.32	-0.44	3168	-1783	0.23	-0.52	901	-819	0.02	-0.19	1641	0.24	-0.15	-302	-0.04	0.03
	22.8	1.61	2.64	124	52	3035	-1146	0.30	-0.43	3026	-1783	0.21	-0.51	799	-818	0.01	-0.18	1607	0.24	-0.15	-291	-0.04	0.03
	21.6	1.61	2.64	124	52	2878	-1146	0.27	-0.41	2869	-1782	0.19	-0.50	685	-817	-0.01	-0.17	1569	0.23	-0.14	-279	-0.04	0.03
	20.4	1.61	2.64	124	52	2710	-1145	0.25	-0.40	2696	-1781	0.16	-0.48	560	-815	-0.03	-0.16	1526	0.23	-0.14	-268	-0.04	0.02
	19.2	1.61	2.64	124	52	2532	-1145	0.22	-0.38	2509	-1780	0.13	-0.46	424	-814	-0.05	-0.15	1480	0.22	-0.13	-256	-0.04	0.02
	18.0	1.61	2.64	124	52	2342	-1144	0.19	-0.36	2307	-1779	0.10	-0.45	277	-813	-0.07	-0.13	1432	0.21	-0.13	-245	-0.04	0.02
	16.8	1.61	2.64	124	52	2140	-1143	0.16	-0.35	2088	-1778	0.07	-0.43	119	-811	-0.09	-0.12	1376	0.20	-0.12	-233	-0.03	0.02
	15.6	1.61	2.64	124	52	1928	-1142	0.13	-0.33	1856	-1777	0.04	-0.41	-51	-810	-0.12	-0.10	1319	0.20	-0.12	-222	-0.03	0.02
	14.4	1.61	2.64	124	52	1704	-1142	0.10	-0.31	1608	-1776	0.00	-0.38	-234	-809	-0.14	-0.09	1258	0.19	-0.11	-211	-0.03	0.02
iting Pier 1	14.0	1.61	2.64	124	52	1620	-1141	0.09	-0.30	1508	-1775	-0.01	-0.37	-301	-808	-0.15	-0.08	1227	0.18	-0.11	-207	-0.03	0.02
	12.5	1.61	2.64	124	52	1421	-1062	0.07	-0.27	1279	-1698	-0.04	-0.34	-461	-807	-0.18	-0.07	1160	0.17	-0.10	-195	-0.03	0.02
	11.5	1.61	2.64	124	52	1348	-1061	0.06	-0.26	1160	-1697	-0.06	-0.33	-495	-806	-0.18	-0.06	1094	0.16	-0.10	-185	-0.03	0.02
West Span Extension	10.6	1.61	2.64	124	52	1269	-1061	0.05	-0.26	1033	-1695	-0.07	-0.32	-537	-805	-0.19	-0.06	1028	0.15	-0.09	-175	-0.03	0.02
	9.6	1.61	2.64	124	52	1183	-1060	0.03	-0.25	895	-1694	-0.09	-0.31	-588	-804	-0.19	-0.05	959	0.14	-0.09	-166	-0.02	0.01
	8.4	1.61	2.64	124	52	1061	-1059	0.02	-0.24	701	-1692	-0.12	-0.29	-667	-803	-0.21	-0.05	867	0.13	-0.08	-155	-0.02	0.01
	7.2	1.61	2.64	124	52	926	-1058	0.00	-0.23	488	-1690	-0.15	-0.27	-762	-802	-0.22	-0.04	770	0.11	-0.07	-144	-0.02	0.01
	6.0	1.61	2.64	124	52	778	-1057	-0.03	-0.21	254	-1689	-0.19	-0.25	-873	-801	-0.24	-0.03	666	0.10	-0.06	-136	-0.02	0.01
	4.8	1.61	2.64	124	52	617	-1057	-0.05	-0.20	0	-1687	-0.23	-0.23	-1002	-800	-0.26	-0.02	558	0.08	-0.05	-130	-0.02	0.01
	3.6	1.61	2.64	124	52	436	-1055	-0.08	-0.18	-286	-1685	-0.27	-0.20	-1162	-799	-0.28	0.00	445	0.07	-0.04	-130	-0.02	0.01
	2.4	1.61	2.64	124	52	234	-1054	-0.11	-0.16	-605	-1682	-0.31	-0.17	-1372	-798	-0.31	0.02	327	0.05	-0.03	-150	-0.02	0.01
	1.2	1.61	2.64	124	52	18	-1053	-0.14	-0.14	-909	-1680	-0.36	-0.14	-1654	-797	-0.35	0.04	243	0.04	-0.02	-224	-0.03	0.02
Pier 1	0.0	1.61	2.64	124	52	580	-1053	-0.06	-0.19	-1227	-1680	-0.41	-0.11	-1976	-796	-0.40	0.07	-322	-0.05	0.03	-390	-0.06	0.04

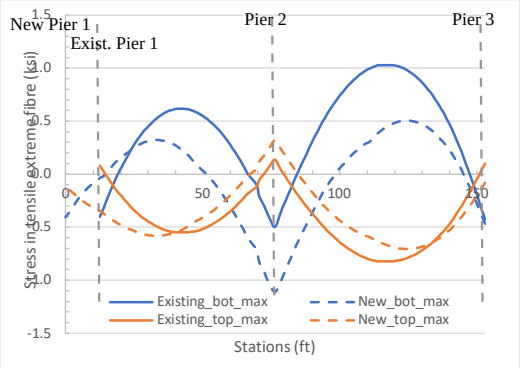
10. Proposed and Existing Structure Concrete Stresses for Uncracked Cross-section Comparison

Stresses for Uncracked Cross-section Plots

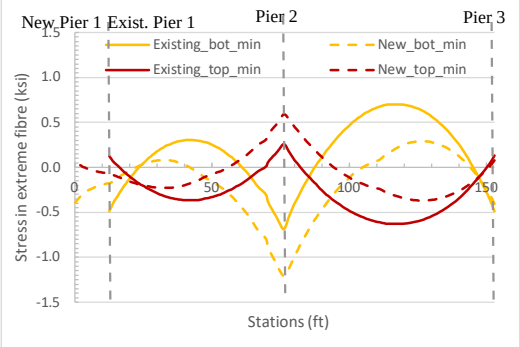
Stresses due to DL+PT or DL



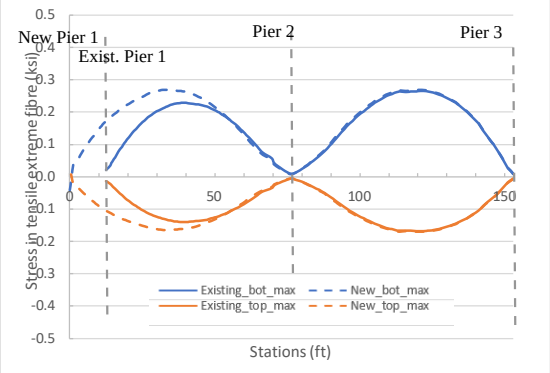
Stresses due to Serviceability III Max Moment



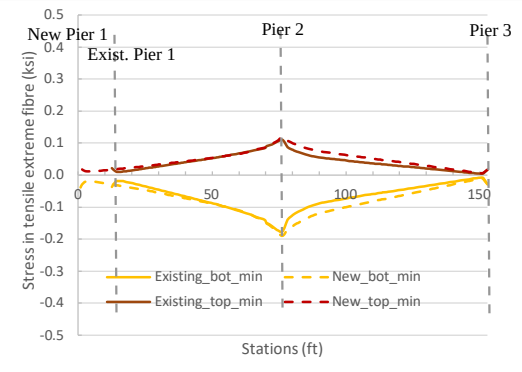
Stresses due to Serviceability III Min Moment



Stresses due to Unfactored Live Load Max Moment



Stresses due to Unfactored Live Load Min Moment



11. Proposed and Existing Structure Reinforcement Tensile Stresses for Cracked Cross-section

COWI

Project I-405 Bridge 17.7 **Project No** A207833

Title Reinforcement Tensile Stress in Cracked Section

PREPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

Description: This sheet summarized the maximum rebar tensile stresses in cracked section at east span, pier, west span, and interface between the new and existing structure when the structure is subjected to unfactored dead load, dead load and tendon forces, or serviceability load combination III. The stresses are computed using RESPONSE 2000.

EXISTING STRUCTURE

DC

	Station (ft)	M (kip.ft)	Stress (ksi)
east	121.53	7346	20
pier	76.3	-2506.62	9.1
west	40.83	4573.68	21.6

SERVICE LIMIT STATE III

	Station (ft)	M (kip.ft)	P (kip)	Stress (ksi)
east	117.67	8021.27	-759	19.20
pier	76.3	-4091.43	-734	9
west	42.93	4893.36	-696	18.9

see Appendix 2 for RESPONSE 2000 stress calculations

NEW STRUCTURE

DC+PT

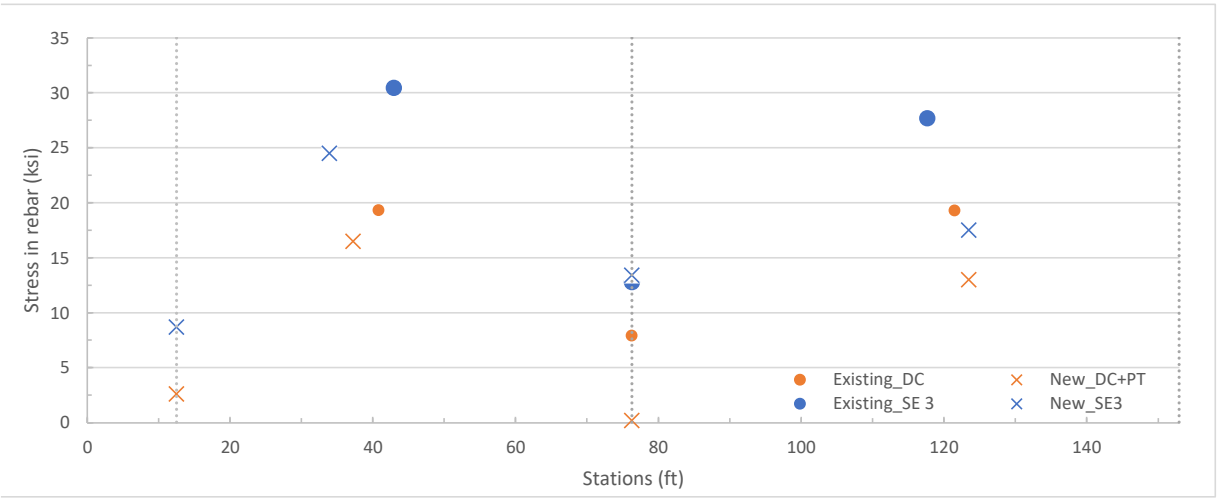
	Station (ft)	M (kip.ft)	P (kip)	Stress (ksi)
east	123.46	5848.88	-1064	12.2
pier	76.3	-2648.23	-1105	1.2
west	37.23	3993.21	-1169	11.8
interface	12.5	1420.56	-1161	0.5

see Appendix 2 for RESPONSE 2000 stress calculations

SERVICE LIMIT STATE III

	Station (ft)	M (kip.ft)	P (kip)	Stress (ksi)
east	123.46	5237.35	-1607	8.70
pier	76.3	-6167.73	-1616	9.6
west	33.925	3790.60	-1507	8.9
interface	12.5	1279.12	-1477	compression

Stress Plots



Appendix 1 - Existing Structure Shear and Flexure D/C Calculations

COWI

Project I-405 Bridge 17.7 **Project No** A207833

Title Existing Structure Shear Capacity Inputs

PREPARED BY LDLI **Date** 11/26/2021

CHECKED BY CUPG **Date** 11/26/2021

Description:
This sheet summarized the inputs of the shear capacity check for Bridge 17.7 **existing** superstructure. The demands are obtained from csibridge load combination 'SUP_STRENGTH ENVELOPE', which is the governing demands in all strength and extreme limit state combinations

Notation Definition:

$V_{2\ max}$	Maximum Vertical Shear Force Demand at the Cross-section	V_p	Tendon Shear Force	d_{s_bot}tom	Vertical Distance from Centroid of Reinforcements in Bottom Slab to NA
$V_{2\ min}$	Minimum Vertical Shear Force Demand at the Cross-section	y_{tendon}	Vertical Distance from Top of Superstructure to Tendon	A_{s_bottom}	Area of Reinforcement in Bottom Slab
T	Torsion Demand	s_{ext}	Trans. Reinf. Spacing in Exterior Webs	P_e	Tendon Force in Service
$M_{3\ max}$	Corresponding Moment about Longitudinal Axis when V2 is max	s_{int}	Trans. Reinf. Spacing in Interior Webs		
$M_{3\ min}$	Corresponding Moment about Longitudinal Axis when V2 is min	d_{s_top}	Vertical Distance from Centroid of Reinforcements in Top Slab to NA		
P	Axial Force Demand	A_{s_top}	Area of Reinforcement in Top Slab		

	Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	~~V_p (kip)~~	~~y_{tendon} (ft)~~	s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
Pier 3	152.93	-364	-786	-244	-1150	-2005	-3475	-664	-1090	~~0~~	~~0~~	30.0	30.0	0.6	0.2	4.0	0.2	0
	151.20	-339	-744	-269	-1238	-1267	-2244	-665	-1089	~~0~~	~~0~~	6.0	8.0	0.6	0.2	4.0	0.2	0
	149.48	-315	-702	-294	-1321	-310	-1360	-667	-1088	~~0~~	~~0~~	6.0	8.0	0.6	0.2	4.0	0.2	0
	147.75	-291	-661	-319	-1398	577	-533	-669	-1088	~~0~~	~~0~~	6.0	8.0	0.6	0.2	4.0	0.2	0
	146.02	-267	-621	-345	-1468	1397	239	-671	-1087	~~0~~	~~0~~	6.0	8.0	0.6	0.2	4.0	0.2	0
	144.29	-241	-581	-578	-1529	2151	1836	-682	-1086	~~0~~	~~0~~	6.0	8.0	0.6	0.2	4.0	0.2	0
	142.57	-215	-543	-635	-1581	2849	2663	-684	-1086	~~0~~	~~0~~	6.0	8.0	0.6	0.2	4.0	0.2	0
	140.84	-187	-502	-400	-1170	3562	3590	-701	-1106	~~0~~	~~0~~	8.0	13.0	0.6	0.2	4.0	0.2	0
	138.91	-160	-463	-437	-1220	4177	4368	-703	-1106	~~0~~	~~0~~	8.0	13.0	0.6	0.2	4.0	0.7	0
	136.98	-131	-416	-477	-1276	4613	5102	-706	-1104	~~0~~	~~0~~	8.0	13.0	0.6	0.2	4.0	0.7	0
	135.05	-103	-376	-513	-1307	5067	5747	-708	-1104	~~0~~	~~0~~	8.0	13.0	0.6	0.2	4.0	0.7	0
	133.11	-75	-337	-548	-1326	5451	6318	-710	-1103	~~0~~	~~0~~	13.0	15.0	0.6	0.2	4.0	0.7	0
	131.18	-48	-298	-582	-1340	5765	6809	-712	-1102	~~0~~	~~0~~	13.0	15.0	0.6	0.2	4.0	0.7	0
	129.25	-20	-260	-616	-1353	6012	7224	-714	-1102	~~0~~	~~0~~	15.0	15.0	0.6	0.2	4.0	0.7	0
	127.32	8	-222	-650	-1360	6190	7587	-719	-1101	~~0~~	~~0~~	15.0	15.0	0.6	0.2	4.0	0.7	0
	125.39	36	-185	-689	-1367	6301	7893	-720	-1102	~~0~~	~~0~~	15.0	15.0	0.6	0.2	4.0	0.7	0
	123.46	65	-153	-728	-1382	6459	8125	-720	-1104	~~0~~	~~0~~	15.0	15.0	0.6	0.2	4.0	0.7	0
	121.53	93	-112	-772	-1358	6329	8302	-719	-1106	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	119.59	121	-83	-811	-1379	6445	8354	-719	-1107	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
East Span	117.66	154	-52	-1254	-973	8899	5771	-725	-1103	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	115.73	190	-25	-1304	-960	8695	5707	-724	-1105	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	113.80	224	2	104	-23	8521	5567	-731	-1119	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	112.93	240	16	84	-31	8427	5345	-730	-1120	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	111.01	273	40	58	-40	8135	5068	-729	-1122	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	109.08	307	64	33	-49	7769	4742	-729	-1124	~~0~~	~~0~~	15.0	15.0	0.6	0.4	4.0	0.7	0
	107.16	340	88	14	-69	7331	4369	-728	-1126	~~0~~	~~0~~	13.0	15.0	0.6	0.4	4.0	0.7	0
	105.23	374	112	-4	-88	6820	3954	-727	-1128	~~0~~	~~0~~	13.0	15.0	0.6	0.4	4.0	0.7	0
	103.31	407	135	-24	-108	6239	3510	-726	-1130	~~0~~	~~0~~	8.0	13.0	0.6	0.4	4.0	0.7	0
	101.39	440	158	-43	-126	5587	3041	-726	-1132	~~0~~	~~0~~	8.0	13.0	0.6	0.4	4.0	0.7	0
	99.46	474	180	-63	-144	4866	2530	-725	-1134	~~0~~	~~0~~	8.0	13.0	0.6	0.4	4.0	0.7	0
	97.54	507	202	-85	-162	4073	1978	-725	-1136	~~0~~	~~0~~	8.0	13.0	0.6	0.4	4.0	0.7	0
	95.62	540	224	-108	-180	3212	1392	-723	-1138	~~0~~	~~0~~	8.0	8.0	0.6	0.4	4.0	0.7	0
	93.69	574	246	-131	-198	2282	763	-723	-1141	~~0~~	~~0~~	8.0	8.0	0.6	0.4	4.0	0.7	0
	91.77	607	268	-154	-215	1285	96	-723	-1143	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.7	0
	89.84	640	290	-178	-233	221	-609	-723	-1145	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.7	0
	87.92	672	311	-201	-250	-810	-1448	-723	-1147	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.4	0
	86.00	705	332	-225	-267	-1859	-2367	-723	-1149	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.4	0
	84.07	738	353	-249	-284	-2957	-3336	-723	-1151	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.4	0
	82.15	771	374	-273	-301	-4101	-4354	-723	-1153	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.4	0
	80.22	803	395	-298	-318	-5265	-5418	-720	-1155	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.4	0
	78.30	835	415	-322	-334	-6466	-6529	-720	-1157	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.4	0
Pier 2	76.30	-364	-766	373	303	-6327	-9008	-1074	-706	~~0~~	~~0~~	30.0	30.0	0.6	0.6	4.0	0.4	0
	74.30	-343	-727	355	264	-5518	-7747	-1074	-702	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.3	0

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Project I-405 Bridge 17.7 **Project No** A207833

Title Existing Structure Shear Capacity Inputs

PREPARED BY LDLI **Date** 11/26/2021

CHECKED BY CUPG **Date** 11/26/2021

Description:
This sheet summarized the inputs of the shear capacity check for Bridge 17.7 **existing** superstructure. The demands are obtained from csibridge load combination 'SUP_STRENGTH ENVELOPE', which is the governing demands in all strength and extreme limit state combinations

Notation Definition:

$V_{2\ max}$	Maximum Vertical Shear Force Demand at the Cross-section	V_p	Tendon Shear Force	d_{s_bottom}	Vertical Distance from Centroid of Reinforcements in Bottom Slab to NA
$V_{2\ min}$	Minimum Vertical Shear Force Demand at the Cross-section	y_{tendon}	Vertical Distance from Top of Superstructure to Tendon	A_{s_bottom}	Area of Reinforcement in Bottom Slab
T	Torsion Demand	s_{ext}	Trans. Reinf. Spacing in Exterior Webs	P_e	Tendon Force in Service
$M_{3\ max}$	Corresponding Moment about Longitudinal Axis when V2 is max	s_{int}	Trans. Reinf. Spacing in Interior Webs		
$M_{3\ min}$	Corresponding Moment about Longitudinal Axis when V2 is min	d_{s_top}	Vertical Distance from Centroid of Reinforcements in Top Slab to NA		
P	Axial Force Demand	A_{s_top}	Area of Reinforcement in Top Slab		

	Station (ft)	V_{2max} (kip)	V_{2min} (kip)	T for V_{umax} (kip.ft)	T for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P for V_{umax} (kip)	P for V_{umin} (kip)	~~V_p (kip)~~	~~y_{tendon} (ft)~~	s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	P_e (kip)
	72.39	-322	-693	340	238	-4754	-6475	-1076	-703	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.3	0
	70.49	-301	-660	324	213	-3992	-5269	-1075	-703	~~0~~	~~0~~	6.0	8.0	0.6	0.6	4.0	0.3	0
	68.58	-280	-627	307	191	-3265	-4130	-1073	-703	~~0~~	~~0~~	8.0	8.0	0.6	0.6	4.0	0.3	0
	66.68	-258	-593	291	170	-2573	-3058	-1071	-703	~~0~~	~~0~~	8.0	8.0	0.6	0.6	4.0	0.3	0
	64.77	-236	-560	274	194	-1918	-2050	-1071	-703	~~0~~	~~0~~	8.0	13.0	0.6	0.6	4.0	0.3	0
	62.86	-214	-526	258	176	-1317	-1117	-1069	-705	~~0~~	~~0~~	8.0	13.0	0.6	0.6	4.0	0.3	0
	60.96	-191	-492	241	160	-738	-255	-1068	-706	~~0~~	~~0~~	8.0	13.0	0.6	0.6	4.0	0.3	0
	59.05	-169	-458	223	144	-199	534	-1066	-707	~~0~~	~~0~~	13.0	13.0	0.6	0.3	4.0	0.3	0
	57.15	-146	-429	205	128	291	1329	-1066	-708	~~0~~	~~0~~	13.0	15.0	0.6	0.3	4.0	0.3	0
	55.24	-125	-394	185	103	720	1937	-1064	-709	~~0~~	~~0~~	13.0	15.0	0.6	0.3	4.0	0.4	0
	54.04	-110	-374	182	101	1020	2320	-1063	-710	~~0~~	~~0~~	13.0	15.0	0.6	0.3	4.0	0.4	0
	52.84	-96	-351	186	110	1250	2616	-1062	-711	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	51.64	-80	-331	192	122	1491	2959	-1061	-712	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	50.44	-66	-307	201	140	1661	3214	-1059	-712	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	48.87	-45	-283	212	163	1958	3602	-1058	-714	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	47.30	-25	-251	223	192	2194	3804	-1056	-715	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	45.85	-5	-224	238	229	2391	4013	-1054	-715	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	44.40	15	-200	291	281	2543	4234	-1052	-714	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	42.62	40	-163	325	315	2706	4270	-1049	-714	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	40.83	65	-132	355	345	2801	4393	-1047	-713	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	39.63	86	-114	513	234	4436	2801	-1046	-712	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	38.43	107	-97	548	246	4439	2791	-1044	-712	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	37.23	129	-80	584	256	4410	2755	-1042	-711	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	36.03	151	-63	618	255	4350	2728	-1038	-711	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	34.83	173	-47	652	265	4257	2668	-1036	-711	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	33.63	195	-31	684	273	4131	2585	-1034	-710	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	32.43	217	-15	714	279	3973	2479	-1031	-710	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	31.22	240	1	741	282	3781	2350	-1029	-709	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	30.02	262	17	766	282	3555	2198	-1026	-708	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	28.82	284	33	787	280	3296	2023	-1024	-708	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	27.62	307	48	805	275	3003	1825	-1021	-707	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	26.42	329	64	818	265	2676	1605	-1018	-706	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	25.22	352	80	826	253	2314	1362	-1015	-705	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	24.02	375	96	829	236	1917	1098	-1012	-704	~~0~~	~~0~~	16.0	15.0	0.6	0.3	4.0	0.4	0
	22.82	397	112	827	-26	1485	-190	-1000	-703	~~0~~	~~0~~	16.0	13.0	0.6	0.3	4.0	0.3	0
	21.62	420	124	818	-37	1019	-356	-997	-702	~~0~~	~~0~~	16.0	13.0	0.6	0.3	4.0	0.3	0
	20.42	443	137	803	-49	517	-536	-994	-701	~~0~~	~~0~~	16.0	13.0	0.6	0.3	4.0	0.3	0
	19.22	466	149	781	-63	-20	-730	-991	-700	~~0~~	~~0~~	13.0	13.0	0.6	0.3	4.0	0.3	0
	18.02	489	162	751	-79	-593	-938	-988	-699	~~0~~	~~0~~	8.0	13.0	0.6	0.3	4.0	0.3	0
	16.82	512	174	713	-96	-1201	-1161	-984	-697	~~0~~	~~0~~	8.0	13.0	0.6	0.3	4.0	0.3	0
	15.61	535	186	667	-115	-1844	-1397	-981	-697	~~0~~	~~0~~	8.0	13.0	0.6	0.3	4.0	0.3	0
	14.41	559	199	619	-132	-2508	-1661	-978	-697	~~0~~	~~0~~	8.0	13.0	0.6	0.3	4.0	0.3	0
	14.00	565	203	597	-145	-2651	-1854	-976	-696	~~0~~	~~0~~	8.0	13.0	0.6	0.3	4.0	0.3	0
Exist Pier 1	12.51	594	218	541	-152	-3189	-2542	-976	-697	~~0~~	~~0~~	30.0	30.0	0.6	0.3	4.0	0.3	0

West Span

COWI

Project I-405 Bridge 17.7 **Project No** A207833

Title Existing Structure Shear Capacity Check Sample Calculations

PREPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

Description:This spreadsheet calculates the shear d/c ratio of a box girder at a single cross-section. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'shear capacity input' using vba for all stations. Definitions of notations and sample calculations at critical section are shown in mathcad sheets.

Station	V_{umax}(kip)	V_{umin}(kip)	T_u for V_{umax} (kip.ft)	T_u for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P_u for V_{umax} (kip)	P_u for V_{umin} (kip)	~~V_p (kip)~~	~~y_{tendon} (ft)~~
33.63	195	-31	684	273	4131	2585	-1034	-710	~~0~~	~~0~~

s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	~~P_e (kip)~~
16.0	15.0	0.6	0.3	4.0	0.4	0

Location	West Span

1 Material Properties

f'_c (ksi)	5
f_{pu} (ksi)	0
f_{p0} (ksi)	0.000
f_{ps} (ksi)	0.000
f_y (ksi)	60
E_s (ksi)	29000
E_p (ksi)	0
E_c (ksi)	4903

2 Structural Properties

Girder

b_t (ft)	36.073
b_b (ft)	23.873
h_{ft} (ft)	0.708
h_{fb} (ft)	0.500
b_w (ft)	3.363
h (ft)	4.25
A_0 (ft^2)	110.6
h_{fillet} (ft)	0.5
h_w (ft)	2.542
h_v (ft)	2.542
h_{tw}(ft)	0.333
A_{ct}_negtive (sf)	30.316
A_{ct}_positive (sf)	17.402

Tendon

A_{ps} (ft^2)	0.00000
N_s	0
N_t	0
$A_{ps.total}$(ft^2)	0.0000
d_{pmax} (ft)	0.000
d_{pmin} (ft)	0.000

Mild Reinforcement

C_t (ft)	0.208
C_b (ft)	0.083
$D_{N.5}$ (ft)	0.052
$A_{N.5}$ (sf)	0.00215
$N_{N.5}$	8
A_s for $V_{u\ max}$(sf)	0.395
A_s for $V_{u\ min}$ (sf)	0.395
d_s for V_{umax} (ft)	4.000
d_s for V_{umin} (ft)	4.000
A_v (sf)	0.0167
α (deg)	90.0

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Project I-405 Bridge 17.7 **Project No** A207833

Title Existing Structure Shear Capacity Check Sample Calculations

PREPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

Description:This spreadsheet calculates the shear d/c ratio of a box girder at a single cross-section. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'shear capacity input' using vba for all stations. Definitions of notations and sample calculations at critical section are shown in mathcad sheets.

Station	V_{umax}(kip)	V_{umin}(kip)	T_u for V_{umax} (kip.ft)	T_u for V_{umin} (kip.ft)	M_u for V_{umax} (kip.ft)	M_u for V_{umin} (kip.ft)	P_u for V_{umax} (kip)	P_u for V_{umin} (kip)	~~V_p (kip)~~	~~y_{tendon} (ft)~~
33.63	195	-31	684	273	4131	2585	-1034	-710	~~0~~	~~0~~

s_{ext} (in)	s_{int} (in)	d_{s_top} (ft)	A_{s_top} (sf)	d_{s_bottom} (ft)	A_{s_bottom} (sf)	~~P_e (kip)~~
16.0	15.0	0.6	0.3	4.0	0.4	0

3 Effective Shear Depth

if the station is in east span:	
d_e for $V_{u\ max}$ (ft)	4.000
d_e for $V_{u\ min}$ (ft)	4.000
d_v for V_{umax} (ft)	3.600
d_v for V_{umin} (ft)	3.600

4 Minimum Transverse Reinforcement Check

λ	1
A_{vmin} (in^2)	0.713
A_v (in^2)	2.40
Min. Trans. Rein.?	yes

5 Maximum Spacing of Transverse Reinforcement

	for V_{max}	for V_{min}
Resistance factor	0.9	0.9
v_u (ksi)	0.124	0.020
s_{max} (in)	24.0	24.0
check s_{max}	yes	yes

Does this check pass for both Vumax and Vumin? yes

5 Nominal Shear Resistance

	for V_{max}	for V_{min}
s_x (in)	11.00	11.00
S_{xe}(in)	11.00	11.00
V_{eff} (kip)	207	-36
ε_s _positive	0.00051	0.00024
ε_s _negative	3.64E-05	2.42E-04
ε_s	0.000508	0.00024
β	3.476	4.063
V_c (kip)	428.189	500.500
θ (deg)	30.778	29.847
V_s (kip)	697.024	723.515

a_g (in)	0.75
λ_{duct}	1

6 Longitudinal Reinforcement Check

$$A_{ps}f_{ps} + A_s f_y \geq \frac{|M_u|}{d_v\phi_f} + 0.5\frac{N_u}{\phi_c} + \left(\left|\frac{V_u}{\phi_v} - V_p\right| - 0.5V_s\right)\cot\theta$$

	for V_{max}	for V_{min}
LHS of eqn. (kip	3412.80	3412.80
RHS of eqn. (kip	882.67	433.66
long. Reinf. Chec	yes	yes

Does this check pass for both Vumax and Vumin? yes

6 Summary

	for V_{max}	for V_{min}
V_n (kip)	1125.2	1224.0
V_u (kip)	195.106	31.281
D/C	0.193	0.028

Governed D/C 0.193

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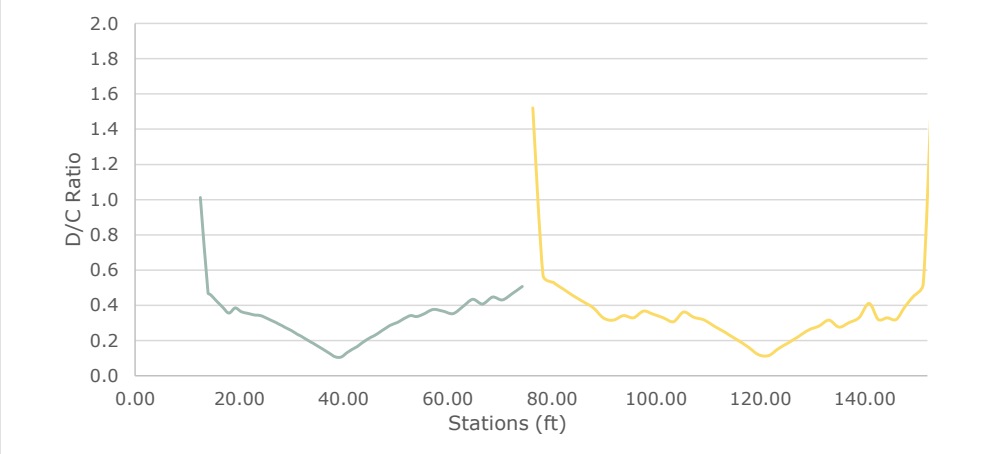
Project I-405 Bridge 17.7
Title Existing Structure Shear Capacity D/C
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

Description:
This sheet summarized the outputs of the shear capacity check for Bridge 17.7 existing superstructure.



	Station (ft)	Min Reinf.?	Max Spacing?	Long. Reinf.?	Governed D/C
Pier 3	152.93	yes	no	no	1.7
	151.20	yes	yes	yes	0.5
	149.48	yes	yes	yes	0.5
	147.75	yes	yes	yes	0.4
	146.02	yes	yes	yes	0.3
	144.29	yes	yes	yes	0.3
	142.57	yes	yes	yes	0.3
	140.84	yes	yes	yes	0.4
	138.91	yes	yes	yes	0.3
	136.98	yes	yes	yes	0.3
	135.05	yes	yes	yes	0.3
	133.11	yes	yes	yes	0.3
	131.18	yes	yes	yes	0.3
	129.25	yes	yes	yes	0.3
	127.32	yes	yes	yes	0.2
	125.39	yes	yes	yes	0.2
	123.46	yes	yes	yes	0.2
	121.53	yes	yes	yes	0.1
	119.59	yes	yes	yes	0.1
	117.66	yes	yes	yes	0.2
East Spa	115.73	yes	yes	yes	0.2
	113.80	yes	yes	yes	0.2
	112.93	yes	yes	yes	0.2
	111.01	yes	yes	yes	0.3
	109.08	yes	yes	yes	0.3
	107.16	yes	yes	yes	0.3
	105.23	yes	yes	yes	0.4
	103.31	yes	yes	yes	0.3
	101.39	yes	yes	yes	0.3
	99.46	yes	yes	yes	0.3
	97.54	yes	yes	yes	0.4
	95.62	yes	yes	yes	0.3
	93.69	yes	yes	yes	0.3
	91.77	yes	yes	yes	0.3
	89.84	yes	yes	yes	0.3
	87.92	yes	yes	yes	0.4
	86.00	yes	yes	yes	0.4
	84.07	yes	yes	yes	0.5
	82.15	yes	yes	yes	0.5
	80.22	yes	yes	yes	0.5
	78.30	yes	yes	yes	0.6
Pier 2	76.30	yes	no	yes	1.5

Station (ft)	Min Reinf.?	Max Spacing	Long. Reinf.	Governed D/C	
74.30	yes	yes	yes	0.5	
72.39	yes	yes	yes	0.5	
70.49	yes	yes	yes	0.4	
68.58	yes	yes	yes	0.4	
66.68	yes	yes	yes	0.4	
64.77	yes	yes	yes	0.4	
62.86	yes	yes	yes	0.4	
60.96	yes	yes	yes	0.4	
59.05	yes	yes	yes	0.4	
57.15	yes	yes	yes	0.4	
55.24	yes	yes	yes	0.3	
54.04	yes	yes	yes	0.3	
52.84	yes	yes	yes	0.3	
51.64	yes	yes	yes	0.3	
50.44	yes	yes	yes	0.3	
48.87	yes	yes	yes	0.3	
47.30	yes	yes	yes	0.3	
45.85	yes	yes	yes	0.2	
44.40	yes	yes	yes	0.2	
42.62	yes	yes	yes	0.2	
40.83	yes	yes	yes	0.1	
39.63	yes	yes	yes	0.1	West Span
38.43	yes	yes	yes	0.1	
37.23	yes	yes	yes	0.1	
36.03	yes	yes	yes	0.1	
34.83	yes	yes	yes	0.2	
33.63	yes	yes	yes	0.2	
32.43	yes	yes	yes	0.2	
31.22	yes	yes	yes	0.2	
30.02	yes	yes	yes	0.3	
28.82	yes	yes	yes	0.3	
27.62	yes	yes	yes	0.3	
26.42	yes	yes	yes	0.3	
25.22	yes	yes	yes	0.3	
24.02	yes	yes	yes	0.3	
22.82	yes	yes	yes	0.3	
21.62	yes	yes	yes	0.4	
20.42	yes	yes	yes	0.4	
19.22	yes	yes	yes	0.4	
18.02	yes	yes	yes	0.4	
16.82	yes	yes	yes	0.4	
15.61	yes	yes	yes	0.4	
14.41	yes	yes	yes	0.5	
14.00	yes	yes	yes	0.5	
12.51	yes	no	yes	1.0	Exist Pier 1

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Project I-405 Bridge 17.7
Title Existing Structure Flexural Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

Description:
This sheet summarizes the inputs of the flexural capacity check for Bridge 17.7 **existing** superstructure.

Notation Definitions:

I_{22} (ft^4)	girder cross-section about horizontal axis	As.1, As.2, and As.3	Longitudinal rein. area in top slab for layers 1, 2, and 3
y (ft)	Vertical distance between top of box girder to centroid	As.1, As.2, and As.3	Longitudinal rein. area in bot slab for layers 8 and 9
A (ft^2)	Box girder cross-section area	ds.1, ds.2, ds.3, ds.8, and ds.9	Vertical distance from top of deck to centroid of reinf.
y_{tendon} (ft)	the centroid of tendon to centroid of girder cross-section	M_{u_max} (kip.ft)	Max. moment demand
P_e (kip)	Tendon Force at Service	M_{u_min} (kip.ft)	Min. moment demand

	Station (ft)	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft	As.1 (in^2)	As.2 (in^2)	As.3 (in^2)	As.8 (in^2)	As.9 (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
Pier 3	152.93	127.82	1.65	52.74	0	0	-1693	-3324	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	151.20	127.82	1.65	52.74	0	0	-680	-2349	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	149.48	127.82	1.65	52.74	0	0	472	-1743	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	147.75	127.82	1.65	52.74	0	0	1547	-1204	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	146.02	127.82	1.65	52.74	0	0	2540	-712	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	144.29	127.82	1.65	52.74	0	0	3460	-262	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	142.57	127.82	1.65	52.74	0	0	4311	145	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	140.84	127.82	1.65	52.74	0	0	5196	524	11.47	11.47	3.10	22.400	22.400	3.44	7.188	11.50	46.50	49.000
	138.91	127.82	1.65	52.74	0	0	5949	883	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	136.98	127.82	1.65	52.74	0	0	6629	1196	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	135.05	127.82	1.65	52.74	0	0	7215	1464	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	133.11	127.82	1.65	52.74	0	0	7719	1688	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	131.18	127.82	1.65	52.74	0	0	8130	1867	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	129.25	127.82	1.65	52.74	0	0	8463	2003	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	127.32	127.82	1.65	52.74	0	0	8712	2095	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	125.39	127.82	1.65	52.74	0	0	8888	2144	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	123.46	127.82	1.65	52.74	0	0	8983	2152	11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740
	121.53	127.82	1.65	52.74	0	0	8994	2117	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	119.59	127.82	1.65	52.74	0	0	8913	2041	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
East Span	117.66	127.82	1.65	52.74	0	0	8764	1923	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	115.73	127.82	1.65	52.74	0	0	8570	1766	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	113.80	127.82	1.65	52.74	0	0	8387	1601	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	112.93	127.82	1.65	52.74	0	0	8243	1503	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	111.01	127.82	1.65	52.74	0	0	7886	1277	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	109.08	127.82	1.65	52.74	0	0	7424	989	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	107.16	127.82	1.65	52.74	0	0	6887	666	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	105.23	127.82	1.65	52.74	0	0	6274	305	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	103.31	127.82	1.65	52.74	0	0	5601	-91	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	101.39	127.82	1.65	52.74	0	0	4862	-525	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	99.46	127.82	1.65	52.74	0	0	4075	-1005	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	97.54	127.82	1.65	52.74	0	0	3236	-1544	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	95.62	127.82	1.65	52.74	0	0	2343	-2136	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	93.69	127.82	1.65	52.74	0	0	1383	-2765	22.20	22.20	6.00	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	91.77	127.82	1.65	52.74	0	0	356	-3430	40.80	33.60	7.20	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	89.84	127.82	1.65	52.74	0	0	-735	-4133	40.80	33.60	7.20	50.800	50.800	3.56	7.063	11.50	46.76	48.740
	87.92	127.82	1.65	52.74	0	0	-1874	-4885	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
	86.00	127.82	1.65	52.74	0	0	-2912	-5779	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875

COWI

Project I-405 Bridge 17.7
Title Existing Structure Flexural Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

Description:
This sheet summarizes the inputs of the flexural capacity check for Bridge 17.7 **existing** superstructure.

Notation Definitions:

I_{22} (ft^4)	girder cross-section about horizontal axis	As.1, As.2, and As.3	Longitudinal rein. area in top slab for layers 1, 2, and 3
y (ft)	Vertical distance between top of box girder to centroid	As.1, As.2, and As.3	Longitudinal rein. area in bot slab for layers 8 and 9
A (ft^2)	Box girder cross-section area	ds.1, ds.2, ds.3, ds.8, and ds.9	Vertical distance from top of deck to centroid of reinf.
y_{tendon} (ft)	the centroid of tendon to centroid of girder cross-section	M_{u_max} (kip.ft)	Max. moment demand
P_e (kip)	Tendon Force at Service	M_{u_min} (kip.ft)	Min. moment demand

	Station (ft)	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)	As.1 (in^2)	As.2 (in^2)	As.3 (in^2)	As.8 (in^2)	As.9 (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
	84.07	127.82	1.65	52.74	0	0	-3810	-6907	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
	82.15	127.82	1.65	52.74	0	0	-4736	-8100	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
	80.22	127.82	1.65	52.74	0	0	-5690	-9358	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
	78.30	127.82	1.65	52.74	0	0	-6649	-10678	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
Pier 2	76.30	127.82	1.65	52.74	0	0	-7381	-11916	40.80	33.60	7.20	28.100	28.100	3.56	7.063	11.50	46.63	48.875
	74.30	123.83	1.61	51.84	0	0	-6432	-10411	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	72.39	123.83	1.61	51.84	0	0	-5531	-9136	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	70.49	123.83	1.61	51.84	0	0	-4622	-7923	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	68.58	123.83	1.61	51.84	0	0	-3736	-6774	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	66.68	123.83	1.61	51.84	0	0	-2879	-5688	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	64.77	123.83	1.61	51.84	0	0	-2000	-4718	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	62.86	123.83	1.61	51.84	0	0	-962	-3984	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	60.96	123.83	1.61	51.84	0	0	61	-3299	40.80	33.60	7.20	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	59.05	123.83	1.61	51.84	0	0	1030	-2668	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	57.15	123.83	1.61	51.84	0	0	1941	-2082	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
	55.24	123.83	1.61	51.84	0	0	2774	-1536	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	54.04	123.83	1.61	51.84	0	0	3289	-1209	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	52.84	123.83	1.61	51.84	0	0	3770	-917	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	51.64	123.83	1.61	51.84	0	0	4256	-642	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	50.44	123.83	1.61	51.84	0	0	4690	-388	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	48.87	123.83	1.61	51.84	0	0	5237	-76	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	47.30	123.83	1.61	51.84	0	0	5717	212	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	45.85	123.83	1.61	51.84	0	0	6136	456	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	44.40	123.83	1.61	51.84	0	0	6510	677	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	42.62	123.83	1.61	51.84	0	0	6907	918	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	40.83	123.83	1.61	51.84	0	0	7237	1126	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
West Span	39.63	123.83	1.61	51.84	0	0	7419	1246	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	38.43	123.83	1.61	51.84	0	0	7570	1351	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	37.23	123.83	1.61	51.84	0	0	7693	1440	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	36.03	123.83	1.61	51.84	0	0	7803	1514	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	34.83	123.83	1.61	51.84	0	0	7880	1572	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	33.63	123.83	1.61	51.84	0	0	7926	1614	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	32.43	123.83	1.61	51.84	0	0	7941	1641	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	31.22	123.83	1.61	51.84	0	0	7924	1652	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	30.02	123.83	1.61	51.84	0	0	7875	1648	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	28.82	123.83	1.61	51.84	0	0	7794	1628	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
	27.62	123.83	1.61	51.84	0	0	7681	1593	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875

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Project I-405 Bridge 17.7
Title Existing Structure Flexural Inputs
PREPARED BY LDLI
CHECKED BY CUPG

Project No A207833
Date 11/25/2021
Date 11/25/2021

Description:
This sheet summarizes the inputs of the flexural capacity check for Bridge 17.7 **existing** superstructure.

Notation Definitions:

I_{22} (ft^4)	girder cross-section about horizontal axis	As.1, As.2, and As.3	Longitudinal rein. area in top slab for layers 1, 2, and 3
y (ft)	Vertical distance between top of box girder to centroid	As.1, As.2, and As.3	Longitudinal rein. area in bot slab for layers 8 and 9
A (ft^2)	Box girder cross-section area	ds.1, ds.2, ds.3, ds.8, and ds.9	Vertical distance from top of deck to centroid of reinf.
y_{tendon} (ft)	the centroid of tendon to centroid of girder cross-section	M_{u_max} (kip.ft)	Max. moment demand
P_e (kip)	Tendon Force at Service	M_{u_min} (kip.ft)	Min. moment demand

Station (ft)	I_{22} (ft^4)	y (ft)	A (ft^2)	y_{tendon} (ft)	P_e (kip)	M_{u_max} (kip.ft)	M_{u_min} (kip.ft	As.1 (in^2)	As.2 (in^2)	As.3 (in^2)	As.8 (in^2)	As.9 (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
26.42	123.83	1.61	51.84	0	0	7536	1542	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
25.22	123.83	1.61	51.84	0	0	7358	1475	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
24.02	123.83	1.61	51.84	0	0	7196	1409	24.00	18.00	6.00	28.440	28.440	3.56	7.063	11.50	47.13	48.875
22.82	123.83	1.61	51.84	0	0	7015	1333	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
21.62	123.83	1.61	51.84	0	0	6813	1246	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
20.42	123.83	1.61	51.84	0	0	6589	1149	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
19.22	123.83	1.61	51.84	0	0	6346	1042	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
18.02	123.83	1.61	51.84	0	0	6084	924	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
16.82	123.83	1.61	51.84	0	0	5795	797	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
15.61	123.83	1.61	51.84	0	0	5489	659	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
14.41	123.83	1.61	51.84	0	0	5163	510	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
14.00	123.83	1.61	51.84	0	0	5024	454	24.00	18.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875
12.51	123.83	1.61	51.84	0	0	4595	235	24.00	41.00	6.00	23.700	23.700	3.56	7.063	11.50	47.13	48.875

Exist Pier

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Project	I-405 Bridge 17.7	**Project No**	A207833
Title	Existing Structure Flexural Capacity Sample Calculation		
PREPARED BY	LDLI	**Date**	11/25/2021
CHECKED BY	CUPG	**Date**	11/25/2021

Description:This spreadsheet calculates the flexural d/c ratio of a box girder at a single cross-section. The properties, sizes, and demands at this cross-section are summarized in rows 11-15. These inputs are are automatically transferred from sheet 'flexural capacity input' using vba for all stations. Definitions of notations and sample calculations at critical section are shown in mathcad sheets.

Station	I_{22} (ft^4)	y (ft)	A (ft^2)	~~rtendon (ft~~	~~Pc (kip)~~	M_{u_max} (kip.ft)	M_{u_min} (kip.ft)
123.46	127.82	1.65	52.74	0	0	8983	2152

$As.1$ (in^2)	$As.2$ (in^2)	$As.3$ (in^2)	$As.8$ (in^2)	$As.9$ (in^2)	ds.1 (in)	ds.2(in)	ds.3 (in)	ds.8(in)	ds.9(in)
11.47	11.47	3.10	50.800	50.800	3.44	7.188	11.50	46.76	48.740

Location East Span

1 Material Properties

f'_c (ksi)	5
f_r (ksi)	0.537
f_y (ksi)	60

2 Structural Properties

Girder

b_t (ft)	36.067
b_b (ft)	24
h_{ft} (ft)	0.708
h_{fb} (ft)	0.500
b_w (ft)	3.363
h (ft)	4.25
h_{fillet} (ft)	0.5
h_w (ft)	2.542

Mild Reinforcement

E_s (ksf)	4177087.2

3 Flexural Demand

	for M_{u_max}	for M_{u_min}
1.33M_u(kip.ft)	11947.49	2861.54
S_c (ft)	49.105	49.105
M_{cr} (kip.ft)	4067.97	4067.97

γ_1	1.6
γ_2	1
γ_3	0.67

4 Factored Flexural Resistance

	for M_{u_max}	for M_{u_min}
c (in)	4.14	4.14
a (in)	3.314	3.314
Φ	0.9	0.9
M_n (kip.ft)	23415.22	23415.22

β_1	0.8

Note:

1. compression reinforcements and all the reinforcements in the webs are ignored in the calculation of c
2. the tensile reinforcements in the slabs are assumed to yield in the calculation of c

5 Summary

for M_{u_max}

	Demand	Resistance	D/C
Minimum Reinforcement	4067.974	21073.698	0.193
Ultimate Capcity	8983.074	21073.698	0.426

for M_{u_min}

	Demand	Resistance	D/C
Minimum Reinforcement	2861.537	21073.698	0.136
Ultimate Capcity	2151.531	21073.698	0.102

For maximum demand

This cross-section is governed by:

ultimate capacity check

D/C ratio for this cross section is:

0.426

For minimum demand

This cross-section is governed by:

minimum reinforcement check

D/C ratio for this cross section is:

0.136

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Project I-405 Bridge 17.7 **Project No** A207833

Title Existing Structure Flexural Capacities

PREPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

Description:
This sheet summarizes the outputs of the flexural capacity check for Bridge 17.7 existing superstructure.

	Station (ft)	Demand for Max. M	Demand for Min. M	Capacity for Max. M	Capacity for Min. M
Pier 3	152.93	-2251.85	-4420.64	-5212.12	-5212.1
	151.20	-904.95	-3123.83	-5212.12	-5212.1
	149.48	627.68	-2318.35	9508.34	-5212.1
	147.75	2056.90	-1601.63	9508.34	-5212.1
	146.02	3378.61	-946.74	9508.34	-5212.1
	144.29	4067.97	-349.07	9508.34	-5212.1
	142.57	4311.34	193.00	9508.34	9508.3
	140.84	5196.02	696.73	9508.34	9508.3
	138.91	5949.25	1174.30	21073.70	21073.7
	136.98	6629.43	1591.07	21073.70	21073.7
	135.05	7214.78	1947.74	21073.70	21073.7
	133.11	7719.18	2244.98	21073.70	21073.7
	131.18	8130.14	2483.42	21073.70	21073.7
	129.25	8463.34	2663.66	21073.70	21073.7
	127.32	8711.86	2786.35	21073.70	21073.7
	125.39	8887.62	2852.10	21073.70	21073.7
	123.46	8983.07	2861.54	21073.70	21073.7
	121.53	8993.55	2815.27	21073.70	21073.7
	119.59	8913.40	2713.91	21073.70	21073.7
	117.66	8764.31	2558.09	21073.70	21073.7
East Span	115.73	8569.91	2348.41	21073.70	21073.7
	113.80	8387.29	2128.98	21073.70	21073.7
	112.93	8243.05	1999.14	21073.70	21073.7
	111.01	7886.29	1698.27	21073.70	21073.7
	109.08	7423.50	1315.86	21073.70	21073.7
	107.16	6886.84	885.16	21073.70	21073.7
	105.23	6274.31	406.18	21073.70	21073.7
	103.31	5601.09	-121.10	21073.70	-9914.7
	101.39	4861.83	-697.80	21073.70	-9914.7
	99.46	4074.52	-1336.34	21073.70	-9914.7
	97.54	4067.97	-2053.35	21073.70	-9914.7
	95.62	3115.87	-2841.06	21073.70	-9914.7
	93.69	1839.28	-3677.09	21073.70	-9914.7
	91.77	473.20	-4561.56	21073.70	-15898.3
	89.84	-977.81	-5496.29	-15898.28	-15898.3
	87.92	-2492.78	-6429.23	-15898.28	-15898.3
	86.00	-3873.20	-6429.23	-15898.28	-15898.3
	84.07	-5067.37	-6907.07	-15898.28	-15898.3
	82.15	-6298.74	-8100.28	-15898.28	-15898.3
	80.22	-6429.23	-9357.55	-15898.28	-15898.3
	78.30	-6649.10	-10677.68	-15898.28	-15898.3
Pier 2	76.30	-7380.74	-11915.65	-15898.28	-15898.3

	Station (ft)	Demand for Max. M	Demand for Min. M	Capacity for Max. M	Capacity for Min. M
	74.30	-6432.16	-10411.35	-15898.3	-15898.3
	72.39	-6371.67	-9136.16	-15898.3	-15898.3
	70.49	-6147.07	-7923.28	-15898.3	-15898.3
	68.58	-4968.97	-6773.60	-15898.3	-15898.3
	66.68	-3828.95	-6371.67	-15898.3	-15898.3
	64.77	-2660.44	-6274.86	-15898.3	-15898.3
	62.86	-1280.05	-5298.07	-15898.3	-15898.3
	60.96	81.14	-4387.83	10073.5	-15898.3
	59.05	1369.50	-3548.20	10073.5	-9494.6
	57.15	2581.03	-2768.66	10073.5	-9494.6
	55.24	3689.09	-2042.34	12048.6	-9494.6
	54.04	3885.76	-1607.86	12048.6	-9494.6
	52.84	3885.76	-1220.21	12048.6	-9494.6
	51.64	4256.39	-854.31	12048.6	-9494.6
	50.44	4690.37	-515.39	12048.6	-9494.6
	48.87	5236.98	-101.18	12048.6	-9494.6
	47.30	5717.06	281.52	12048.6	12048.6
	45.85	6135.63	606.02	12048.6	12048.6
	44.40	6509.92	900.35	12048.6	12048.6
	42.62	6906.84	1221.57	12048.6	12048.6
	40.83	7237.15	1497.56	12048.6	12048.6
	39.63	7419.46	1657.40	12048.6	12048.6
West Span	38.43	7570.07	1796.59	12048.6	12048.6
	37.23	7693.35	1915.12	12048.6	12048.6
	36.03	7802.51	2012.97	12048.6	12048.6
	34.83	7880.10	2090.14	12048.6	12048.6
	33.63	7926.43	2146.62	12048.6	12048.6
	32.43	7941.00	2182.40	12048.6	12048.6
	31.22	7923.85	2197.48	12048.6	12048.6
	30.02	7874.88	2191.85	12048.6	12048.6
	28.82	7794.00	2165.51	12048.6	12048.6
	27.62	7680.77	2118.47	12048.6	12048.6
	26.42	7535.52	2050.71	12048.6	12048.6
	25.22	7358.31	1962.22	12048.6	12048.6
	24.02	7195.55	1874.07	12048.6	12048.6
	22.82	7015.30	1772.31	10073.5	10073.5
	21.62	6813.36	1656.87	10073.5	10073.5
	20.42	6589.47	1527.70	10073.5	10073.5
	19.22	6346.21	1385.28	10073.5	10073.5
	18.02	6083.86	1229.35	10073.5	10073.5
	16.82	5795.08	1059.80	10073.5	10073.5
	15.61	5489.34	876.86	10073.5	10073.5
	14.41	5162.84	678.00	10073.5	10073.5
	14.00	5024.45	604.37	10073.5	10073.5
Exist Pier 1	12.51	4595.09	312.15	10073.5	10073.5

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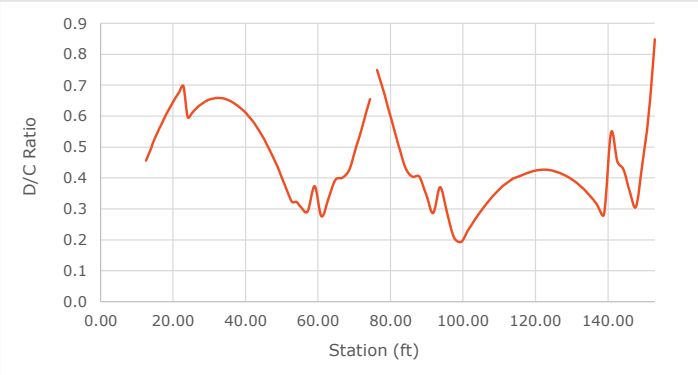
Project I-405 Bridge 17.7 **Project No** A207833

Title Existing Structure Flexural D/C

REPARED BY LDLI **Date** 11/25/2021

CHECKED BY CUPG **Date** 11/25/2021

Description:
This sheet summarizes the outputs of the flexural capacity check for Bridge 17.7 existing superstructure.



Note: the longitudinal reinforcements in the flare region for stations between 113.8 and 140.84ft are ignored in the capacity calculation, so the actual D/C ratios in cross-sections within the flare region are less than shown in

	Station (ft)	D/C Ratio for Max. M	D/C Ratio for Min. M	Governed D/C
Pier 3	152.93	0.43	0.85	0.8
	151.20	0.17	0.60	0.6
	149.48	0.07	0.44	0.4
	147.75	0.22	0.31	0.3
	146.02	0.36	0.18	0.4
	144.29	0.43	0.07	0.4
	142.57	0.45	0.02	0.5
	140.84	0.55	0.07	0.5
	138.91	0.28	0.06	0.3
	136.98	0.31	0.08	0.3
	135.05	0.34	0.09	0.3
	133.11	0.37	0.11	0.4
	131.18	0.39	0.12	0.4
	129.25	0.40	0.13	0.4
	127.32	0.41	0.13	0.4
	125.39	0.42	0.14	0.4
	123.46	0.43	0.14	0.4
	121.53	0.43	0.13	0.4
	119.59	0.42	0.13	0.4
	117.66	0.42	0.12	0.4
East Span	115.73	0.41	0.11	0.4
	113.80	0.40	0.10	0.4
	112.93	0.39	0.09	0.4
	111.01	0.37	0.08	0.4
	109.08	0.35	0.06	0.4
	107.16	0.33	0.04	0.3
	105.23	0.30	0.02	0.3
	103.31	0.27	0.01	0.3
	101.39	0.23	0.07	0.2
	99.46	0.19	0.13	0.2
	97.54	0.19	0.21	0.2
	95.62	0.15	0.29	0.3
	93.69	0.09	0.37	0.4
	91.77	0.02	0.29	0.3
	89.84	0.06	0.35	0.3
	87.92	0.16	0.40	0.4
	86.00	0.24	0.40	0.4
	84.07	0.32	0.43	0.4
	82.15	0.40	0.51	0.5
	80.22	0.40	0.59	0.6
	78.30	0.42	0.67	0.7
Pier 2	76.30	0.46	0.75	0.7

	Station (ft)	D/C Ratio for Max. M	D/C Ratio for	Governed D/C
	74.30	0.40	0.65	0.7
	72.39	0.40	0.57	0.6
	70.49	0.39	0.50	0.5
	68.58	0.31	0.43	0.4
	66.68	0.24	0.40	0.4
	64.77	0.17	0.39	0.4
	62.86	0.08	0.33	0.3
	60.96	0.01	0.28	0.3
	59.05	0.14	0.37	0.4
	57.15	0.26	0.29	0.3
	55.24	0.31	0.22	0.3
	54.04	0.32	0.17	0.3
	52.84	0.32	0.13	0.3
	51.64	0.35	0.09	0.4
	50.44	0.39	0.05	0.4
	48.87	0.43	0.01	0.4
	47.30	0.47	0.02	0.5
	45.85	0.51	0.05	0.5
	44.40	0.54	0.07	0.5
	42.62	0.57	0.10	0.6
	40.83	0.60	0.12	0.6
	39.63	0.62	0.14	0.6
West Span	38.43	0.63	0.15	0.6
	37.23	0.64	0.16	0.6
	36.03	0.65	0.17	0.6
	34.83	0.65	0.17	0.7
	33.63	0.66	0.18	0.7
	32.43	0.66	0.18	0.7
	31.22	0.66	0.18	0.7
	30.02	0.65	0.18	0.7
	28.82	0.65	0.18	0.6
	27.62	0.64	0.18	0.6
	26.42	0.63	0.17	0.6
	25.22	0.61	0.16	0.6
	24.02	0.60	0.16	0.6
	22.82	0.70	0.18	0.7
	21.62	0.68	0.16	0.7
	20.42	0.65	0.15	0.7
	19.22	0.63	0.14	0.6
	18.02	0.60	0.12	0.6
	16.82	0.58	0.11	0.6
	15.61	0.54	0.09	0.5
	14.41	0.51	0.07	0.5
	14.00	0.50	0.06	0.5
Exist Pier 1	12.51	0.46	0.03	0.5

COWI

SUBJECT: _SLS Check for East Span____
PROJECT:_ BR17.7__
DESIGNED BY:_LDLI DATE:_11/26/2021
_CHECKED BY:_CUPG DATE: 11/26/2021_

1. Purpose and Assumptions

1. The purpose is to calculate and check the deformation and the reinforcement distribution for the service limit states in east span
2. The calculation is based on AASHTO 8th Edition, 2017

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}}\right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi}\right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft = 432.876 \cdot in$
Bottom Width	$b_b := 24.021ft = 288.252 \cdot in$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6.5in$
Web Thickness	$b_w := (10in \cdot 2 + 9in \cdot 2) = 3.167 \cdot ft$
Total Depth	$h := 4.25ft = 51 \cdot in$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 127.82ft^4$
Vertical Distance from Top of Deck to Centroid of Cross-section	$y := 1.647ft$
Cross-section Area	$A := 52.739ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$
West Span Length	$L_{west} := 76.295ft$
East Span Length	$L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab	$c_{t.t} := 2.5in$
Top Cover for Bottom Slab	$c_{t.b} := 1in$
Bottom Cover for Top/Bottom Slab	$c_b := 1in$
Diameter of #10	$D_{N10} := 1.27in$
Diameter of #7	$D_{N7} := 0.875in$
Diameter of #5	$D_{N5} := 0.625in$
Diameter of #4	$D_{N4} := 0.5in$
Vertical Reinforcement Spacing in Web	$s_w := 8in$
Young's Modulus of Reinforcement	$E_{sv} := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	37	$A_{s.1} := 37 \cdot 0.6in^2 = 22.2 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	27	$A_{s.2} := 27 \cdot 0.6in^2 = 16.2 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#10	40	$A_{s.8} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#10	40	$A_{s.9} := 40 \cdot 1.27in^2 = 50.8 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4) $d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$

Bar in Bottom Slab (Layer 1) $d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N10}}{2}\right) = 46.635 \cdot in$

Bar in Bottom Slab (Layer 2) $d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N10}}{2}\right) = 48.865 \cdot in$

Maximum Spacing in the Layer Closest to the Tension Face $S_{max} := \frac{(3ft + 10.875in)}{4} = 11.719 \cdot in$

Minimum Spacing in the Layer Closest to the Tension Face $S_{min} := \frac{(3ft + 0.75in)}{9} = 4.083 \cdot in$

3. Reinforcement Distribution

Exposure Condition: Class 2

Exposure Factor (5.6.7) $\gamma_e := 0.75$

Aggregate Size $a_g := 0.75in$

Note: as per AASHTO C5.4.2.1

Positive Moment at SLS $M_{sls} := 8870kip \cdot ft$

Note: maximum moment at east span from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination

Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto $d_c := c_b + D_{N4} + \frac{D_{N10}}{2} = 2.135 \cdot in$

Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7) $\beta_s := 1 + \frac{d_c}{0.7 \cdot \left(h - d_c\right)} = 1.062$

Tensile strain in nonprestressed reinforcement at SLS

$\varepsilon_s := 0.88 \cdot 10^{-3}$

Note: From Response 2000 Tensile Stress in Nonprestressed Reinforcement at SLS

Tensile stress in nonprestressed reinforcement at SLS

$f_{ss} := \varepsilon_s \cdot E_s = 25.527 \cdot ksi$

Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7)

$S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 15.088 \cdot in$

Minimum Spacing for CIP Concrete (5.10.3.1.1)

$S_{min.limit} := \max\left(1.5 \cdot D_{N10}, 1.5in, 1.5 \cdot a_g\right) = 1.905 \cdot in$

Maximum Allowable Spacing (5.10.3.2)

$S_{max.limit} := S_{max.limit} = 15.088 \cdot in$

4. Deformations

Deflection due to Vehicle and Pedestrian Load

$\Delta := 0.0144ft = 0.173 \cdot in$

Note: deflection is the maximum absolute deflection from csibridge load combination "LL_Sup_Def"

Cracking Moment (5.6.3.5.2)

$M_{cr} := f_r \cdot \frac{I_{22}}{h - y} = 3.795 \times 10^3 \cdot kip \cdot ft$

Maximum Moment in a Component at the Stage for which Deformation is Computed

$M_a := 9434kip \cdot ft$

Note: M.sls is from csibridge. It's the maximum moment in mid span in load combintation 'SE1-1 SUP_DEF_ENV'

Curvature at Ma (from Response 2000)

$\rho := \frac{23.84}{10^6} \frac{rad}{in}$

Effective Moment of Inertia (5.6.3.5.2)

$I_{eff} := \frac{M_a}{E_c \cdot \rho} = 9.684 \times 10^5 \cdot in^4$

Adjustment Factor for Deflection

$\alpha := \frac{I_{eff}}{I_{22}} = 0.365$

Adjusted Deflection due to Vehicle and Pedestrian Load

$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.473 \cdot in$

Maximum Allowable Deflection (2.5.2.6.2)

$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$

5. Summary

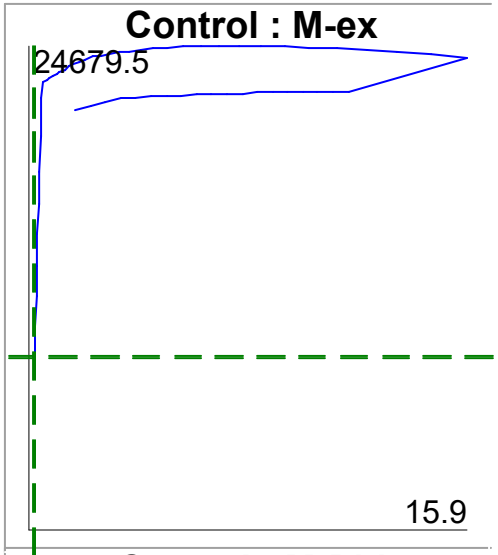
Maximum Spacing in Design	$S_{max} = 11.719 \cdot in$	
Maximum Allowable Spacing	$S_{max.limit} = 15.088 \cdot in$	
Maximum Spacing Check	$if(S_{max} \le S_{max.limit}, "ok", "fail") = "ok"$	$\frac{(S_{max})}{(S_{max.limit})} = 0.777$
Minimum Spacing in Design	$S_{min} = 4.083 \cdot in$	
Minimum Allowable Spacing	$S_{min.limit} = 1.905 \cdot in$	
Minimum Spacing Check	$if(S_{min} \ge S_{min.limit}, "ok", "fail") = "ok"$	$\frac{((S_{min.limit}))}{((S_{min}))} = 0.467$
Adjusted Deflection due to Vehicle and Pedestrian Load	$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.473 \cdot in$	
Maximum Allowable Deflection (2.5.2.6.2)	$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$	$\frac{\Delta_{adjusted}}{\Delta_{max.limit}} = 0.514$
Deflection Check	$if(\Delta_{adjusted} \le \Delta_{max.limit}, "ok", "fail") = "ok"$	

Response-2000 v 1.0.5

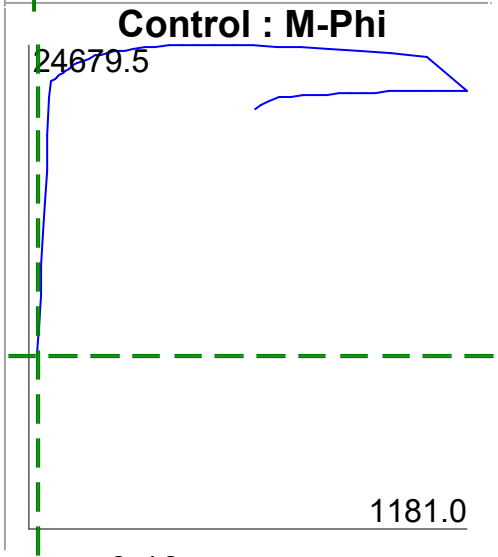
Enter Title Here

2021/11/25 - 2:34 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.19 ms

ϕ = 22.42 rad/10^6 in

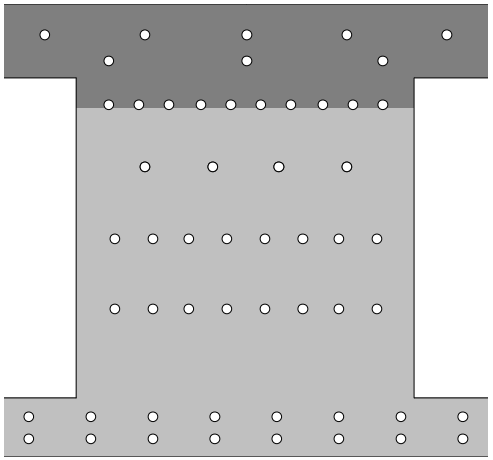
γ_{xy}(avg) = 0.00 ms

Axial Load = -1.0 kips

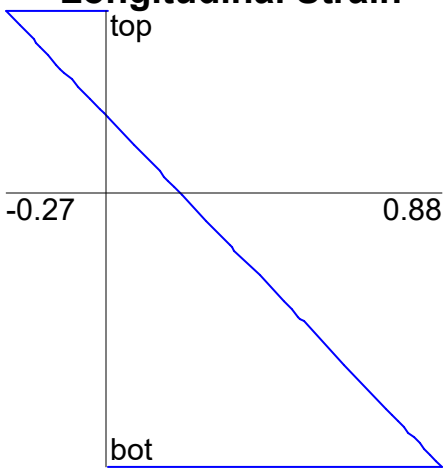
Moment:= 8863.8 ft-kips

Shear = 0.0 kips

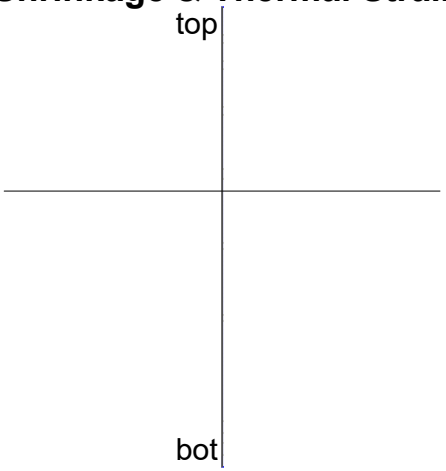
Cross Section



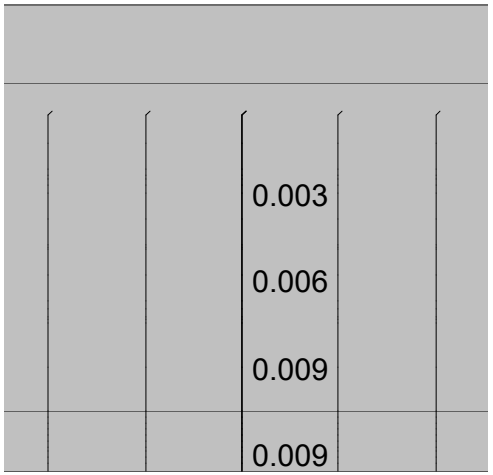
Longitudinal Strain



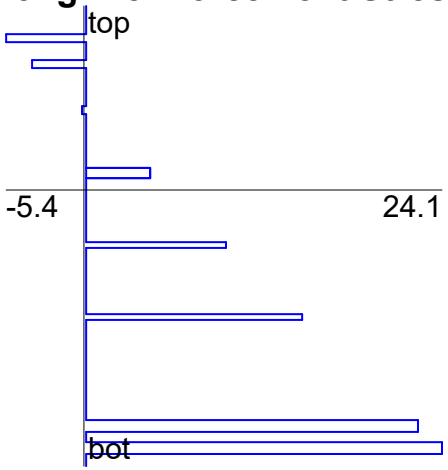
Shrinkage & Thermal Strain



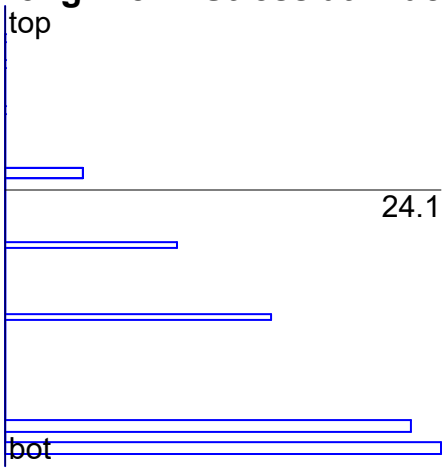
Crack Diagram



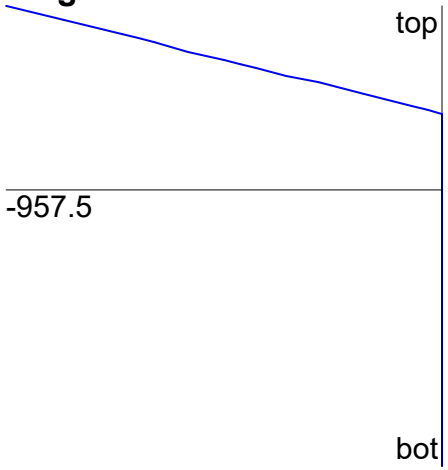
Long. Reinforcement Stress



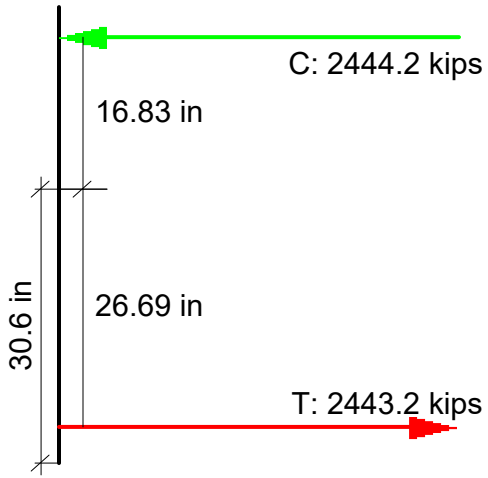
Long. Reinf Stress at Crack



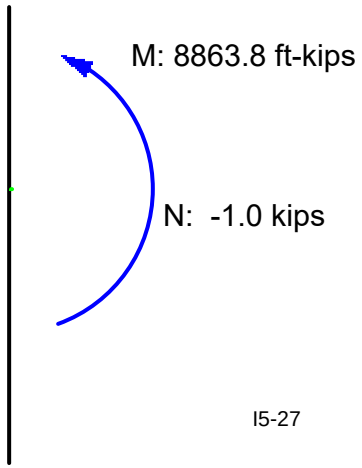
Longitudinal Concrete Stress

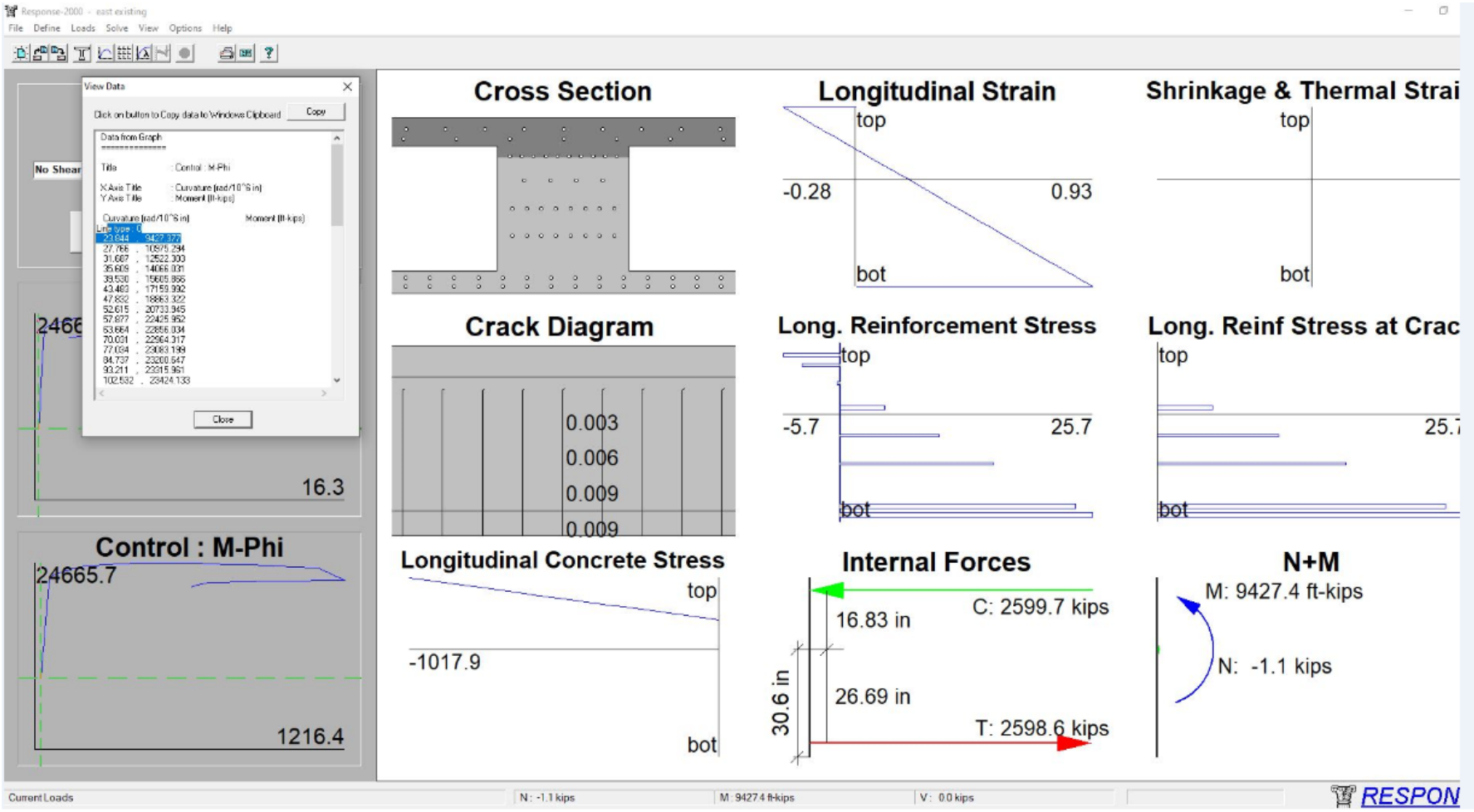


Internal Forces



N+M




Response-2000 - east existing
File Define Loads Solve View Options Help
View Data
Click on button to Copy data to Windows Clipboard
Copy
Data from Graph
Title : Control : M-Phi
X Axis Title : Curvature (rad/10^5 in)
Y Axis Title : Moment (ft-kips)
Curvature (rad/10^5 in) Moment (ft-kips)
Line type 0
23.644 , 9427.377
27.766 , 10975.294
31.687 , 12522.303
35.609 , 14066.031
39.530 , 15605.866
43.483 , 17159.992
47.892 , 18863.322
52.615 , 20733.945
57.877 , 22425.952
63.664 , 22856.034
70.031 , 22964.317
77.034 , 23083.199
84.737 , 23200.547
93.211 , 23315.961
102.532 , 23424.133
Close
No Shear
2466
16.3
Control : M-Phi
24665.7
1216.4
Cross Section
Longitudinal Strain
top
-0.28
0.93
bot
Shrinkage & Thermal Strai
top
bot
Crack Diagram
0.003
0.006
0.009
0.009
Long. Reinforcement Stress
top
-5.7
25.7
bot
Long. Reinf Stress at Crac
top
25.7
bot
Longitudinal Concrete Stress
top
-1017.9
bot
Internal Forces
C: 2599.7 kips
16.83 in
30.6 in
26.69 in
T: 2598.6 kips
N+M
M: 9427.4 ft-kips
N: -1.1 kips
Current Loads
N : -1.1 kips
M: 9427.4 ft-kips
V : 0.0 kips
RESPON

1. Purpose and Assumptions

1. The purpose is to calculate and check the reinforcement distribution for the service limit states
2. The calculation is based on AASHTO 8th Edition, 2017.

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}} \right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi} \right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft = 432.876 \cdot in$
Bottom Width	$b_b := 24.021ft = 288.252 \cdot in$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6.5in$
Total Depth	$h := 4.25ft = 51 \cdot in$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 127.82ft^4$
Vertical Distance from Top of Deck to Centroid	$y := 1.647ft$
Cross-section Area	$A := 52.739ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$
West Span Length	$L_{west} := 76.295$
East Span Length	$L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #8 $D_{N8} := 1in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	68	$A_{s.1} := 68 \cdot 0.6in^2 = 40.8 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	56	$A_{s.2} := 56 \cdot 0.6in^2 = 33.6 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	12	$A_{s.3} := 12 \cdot 0.6in^2 = 7.2 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	30	$A_{s.8.1} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#6	10	$A_{s.8.2} := 10 \cdot 0.44in^2 = 4.4 \cdot in^2$
Total Bar in Bottom Slab (Layer 1)			$A_{s.8} := A_{s.8.1} + A_{s.8.2} = 28.1 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	30	$A_{s.9.1} := 30 \cdot 0.79in^2 = 23.7 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#6	10	$A_{s.9.2} := 10 \cdot 0.44in^2 = 4.4 \cdot in^2$
Total Bar in Bottom Slab (Layer 2)			$A_{s.9} := A_{s.9.1} + A_{s.9.2} = 28.1 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := \left(h - c_{t.t} - D_{N5}\right) - \frac{D_{N7}}{2} = 47.437 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left[h - \left(h_{ft} - c_b\right) + D_{N5}\right] + \frac{D_{N7}}{2} = 44.563 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \left(h - h_{ft}\right) - \frac{h_{fillet}}{2} = 39.5 \cdot in$

Bar in Web (Layer 1)	$d_{s.4} := 32.5in = 32.5 \cdot in$
Bar in Web (Layer 2)	$d_{s.5} := d_{s.4} - s_w = 24.5 \cdot in$
Bar in Web (Layer 3)	$d_{s.6} := d_{s.5} - s_w = 16.5 \cdot in$
Bar in Web (Layer 4)	$d_{s.7} := d_{s.6} - s_w = 8.5 \cdot in$
Bar in Bottom Slab (Layer 1)	$d_{s.8} := h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2} = 4.5 \cdot in$
Bar in Bottom Slab (Layer 2)	$d_{s.9} := c_b + D_{N4} + \frac{D_{N8}}{2} = 2 \cdot in$
Maximum Spacing in the Layer Closest to the Tension Face	$S_{max} := \frac{(3ft + 6.25in)}{6} = 7.042 \cdot in$
Minimum Spacing in the Layer Closest to the Tension Face	$S_{min} := \frac{(2ft + 10.875in)}{6} = 5.813 \cdot in$

3. Reinforcement Distribution

Exposure Condition	Class 2	
Exposure Factor (5.6.7)	$\gamma_e := 0.75$	
Aggregate Size	$a_g := 0.75in$	Note: as per AASHTO C5.4.2.1
Negative Moment at SLS	$M_{sls} := 5343kip \cdot ft$	Note: maximum moment at pier from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination
Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto	$d_c := c_{t.t} + D_{N4} + \frac{D_{N7}}{2} = 3.438 \cdot in$	
Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7)	$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1.103$	
Tensile strain in nonprestressed reinforcement at SLS	$\varepsilon_s := 0.92 \cdot 10^{-3}$	Note: From Response 2000 Tensile Stress in Nonprestressed Reinforcement at SLS
Tensile stress in nonprestressed reinforcement at SLS	$f_{ss} := \varepsilon_s \cdot E_s = 26.687 \cdot ksi$	
Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7)	$S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 10.956 \cdot in$	

Minimum Spacing for CIP Concrete (5.10.3.1.1)

$$S_{min.limit} := \max\left(1.5 \cdot D_{N7}, 1.5in, 1.5 \cdot a_g\right) = 1.5 \cdot in$$

4. Summary

Maximum Spacing in Design

$$S_{max} = 7.042 \cdot in$$

Maximum Allowable Spacing

$$S_{max.limit} = 10.956 \cdot in$$

$$\frac{\left(S_{max}\right)}{\left(S_{max.limit}\right)} = 0.643$$

Maximum Spacing Check

$$\text{if}\left(S_{max} \le S_{max.limit}, \text{"ok"}, \text{"fail"}\right) = \text{"ok"}$$

Minimum Spacing in Design

$$S_{min} = 5.813 \cdot in$$

Minimum Allowable Spacing

$$S_{min.limit} = 1.5 \cdot in$$

$$\frac{\left(S_{min.limit}\right)}{S_{min}} = 0.258$$

Minimum Spacing Check

$$\text{if}\left(S_{min} \ge S_{min.limit}, \text{"ok"}, \text{"fail"}\right) = \text{"ok"}$$

Response-2000 v 1.0.5

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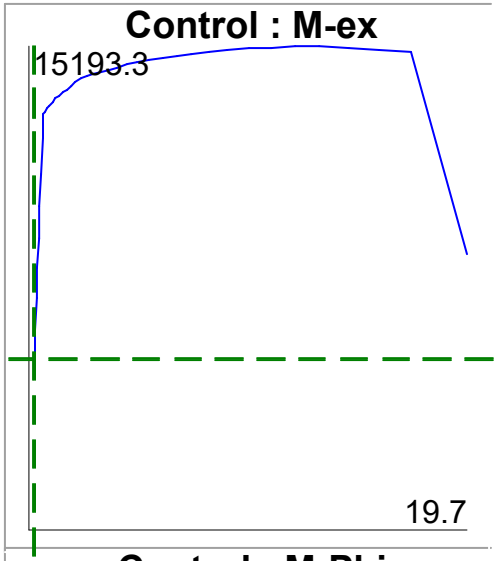
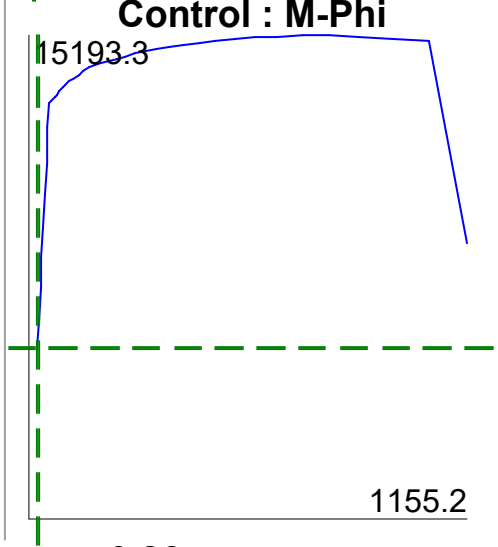
2021/11/25 - 2:34 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.26 ms

ϕ = 21.69 rad/10^6 in

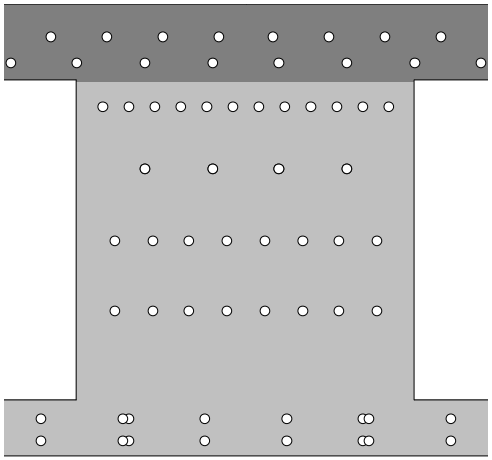
γ_{xy}(avg) = 0.00 ms

Axial Load = -0.8 kips

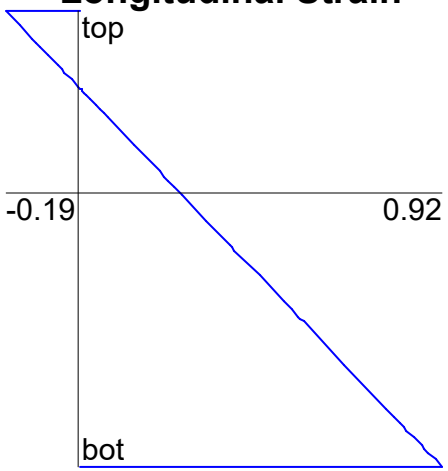
Moment:= 5339.1 ft-kips

Shear = 0.0 kips

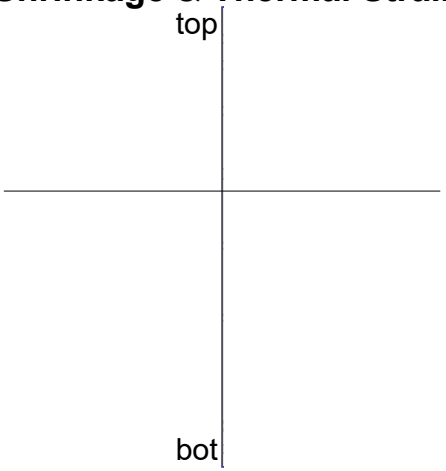
Cross Section



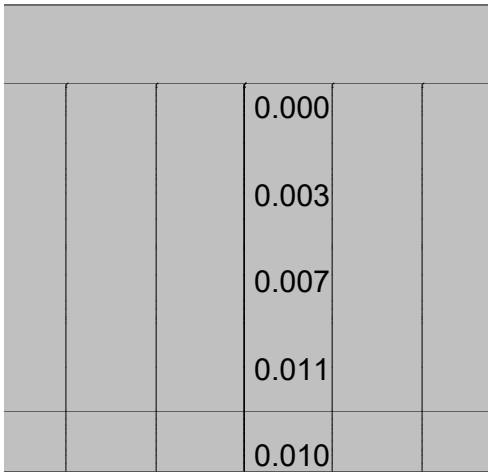
Longitudinal Strain



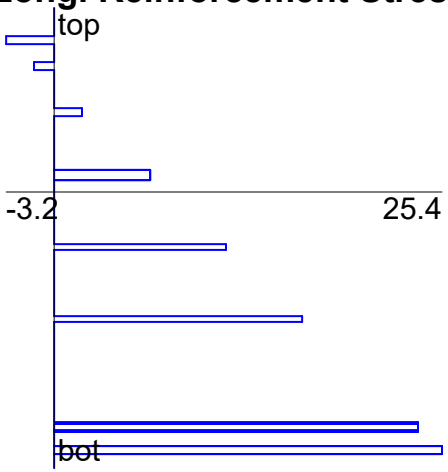
Shrinkage & Thermal Strain



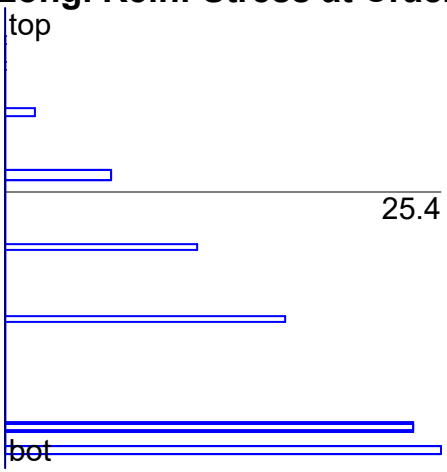
Crack Diagram



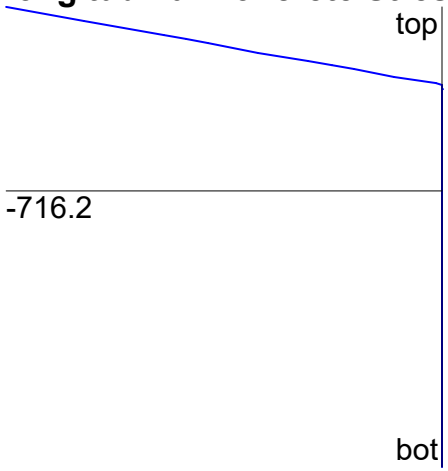
Long. Reinforcement Stress



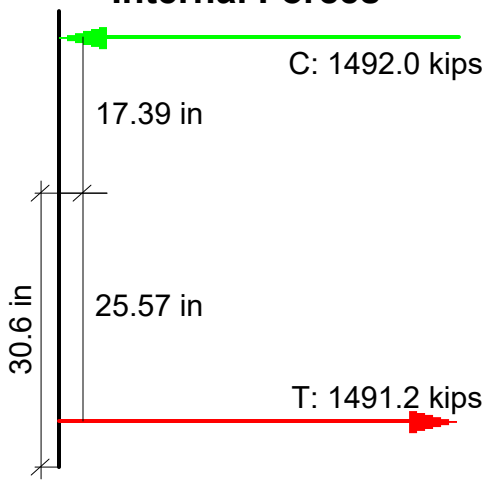
Long. Reinf Stress at Crack



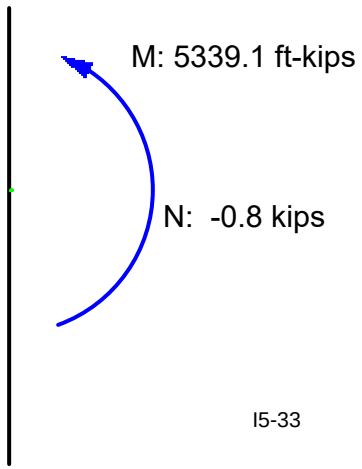
Longitudinal Concrete Stress



Internal Forces



N+M



1. Purpose and Assumptions

1. The purpose is to calculate and check the deformation and the reinforcement distribution for the service limit states in west span
2. The calculation is based on AASHTO 8th Edition, 2017

2. Material Properties

Concrete Compressive Strength	$f_c := 5ksi$
Concrete Modulus of Fracture (5.4.2.6)	$f_r := 0.24 \cdot \sqrt{\frac{f_c}{ksi}} \cdot ksi = 0.537 \cdot ksi$
Reinforcement Tensile Yield Strength	$f_y := 60ksi$
Reinforcement Compressive Yield Strength	$f_{y'} := 60ksi$
Young's Modulus of Mild Reinforcement or Tendon	$E_s := 200GPa = 2.901 \times 10^4 \cdot ksi$
Unit Weight of Concrete	$w_c := 0.155 \frac{kip}{ft^3}$
Correction Factor for Source of Aggregate (5.4.2.4)	$K_1 := 1$
Young's Modulus of Concrete (5.4.2.4)	$E_c := 120000 \cdot \left(\frac{w_c}{\frac{kip}{ft^3}} \right)^2 \cdot K_1 \cdot \left(\frac{f_c}{ksi} \right)^{0.33} ksi = 4.903 \times 10^3 \cdot ksi$

3. Structural Properties

Girder

Girder Type	Box girder
Number of Cells	3
Top Width	$b_t := 36.073ft$
Bottom Width	$b_b := 24ft$
Top Slab Thickness	$h_{ft} := 8.5in$
Bottom Slab Thickness	$h_{fb} := 6in$
Web Thickness	$b_w := (10in \cdot 2 + 9in \cdot 2) = 3.167 \cdot ft$
Total Depth	$h := 4.25ft$
Cross-section Moment of Inertia about Horizontal Axis	$I_{22} := 123.83ft^4$
Vertical Distance from Top of Deck to Centroid of Cross-section	$y := 1.61ft$
Cross-section Area	$A := 51.842ft^2$
Height of the Fillet	$h_{fillet} := 0.5ft$
West Span Length	$L_{west} := 76.295ft$
East Span Length	$L_{east} := 76.63ft$

Mild Reinforcement

Top Cover for Top Slab $c_{t.t} := 2.5in$

Top Cover for Bottom Slab $c_{t.b} := 1in$

Bottom Cover for Top/Bottom Slab $c_b := 1in$

Diameter of #10 $D_{N8} := 1in$

Diameter of #7 $D_{N7} := 0.875in$

Diameter of #5 $D_{N5} := 0.625in$

Diameter of #4 $D_{N4} := 0.5in$

Vertical Reinforcement Spacing in Web $s_w := 8in$

Young's Modulus of Reinforcement $E_{sw} := 200GPa = 2.901 \times 10^4 \cdot ksi$

	Bar Size	Qty	Total Area
Bar in Top Slab (Layer 1)	#7	40	$A_{s.1} := 40 \cdot 0.6in^2 = 24 \cdot in^2$
Bar in Top Slab (Layer 2)	#7	30	$A_{s.2} := 30 \cdot 0.6in^2 = 18 \cdot in^2$
Bar in Top Slab (Layer 3)	#7	10	$A_{s.3} := 10 \cdot 0.6in^2 = 6 \cdot in^2$
Bar in Web (Layer 1)	#8 and #10	8	$A_{s.4} := 4 \cdot 1.27 \cdot in^2 + 4 \cdot 0.79in^2 = 8.24 \cdot in^2$
Bar in Web (Layer 2)	#4	8	$A_{s.5} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 3)	#4	8	$A_{s.6} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Web (Layer 4)	#4	8	$A_{s.7} := 8 \cdot 0.2in^2 = 1.6 \cdot in^2$
Bar in Bottom Slab (Layer 1)	#8	36	$A_{s.8} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$
Bar in Bottom Slab (Layer 2)	#8	36	$A_{s.9} := 36 \cdot 0.79in^2 = 28.44 \cdot in^2$

Distance from Extreme Compressive Fiber to Centroid of Reinforcement

Bar in Top Slab (Layer 1) $d_{s.1} := c_{t.t} + D_{N5} + \frac{D_{N7}}{2} = 3.563 \cdot in$

Bar in Top Slab (Layer 2) $d_{s.2} := \left(h_{ft} - c_b\right) - D_{N5} - \frac{D_{N7}}{2} = 6.437 \cdot in$

Bar in Top Slab (Layer 3) $d_{s.3} := \frac{\left(h_{fillet}\right)}{2} + h_{ft} = 11.5 \cdot in$

Bar in Web (Layer 1) $d_{s.4} := h - 32.5in = 18.5 \cdot in$

Bar in Web (Layer 2) $d_{s.5} := d_{s.4} + s_w = 26.5 \cdot in$

Bar in Web (Layer 3) $d_{s.6} := d_{s.5} + s_w = 34.5 \cdot in$

Bar in Web (Layer 4)	$d_{s.7} := d_{s.6} + s_w = 42.5 \cdot in$
Bar in Bottom Slab (Layer 1)	$d_{s.8} := h - \left(h_{fb} - c_b - D_{N4} - \frac{D_{N8}}{2} \right) = 47 \cdot in$
Bar in Bottom Slab (Layer 2)	$d_{s.9} := h - \left(c_b + D_{N4} + \frac{D_{N8}}{2} \right) = 49 \cdot in$
Maximum Spacing in the Layer Closest to the Tension Face	$S_{max} := \frac{(3ft + 8.375in)}{4} = 11.094 \cdot in$
Minimum Spacing in the Layer Closest to the Tension Face	$S_{min} := \frac{(1ft + 5.375in)}{4} = 4.344 \cdot in$

3. Reinforcement Distribution

Exposure Condition	Class 2	
Exposure Factor (5.6.7)	$\gamma_e := 0.75$	
Aggregate Size	$a_g := 0.75in$	Note: as per AASHTO C5.4.2.1
Positive Moment at SLS	$M_{sls} := 5668kip \cdot ft$	Note: maximum moment at west span from csibridge "SE1-2 SUP_STRENGTH_ENV" load combination
Thickness of Concrete Cover Measured from Extreme Tension Fiber to Center of the Flexural Reinforcement Located Closest Thereto	$d_c := c_b + D_{N4} + \frac{D_{N8}}{2} = 2 \cdot in$	
Ratio of Flexural Strain at Extreme Tension Face to the Strain at the Centroid of the Reinforcement Layer Nearest the Tension Face (5.6.7)	$\beta_s := 1 + \frac{d_c}{0.7 \cdot (h - d_c)} = 1.058$	
Tensile strain in nonprestressed reinforcement at SLS	$\varepsilon_s := 0.97 \cdot 10^{-3}$	Note: From Response 2000 Tensile Stress in Nonprestressed Reinforcement at SLS
Tensile stress in nonprestressed reinforcement at SLS	$f_{ss} := \varepsilon_s \cdot E_s = 28.137 \cdot ksi$	
Max Spacing of Nonprestressed Reinforcement Closest to the Tension Face (5.6.7)	$S_{max.limit} := \frac{700 \cdot \gamma_e \cdot in}{\beta_s \cdot \frac{f_{ss}}{ksi}} - 2 \cdot d_c = 13.63 \cdot in$	
Minimum Spacing for CIP Concrete (5.10.3.1.1)	$S_{min.limit} := \max(1.5 \cdot D_{N8}, 1.5in, 1.5 \cdot a_g) = 1.5 \cdot in$	
Maximum Allowable Spacing (5.10.3.2)	$S_{max.limit} := S_{max.limit} = 13.63 \cdot in$	

4. Deformations

Deflection due to Vehicle and Pedestrian Load

$\Delta := 0.0108ft = 0.13 \cdot in$

Note: deflection is the maximum absolute deflection from csibridge load combination "LL_Sup_Def"

Cracking Moment (5.6.3.5.2)

$M_{cr} := f_r \cdot \frac{I_{22}}{h - y} = 3.625 \times 10^3 \cdot kip \cdot ft$

Maximum Moment in a Component at the Stage for which Deformation is Computed

$M_a := 6068kip \cdot ft$

Note: M.sls is from csibridge. It's the maximum moment in mid span in load combintation 'SE1-1 SUP_DEF_ENV'

Curvature at Ma (from Response 2000)

$\rho := \frac{24.69}{10^6} \frac{rad}{in}$

Effective Moment of Inertia (5.6.3.5.2)

$I_{eff} := \frac{M_a}{E_c \cdot \rho} = 6.015 \times 10^5 \cdot in^4$

Adjustment Factor for Deflection

$\alpha := \frac{I_{eff}}{I_{22}} = 0.234$

Adjusted Deflection due to Vehicle and Pedestrian Load

$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.553 \cdot in$

Maximum Allowable Deflection (2.5.2.6.2)

$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$

5. Summary

Maximum Spacing in Design

$S_{max} = 11.094 \cdot in$

$\frac{(S_{max})}{(S_{max.limit})} = 0.814$

Maximum Allowable Spacing

$S_{max.limit} = 13.63 \cdot in$

Maximum Spacing Check

$if(S_{max} \le S_{max.limit}, "ok", "fail") = "ok"$

Minimum Spacing in Design

$S_{min} = 4.344 \cdot in$

Minimum Allowable Spacing

$S_{min.limit} = 1.5 \cdot in$

$\frac{(S_{min.limit})}{(S_{min})} = 0.345$

Minimum Spacing Check

$if(S_{min} \ge S_{min.limit}, "ok", "fail") = "ok"$

Adjusted Deflection due to Vehicle and Pedestrian Load

$\Delta_{adjusted} := \frac{\Delta}{\alpha} = 0.553 \cdot in$

Maximum Allowable Deflection (2.5.2.6.2)

$\Delta_{max.limit} := \frac{L_{east}}{1000} = 0.92 \cdot in$

$\frac{(\Delta_{adjusted})}{(\Delta_{max.limit})} = 0.602$

Deflection Check

$if(\Delta_{adjusted} \le \Delta_{max.limit}, "ok", "fail") = "ok"$

Response-2000 v 1.0.5

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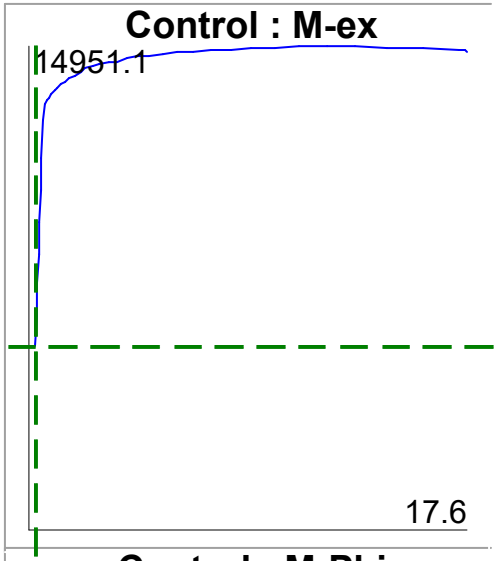
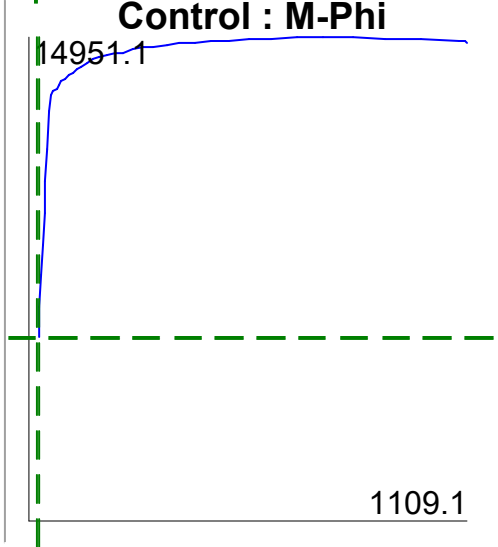
Control : M-ex



Control : M-Phi



ε_{x0} = 0.25 ms

ϕ = 23.06 rad/10^6 in

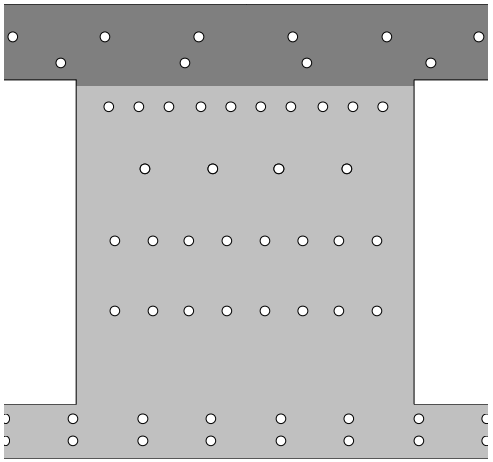
γ_{xy}(avg) = 0.00 ms

Axial Load = -0.8 kips

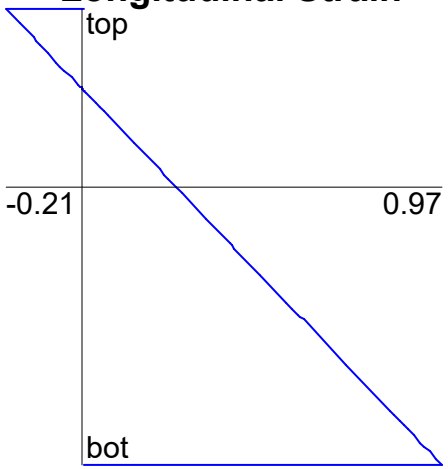
Moment:= 5663.9 ft-kips

Shear = 0.0 kips

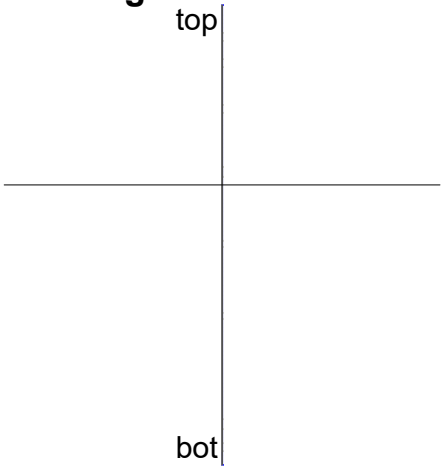
Cross Section



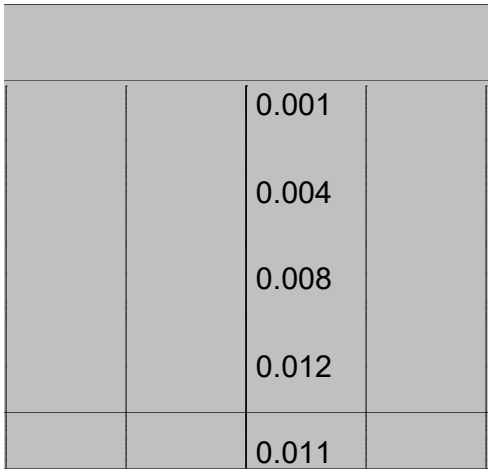
Longitudinal Strain



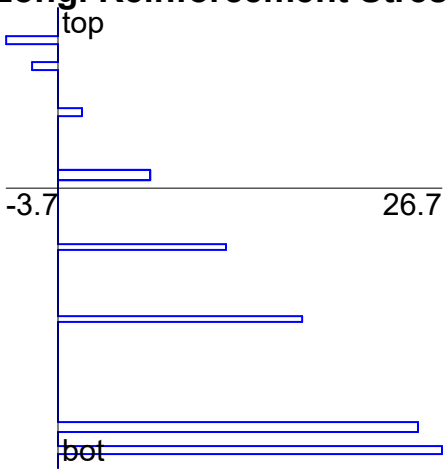
Shrinkage & Thermal Strain



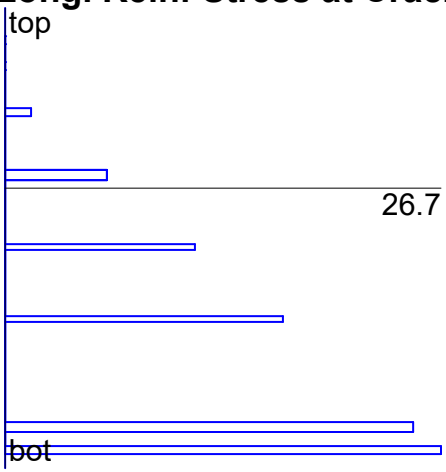
Crack Diagram



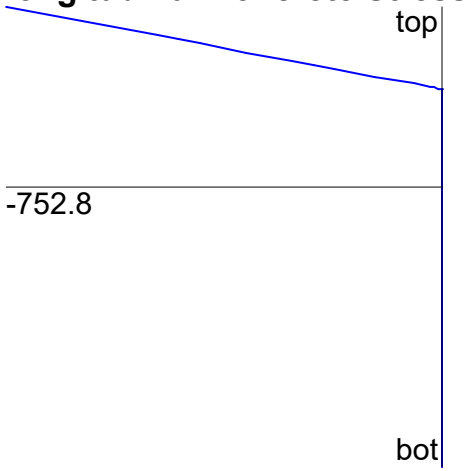
Long. Reinforcement Stress



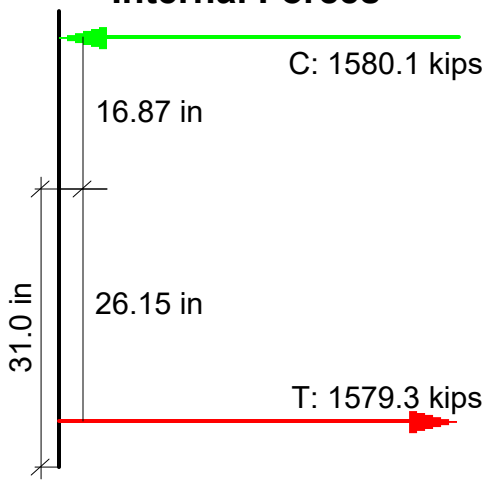
Long. Reinf Stress at Crack



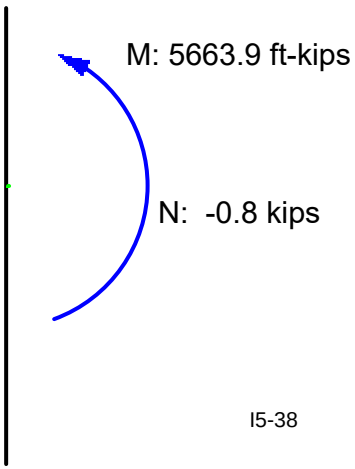
Longitudinal Concrete Stress

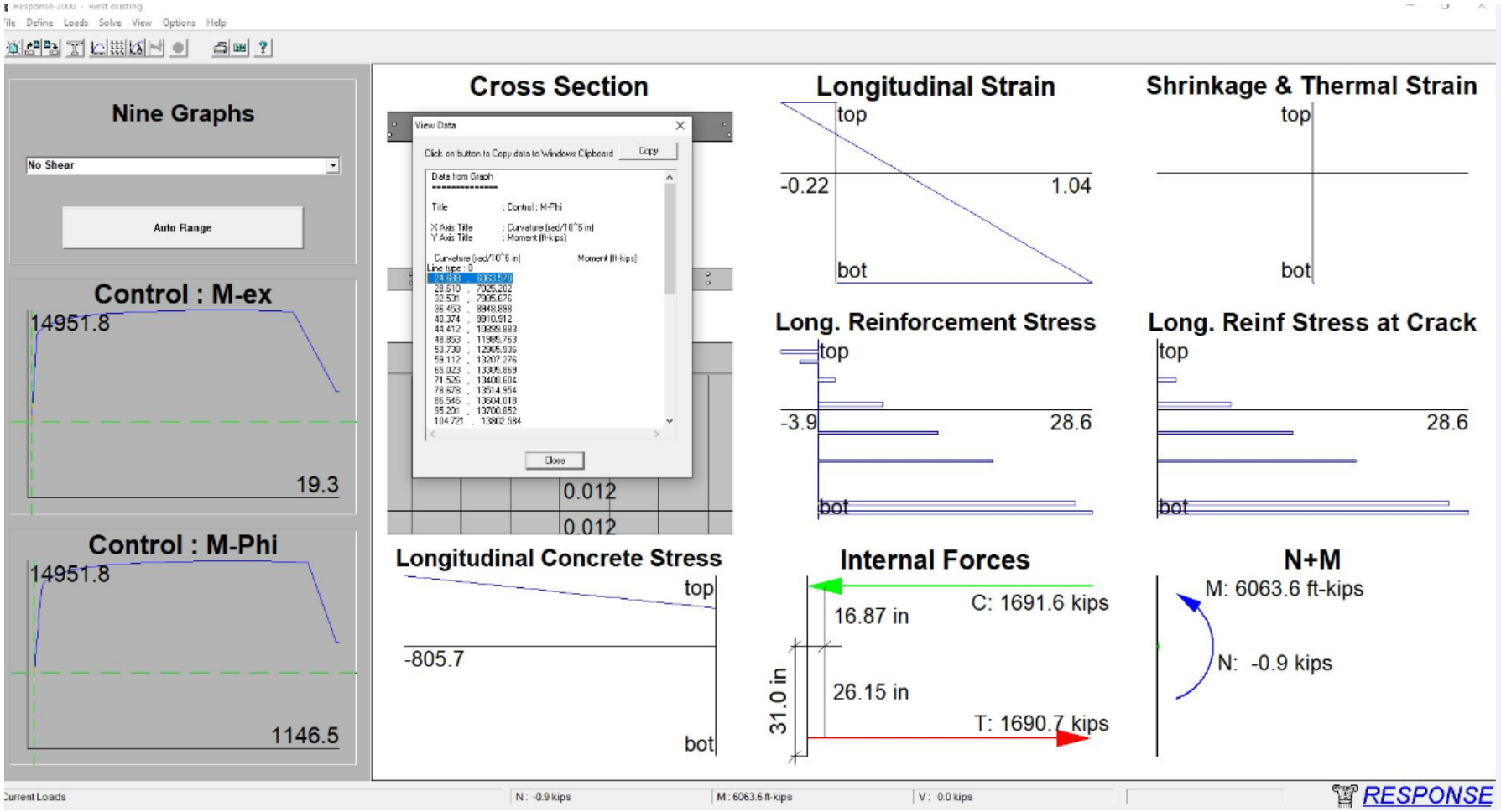


Internal Forces



N+M

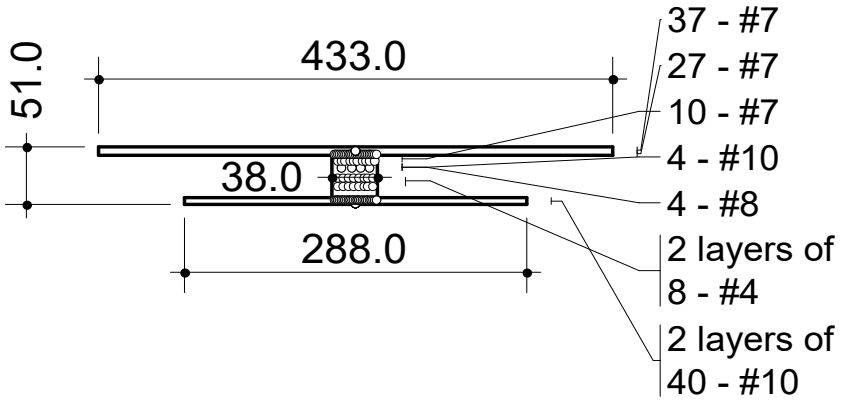



Define Loads Solve View Options Help
Nine Graphs
No Shear
Auto Range
Control : M-ex
14951.8
19.3
Control : M-Phi
14951.8
1146.5
Cross Section
View Data
Click on button to Copy data to Windows Clipboard
Copy
Data from Graph
Title : Control : M-Phi
X Axis Title : Curvature (rad/10^6 in)
Y Axis Title : Moment (ft-kips)
Curvature (rad/10^6 in) Moment (ft-kips)
Line type : 0
28.510 , 7025.202
32.531 , 7985.676
36.493 , 8948.898
40.374 , 9910.912
44.412 , 10886.683
48.853 , 11965.763
53.730 , 12965.936
59.112 , 13207.276
65.023 , 13305.869
71.526 , 13409.604
78.678 , 13514.954
86.546 , 13604.019
95.201 , 13700.652
104.721 , 13802.584
Close
0.012
0.012
Longitudinal Strain
top
-0.22
1.04
bot
Shrinkage & Thermal Strain
top
bot
Long. Reinforcement Stress
top
-3.9
28.6
bot
Long. Reinf Stress at Crack
top
28.6
bot
Longitudinal Concrete Stress
top
-805.7
bot
Internal Forces
C: 1691.6 kips
16.87 in
31.0 in
26.15 in
T: 1690.7 kips
N+M
M: 6063.6 ft-kips
N: -0.9 kips
Current Loads
N: -0.9 kips
M: 6063.6 ft-kips
V: 0.0 kips
RESPONSE

Appendix 3 - Cracked Cross-section Stresses
Response 2000 Section Inputs

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6920.5	8029.1
Inertia (in^4)	2587651.1	3167868.7
y_t (in)	20.4	22.3
y_b (in)	30.6	28.7
S_t (in^3)	126752.4	142110.0
S_b (in^3)	84605.3	110346.7

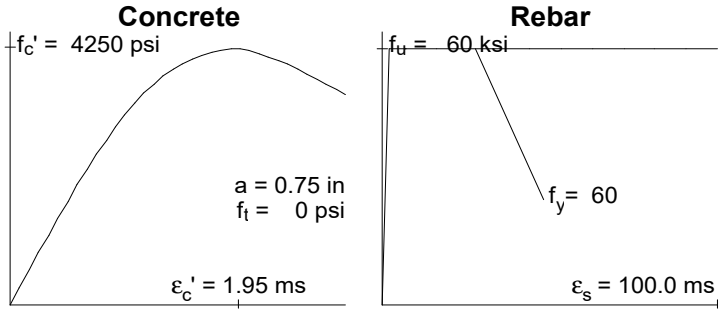


Crack Spacing

2 x dist + 0.1 d_b /ρ

Loading (N,M,V + dN,dM,dV)

-883 , 7573 , 0.0 + 0.0 , 1.0 , 0.0



All dimensions in inches
Clear cover to reinforcement = 1.36 in

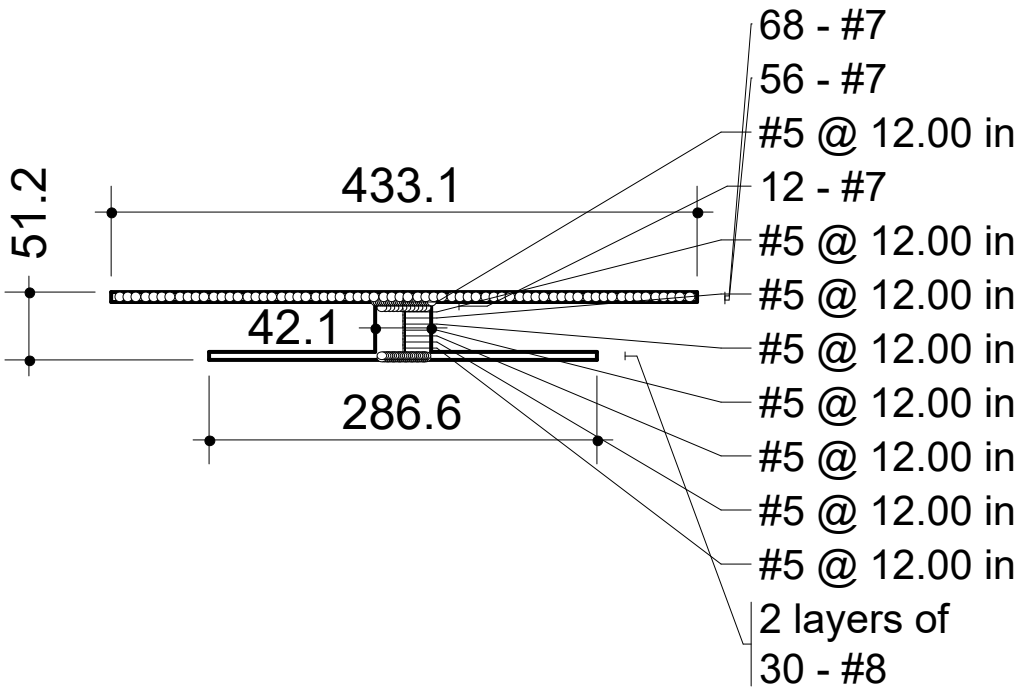
East Span Section

2021/6/16

Concrete Shrinkage/Thermal Strain -0.02 ms

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6946.1	7854.4
Inertia (in^4)	2551466.0	2938023.0
y_t (in)	20.2	20.3
y_b (in)	31.0	30.9
S_t (in^3)	126613.8	144870.9
S_b (in^3)	82230.0	95082.6



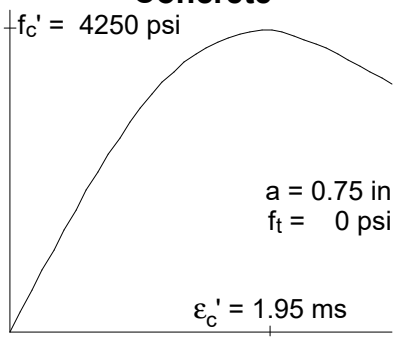
Crack Spacing

2 x dist + 0.1 d_b /ρ

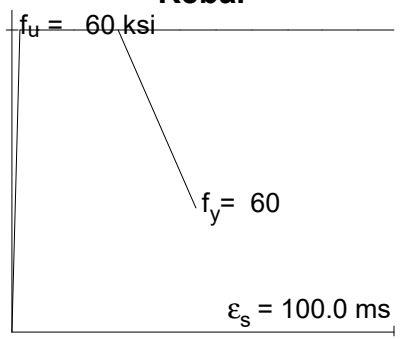
Loading (N,M,V + dN,dM,dV)

-734 , -4091 , 0.0 + 0.0 , -1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to transverse reinforcement = 0.69 in

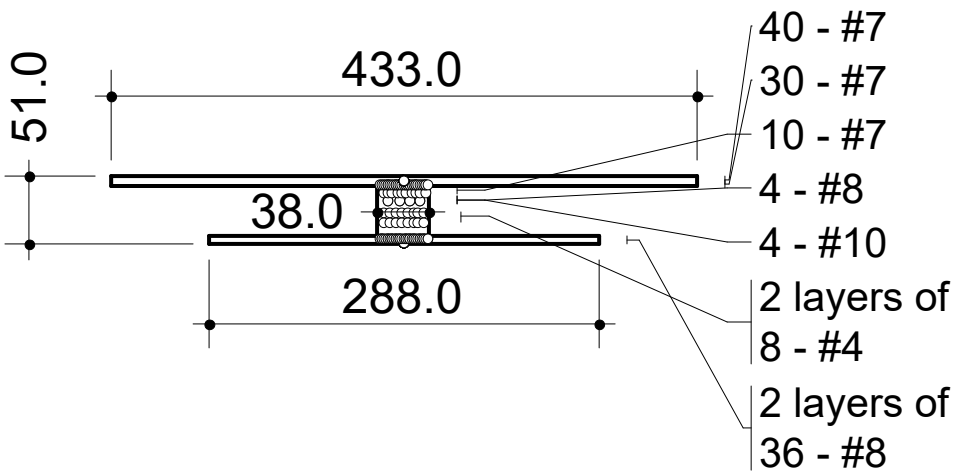
Pier Section

I5-41

2021/2/10

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6795.5	7614.6
Inertia (in^4)	2512262.9	2891210.2
y_t (in)	20.0	20.8
y_b (in)	31.0	30.2
S_t (in^3)	125818.3	138916.1
S_b (in^3)	80955.6	95775.5



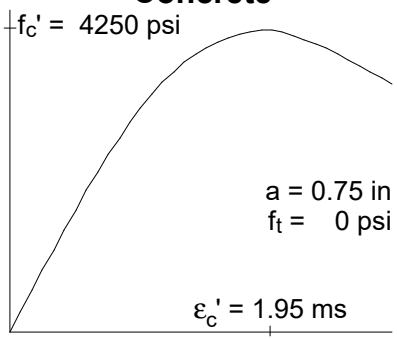
Crack Spacing

2 x dist + 0.1 d_b /ρ

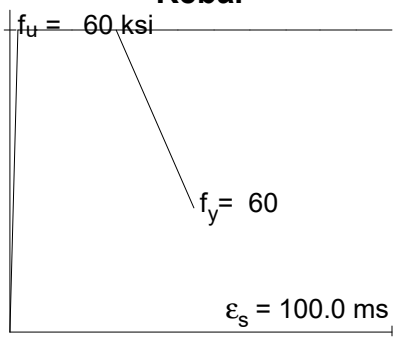
Loading (N,M,V + dN,dM,dV)

-1507 , 3791 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



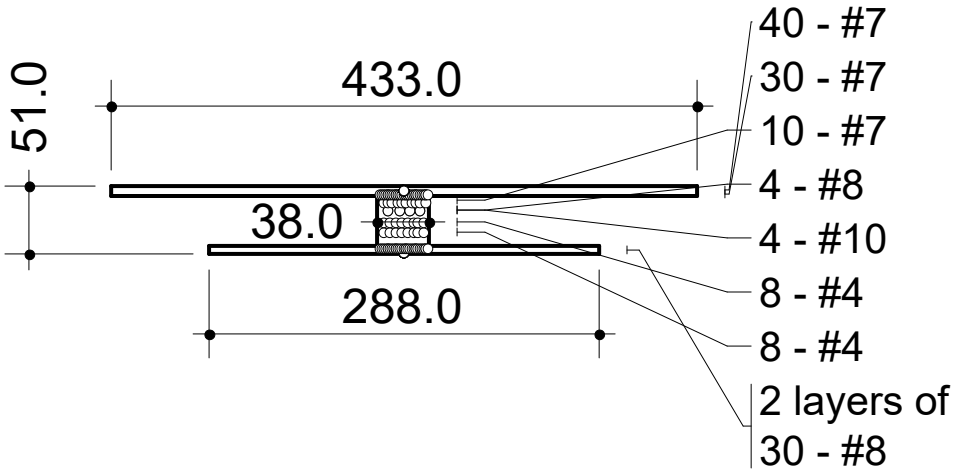
All dimensions in inches
Clear cover to reinforcement = 1.50 in

West Span Section

2021/6/16

Geometric Properties

	Gross Conc.	Trans (n=8.04)
Area (in^2)	6795.5	7547.8
Inertia (in^4)	2512262.9	2842240.7
y_t (in)	20.0	20.6
y_b (in)	31.0	30.4
S_t (in^3)	125818.3	138144.5
S_b (in^3)	80955.6	93416.1



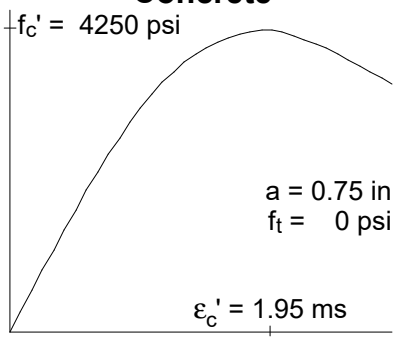
Crack Spacing

2 x dist + 0.1 d_b /ρ

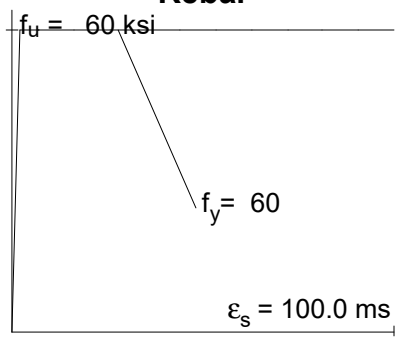
Loading (N,M,V + dN,dM,dV)

-1161 , 1421 , 0.0 + 0.0 , 1.0 , 0.0

Concrete



Rebar



All dimensions in inches
Clear cover to reinforcement = 1.50 in

Interface Section

I5-43

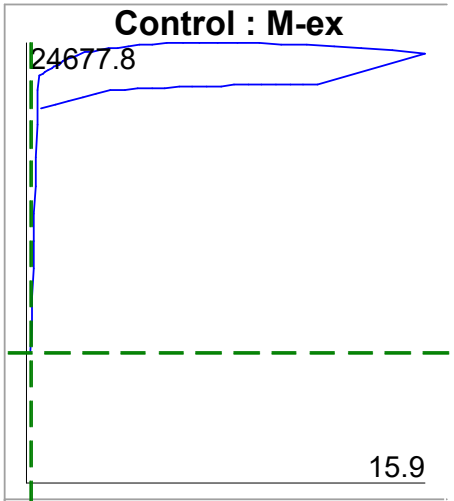
2021/6/16

Response-2000 v 1.0.5

Existing Structure/Unfactored Dead Load/East Span

2021/11/25 - 11:56 am

Control : M-ex



Control : M-Phi

ε_{x0} = 0.16 ms

ϕ = 18.56 rad/10^6 in

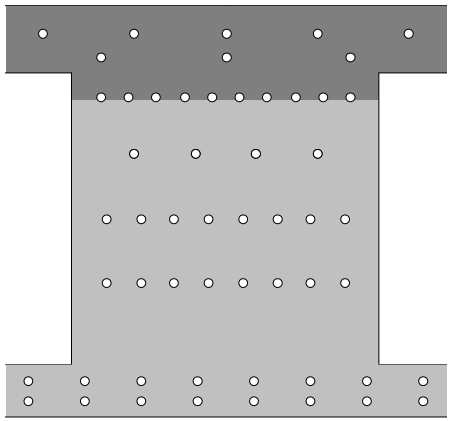
γ_{xy}(avg) = 0.00 ms

Axial Load = -0.8 kips

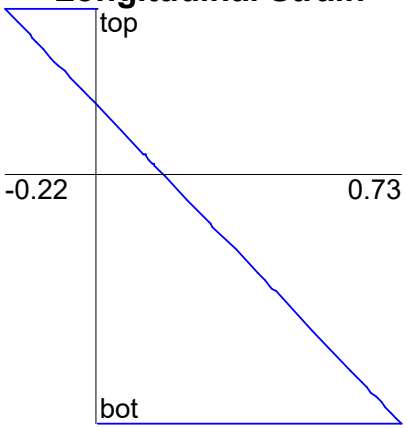
Moment:= 7340.8 ft-kips

Shear = 0.0 kips

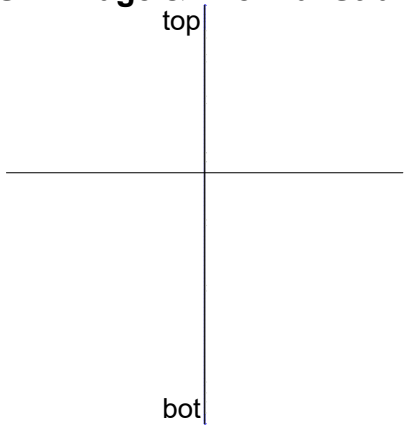
Cross Section



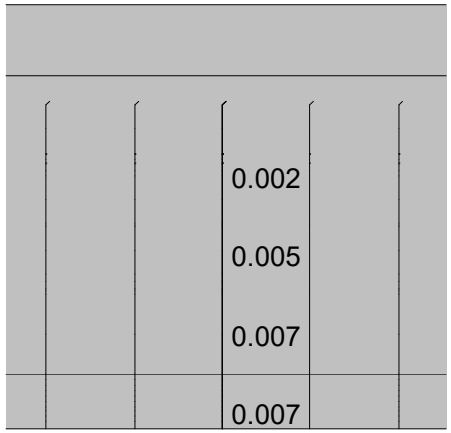
Longitudinal Strain



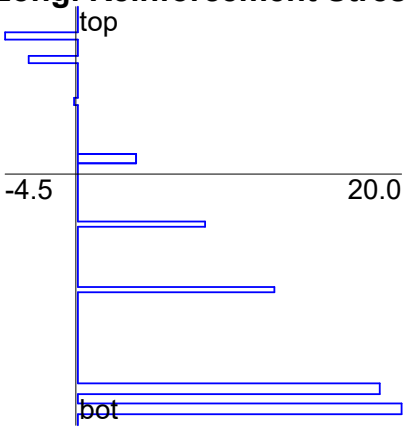
Shrinkage & Thermal Strain



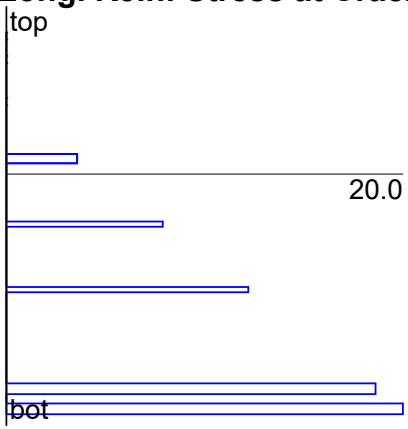
Crack Diagram



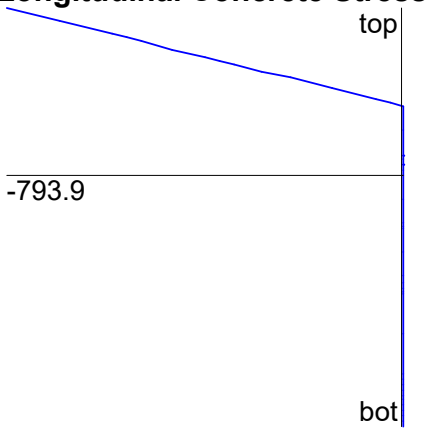
Long. Reinforcement Stress



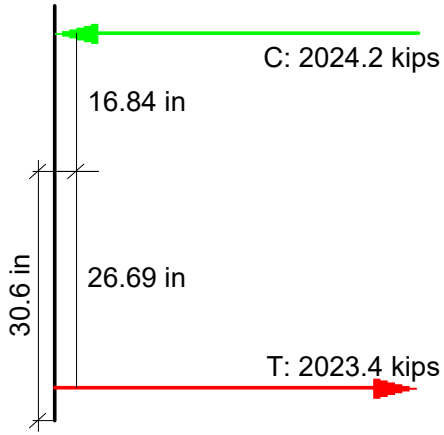
Long. Reinf Stress at Crack



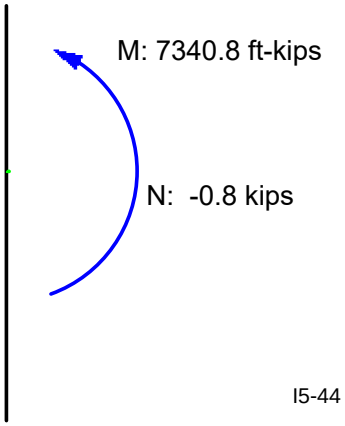
Longitudinal Concrete Stress



Internal Forces



N+M

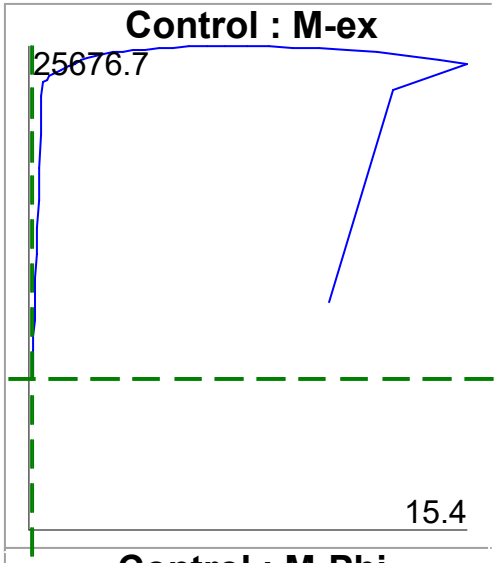


Response-2000 v 1.0.5

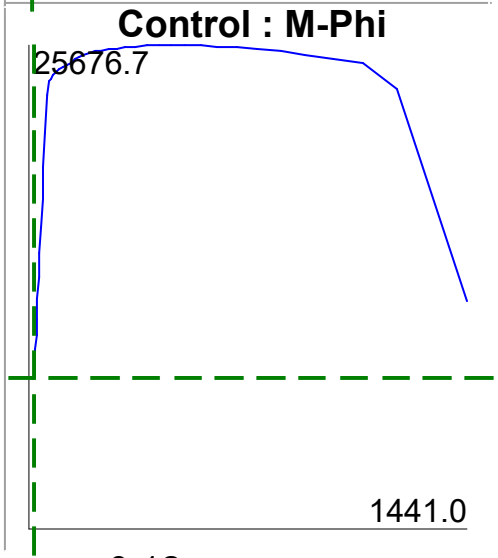
Existing Structure/Serviceability III/East Span

2021/11/25 - 2:13 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.12 ms

ϕ = 18.91 rad/10^6 in

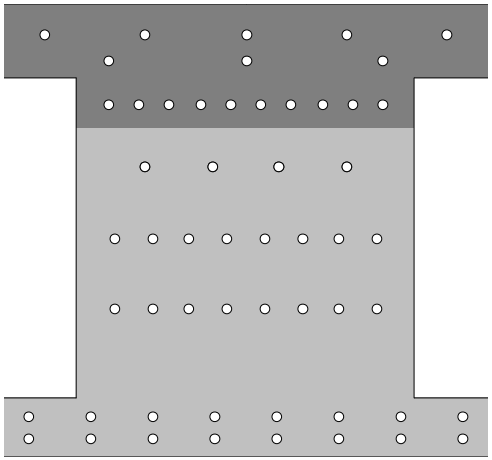
γ_{xy}(avg) = 0.00 ms

Axial Load = -759.5 kips

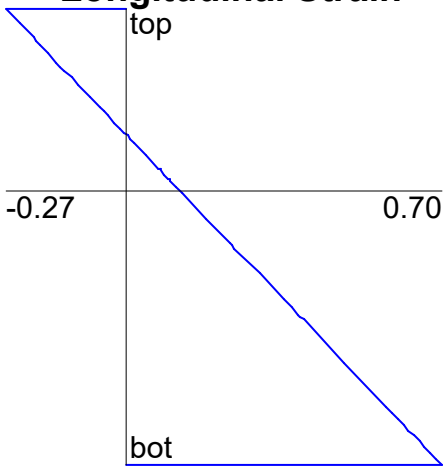
Moment:= 8015.8 ft-kips

Shear = 0.0 kips

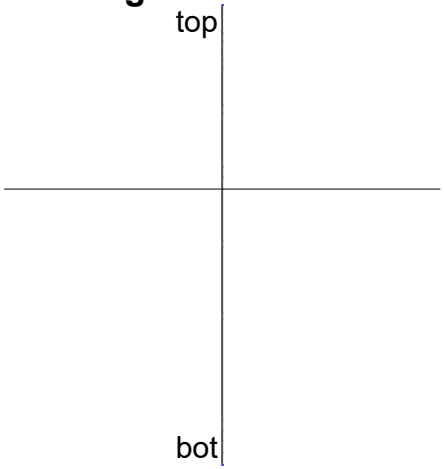
Cross Section



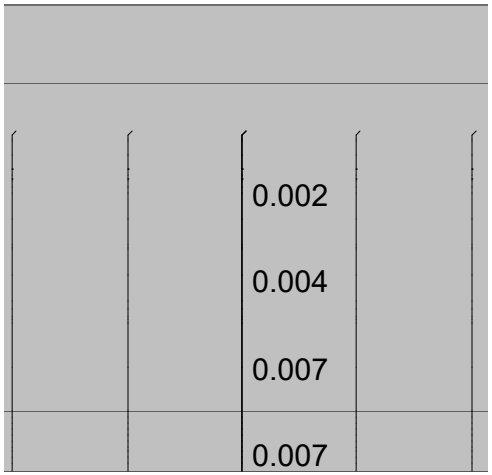
Longitudinal Strain



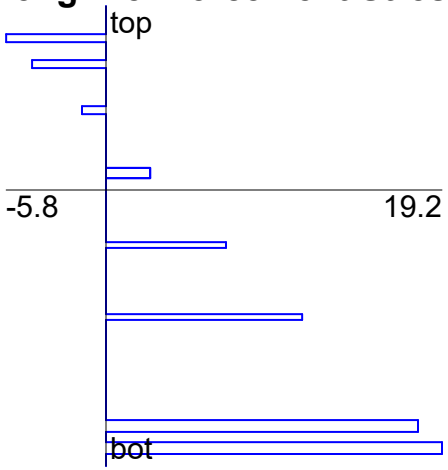
Shrinkage & Thermal Strain



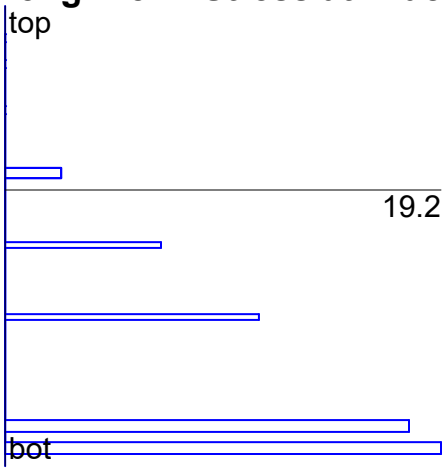
Crack Diagram



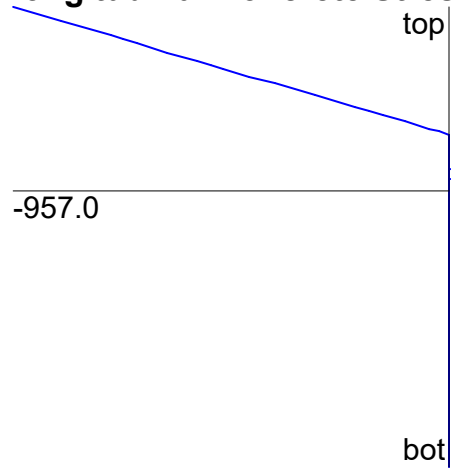
Long. Reinforcement Stress



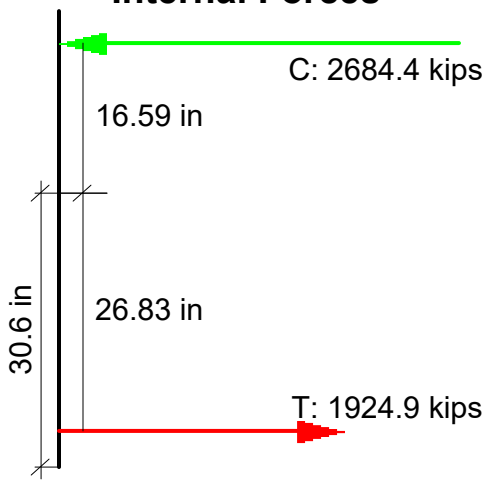
Long. Reinf Stress at Crack



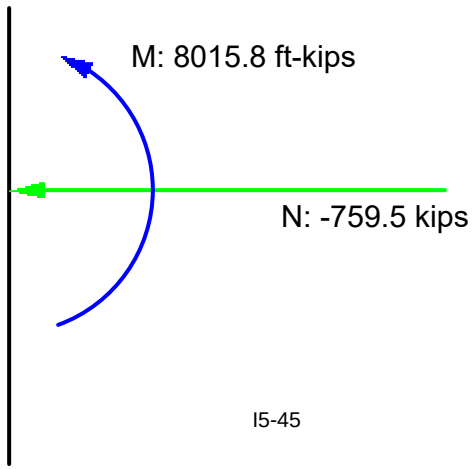
Longitudinal Concrete Stress



Internal Forces



N+M

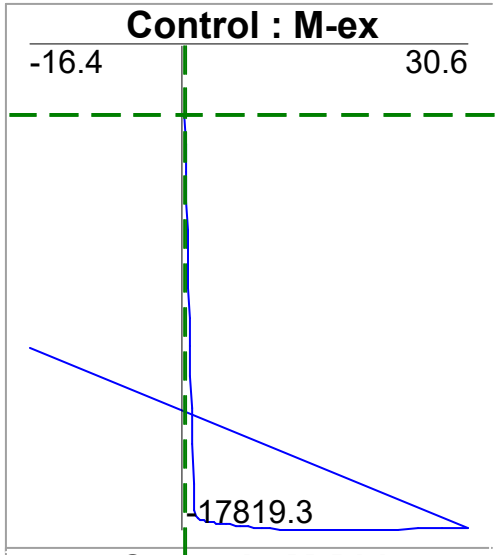


Response-2000 v 1.0.5

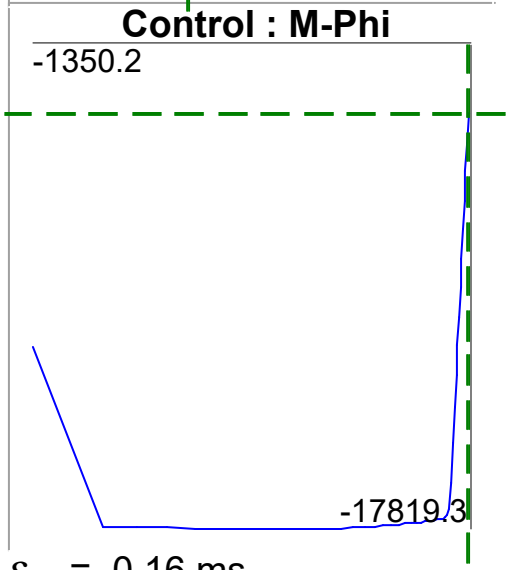
Existing Structure/Unfactored Dead Load /Pier

2021/11/25 - 11:59 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.16 ms

ϕ = -9.11 rad/10^6 in

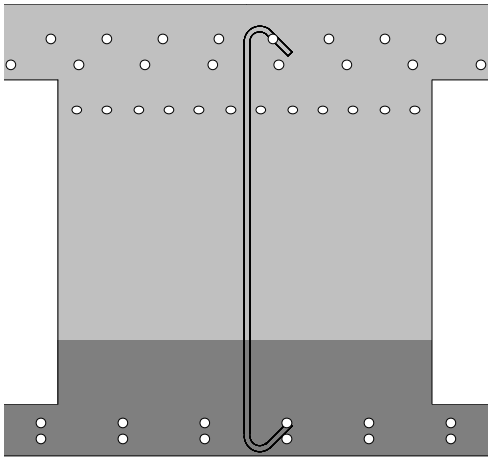
γ_{xy}(avg) = 0.00 ms

Axial Load = -0.5 kips

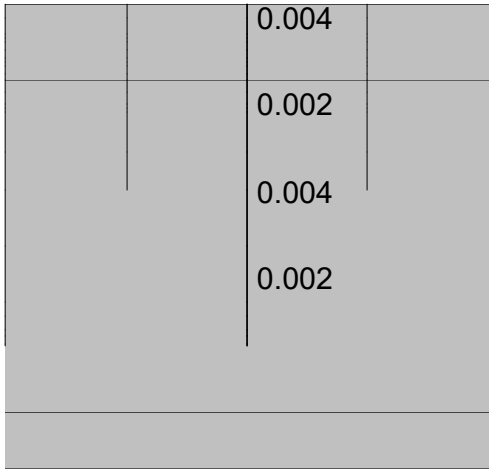
Moment:= -2505.1 ft-kips

Shear = 0.0 kips

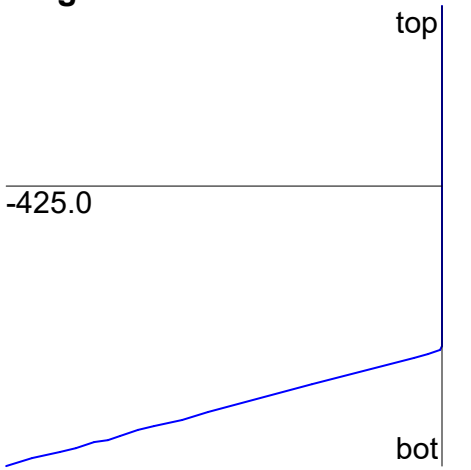
Cross Section



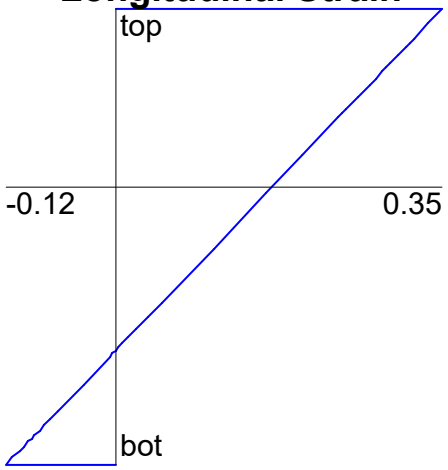
Crack Diagram



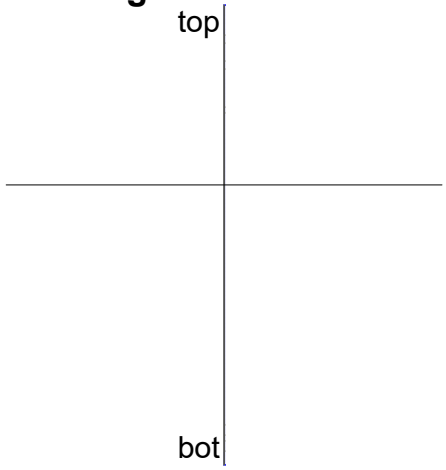
Longitudinal Concrete Stress



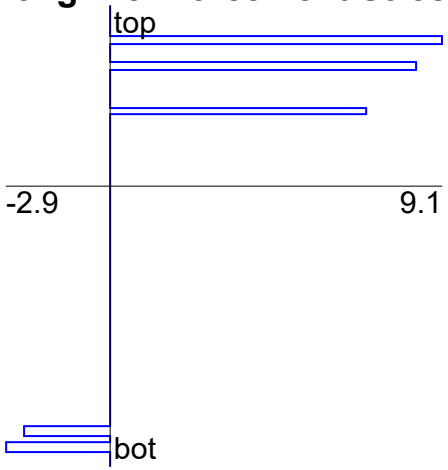
Longitudinal Strain



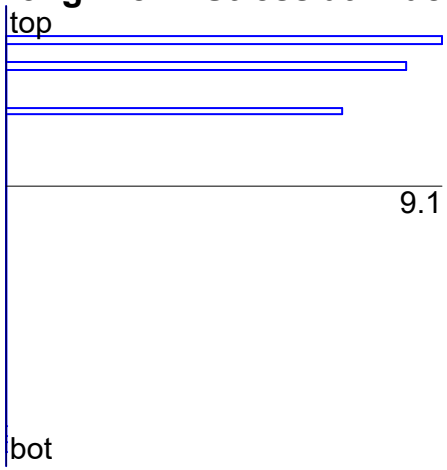
Shrinkage & Thermal Strain



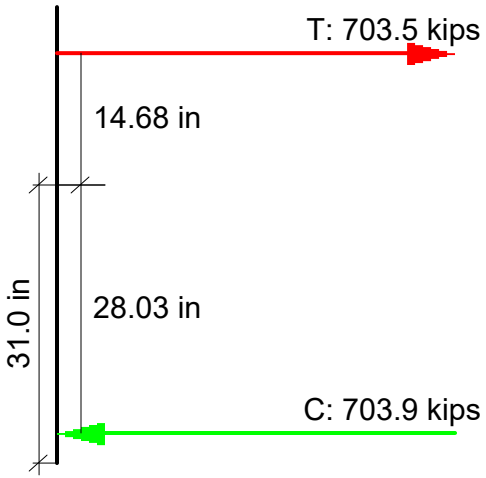
Long. Reinforcement Stress



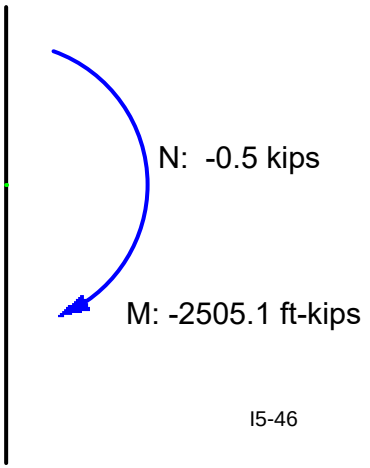
Long. Reinf Stress at Crack



Internal Forces



N+M

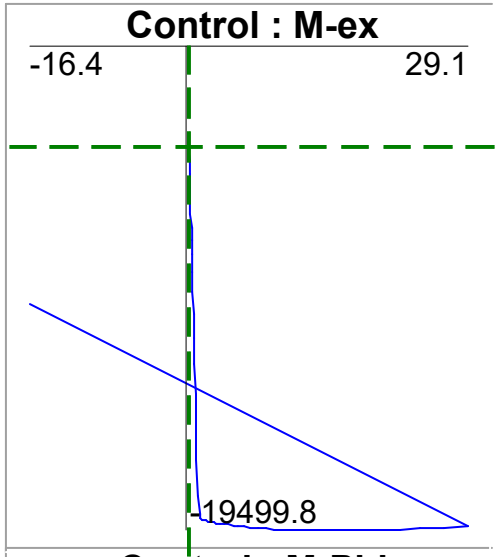


Response-2000 v 1.0.5

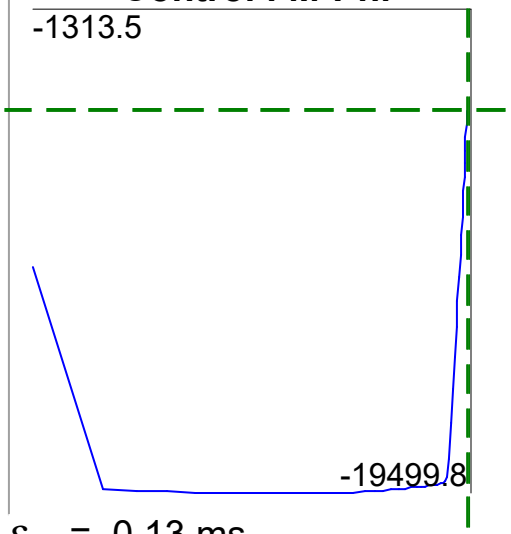
Existing Structure/Serviceability III /Pier

2021/11/25 - 2:14 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.13 ms

ϕ = -11.20 rad/10^6 in

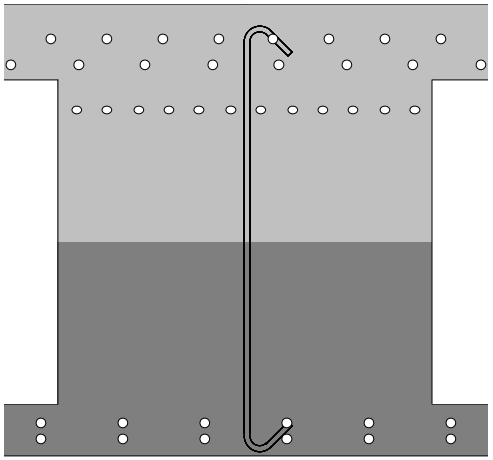
γ_{xy}(avg) = 0.00 ms

Axial Load = -734.2 kips

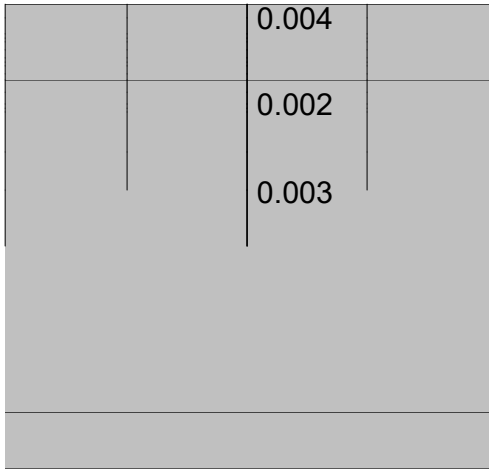
Moment:= -4088.5 ft-kips

Shear = 0.0 kips

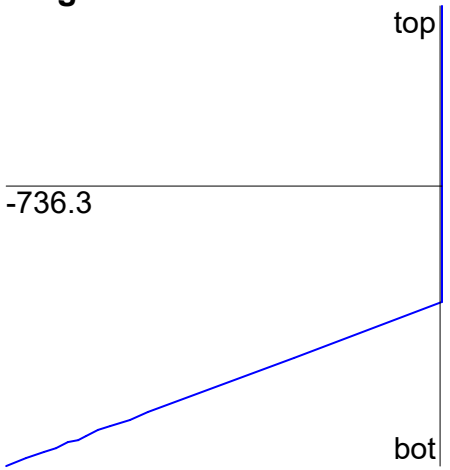
Cross Section



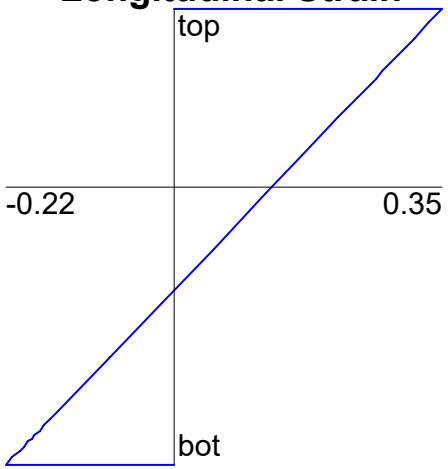
Crack Diagram



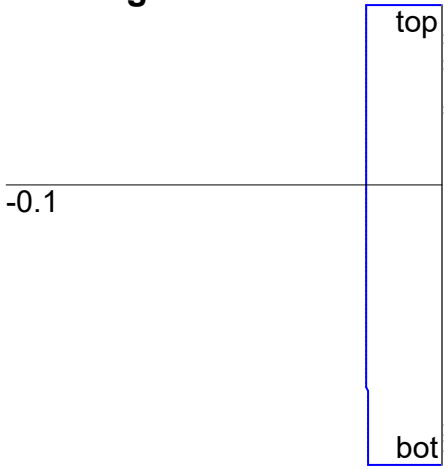
Longitudinal Concrete Stress



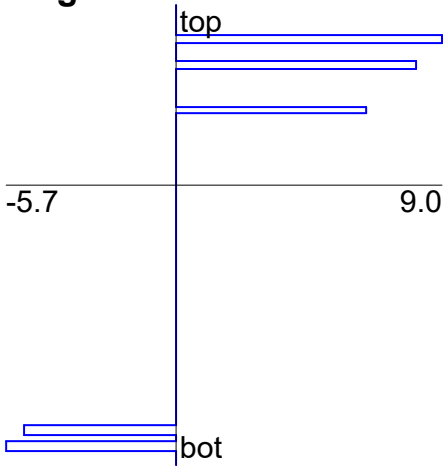
Longitudinal Strain



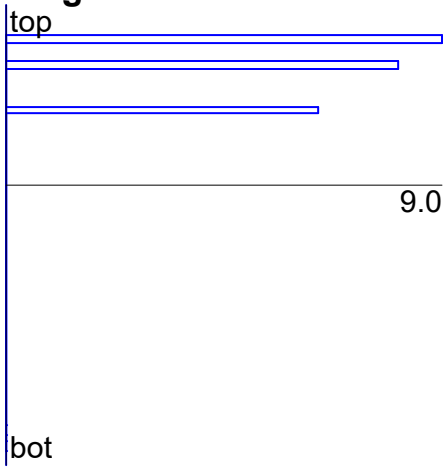
Shrinkage & Thermal Strain



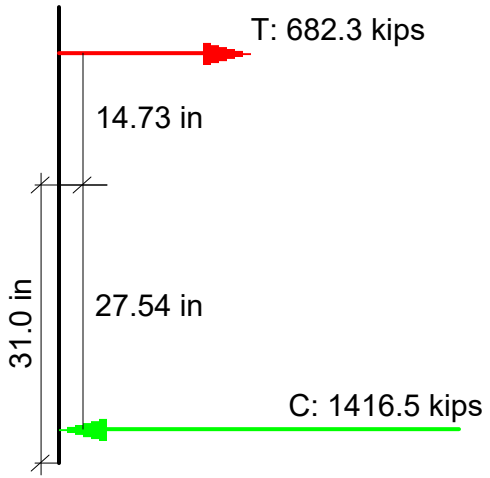
Long. Reinforcement Stress



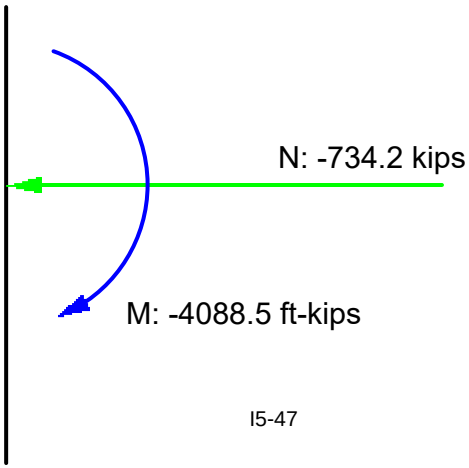
Long. Reinf Stress at Crack



Internal Forces



N+M

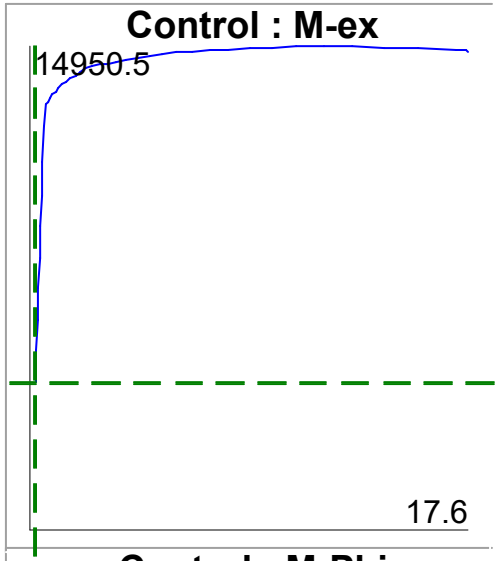


Response-2000 v 1.0.5

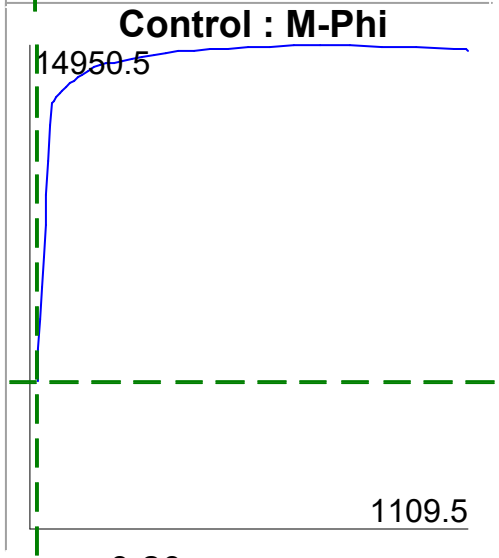
Existing Structure/Unfactored Dead Load /West Span

2021/11/25 - 0:02 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.20 ms

ϕ = 18.61 rad/10^6 in

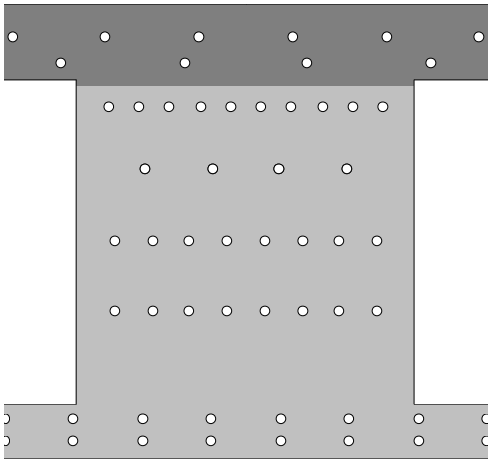
γ_{xy}(avg) = 0.00 ms

Axial Load = -0.7 kips

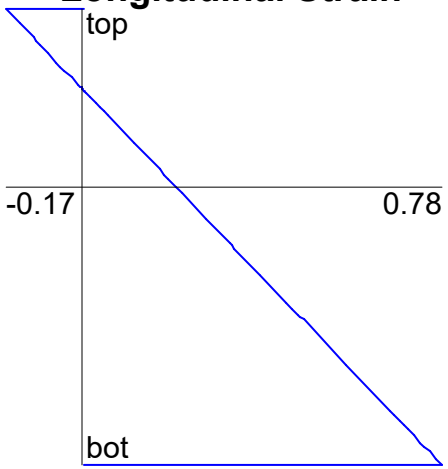
Moment:= 4570.3 ft-kips

Shear = 0.0 kips

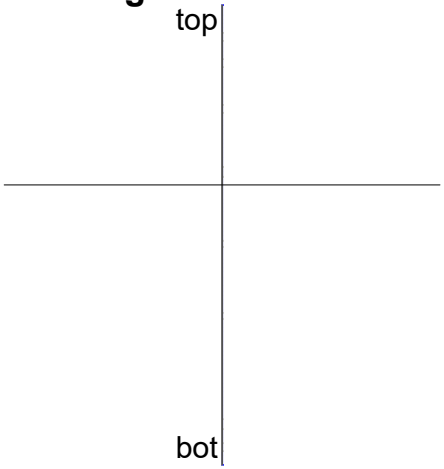
Cross Section



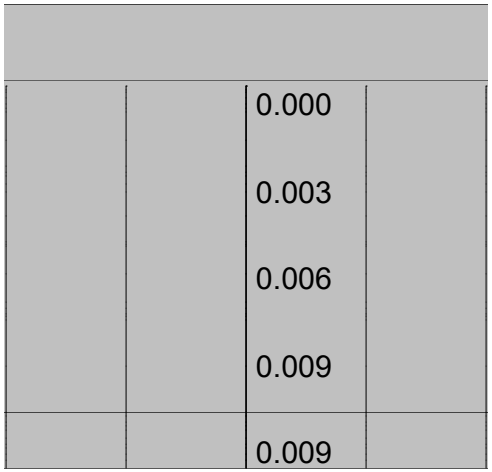
Longitudinal Strain



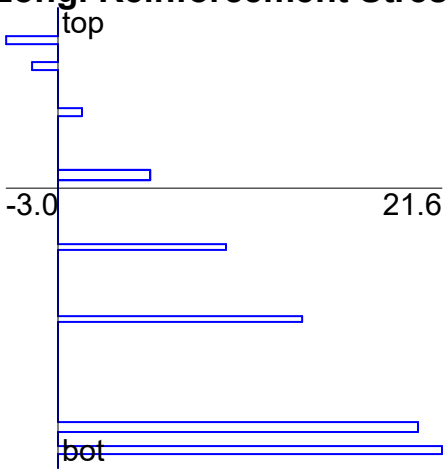
Shrinkage & Thermal Strain



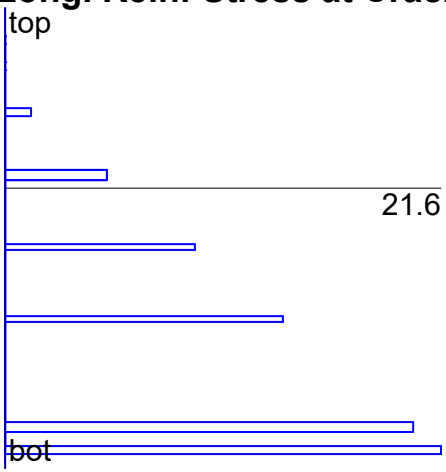
Crack Diagram



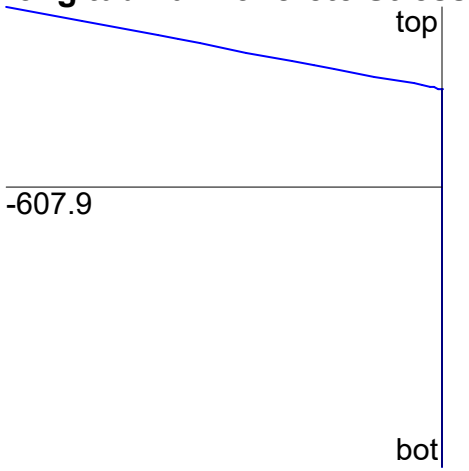
Long. Reinforcement Stress



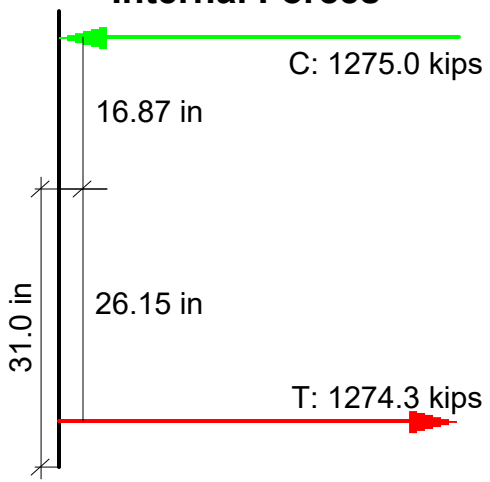
Long. Reinf Stress at Crack



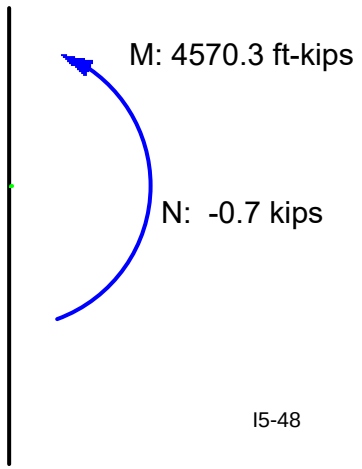
Longitudinal Concrete Stress



Internal Forces



N+M

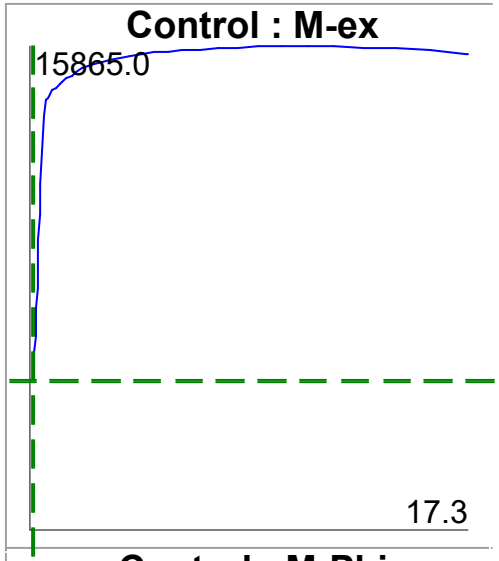


Response-2000 v 1.0.5

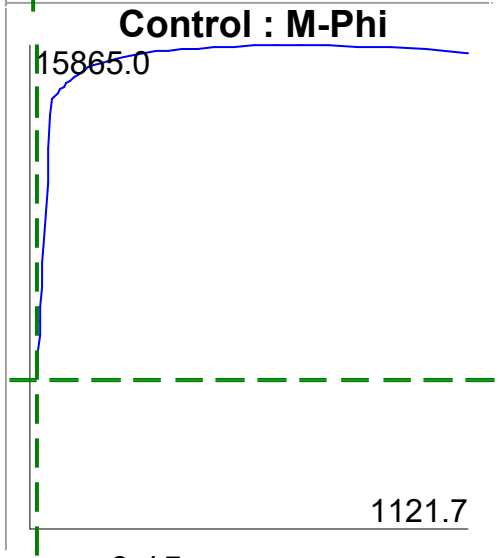
Existing Structure/Serviceability III /West Span

2021/11/25 - 2:15 pm

Control : M-ex



Control : M-Phi



ε_{x0} = 0.15 ms

ϕ = 17.34 rad/10^6 in

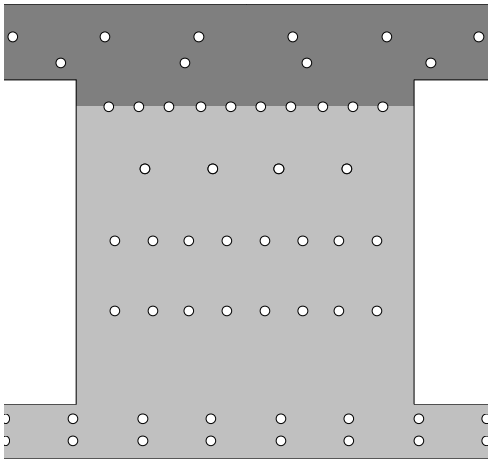
γ_{xy}(avg) = 0.00 ms

Axial Load = -696.3 kips

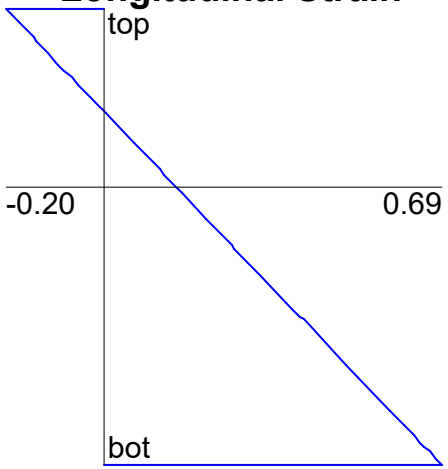
Moment:= 4889.9 ft-kips

Shear = 0.0 kips

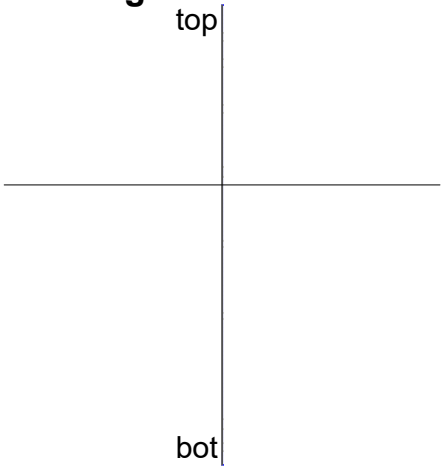
Cross Section



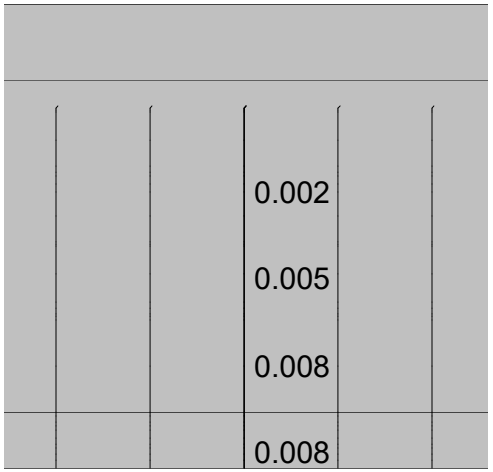
Longitudinal Strain



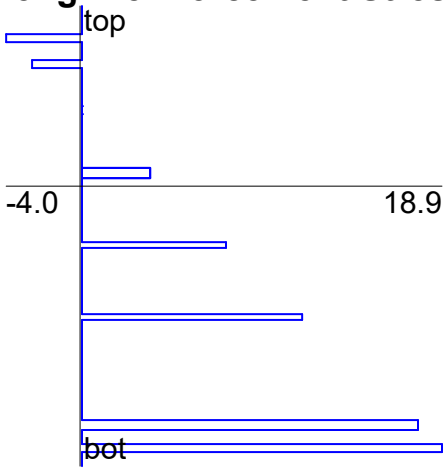
Shrinkage & Thermal Strain



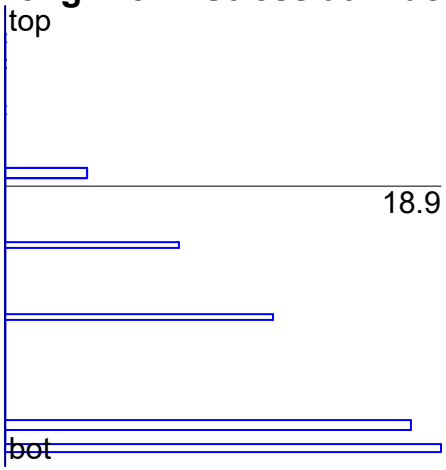
Crack Diagram



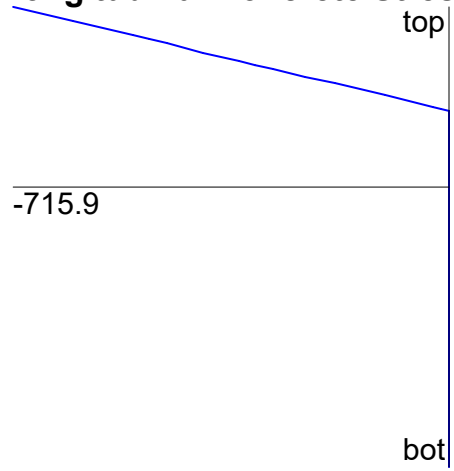
Long. Reinforcement Stress



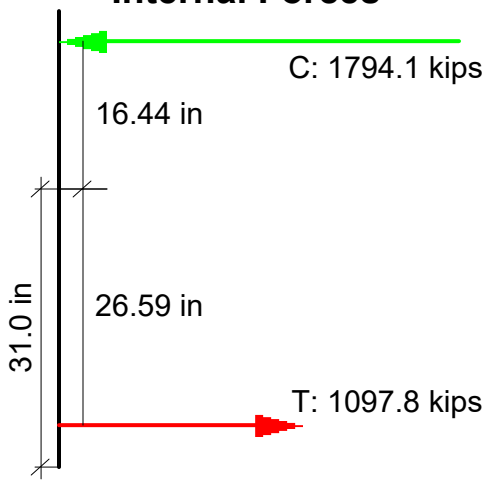
Long. Reinf Stress at Crack



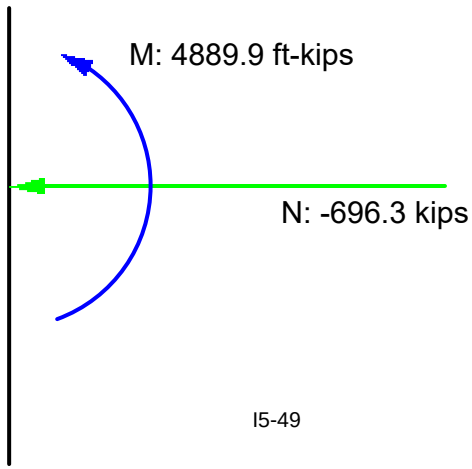
Longitudinal Concrete Stress



Internal Forces



N+M

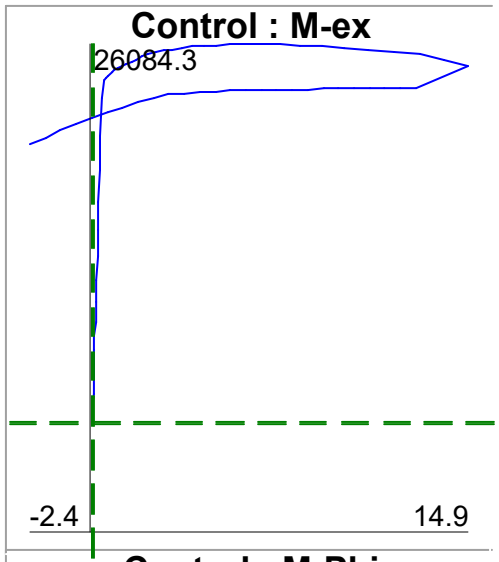


Response-2000 v 1.0.5

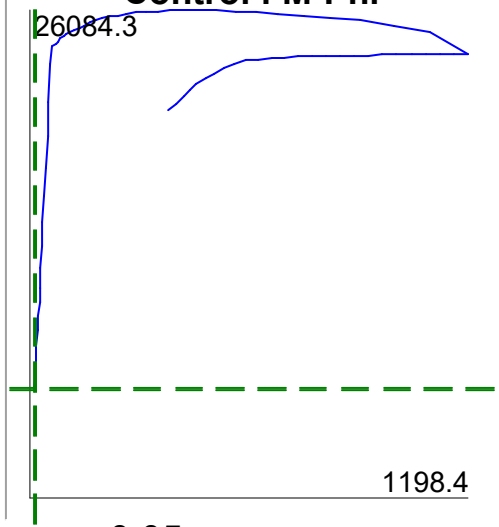
Proposed/Unfactored DC+PT /East Span

2021/11/25 - 11:18 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.05 ms

ϕ = 12.87 rad/10^6 in

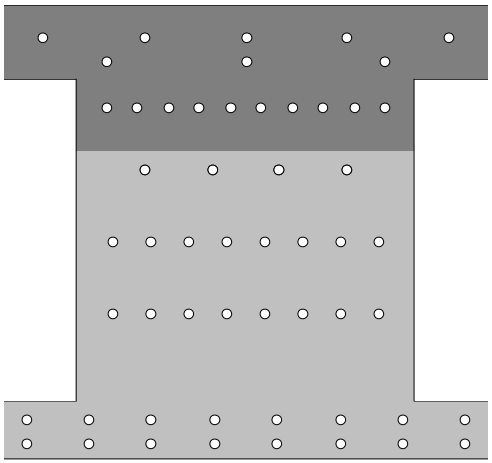
γ_{xy}(avg) = 0.00 ms

Axial Load = -1064.0 kips

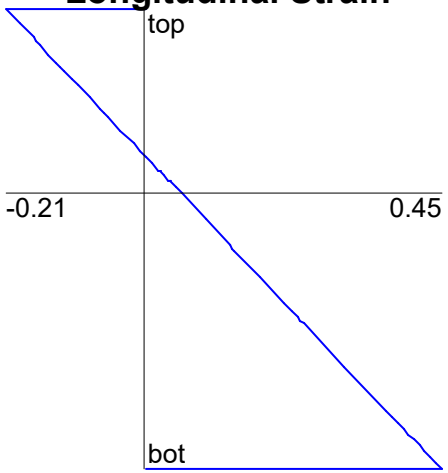
Moment:= 5845.1 ft-kips

Shear = 0.0 kips

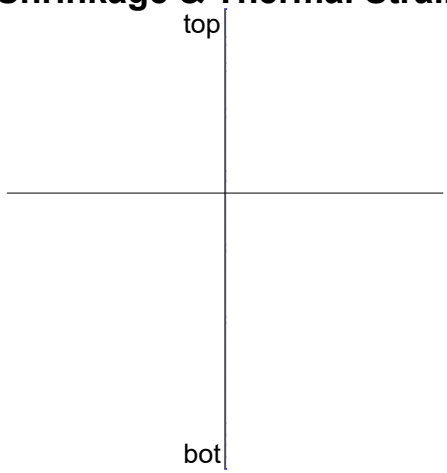
Cross Section



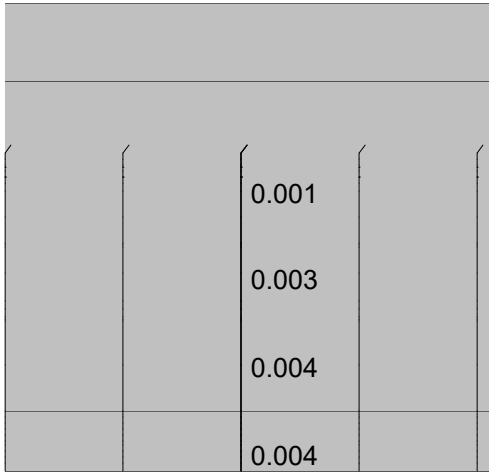
Longitudinal Strain



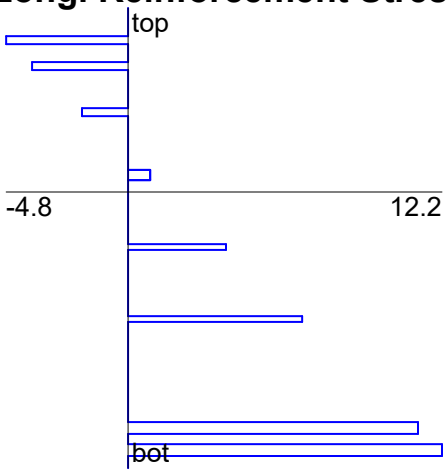
Shrinkage & Thermal Strain



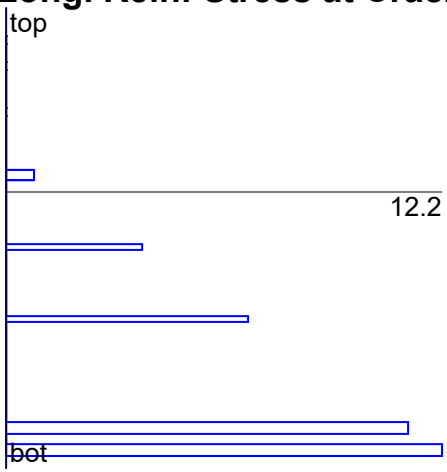
Crack Diagram



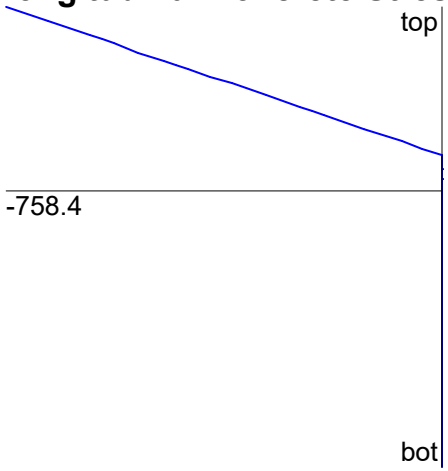
Long. Reinforcement Stress



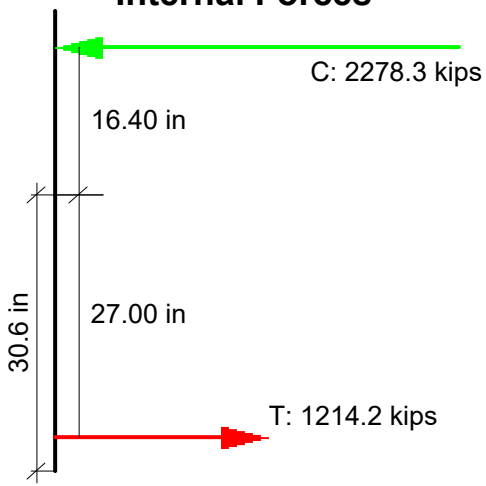
Long. Reinf Stress at Crack



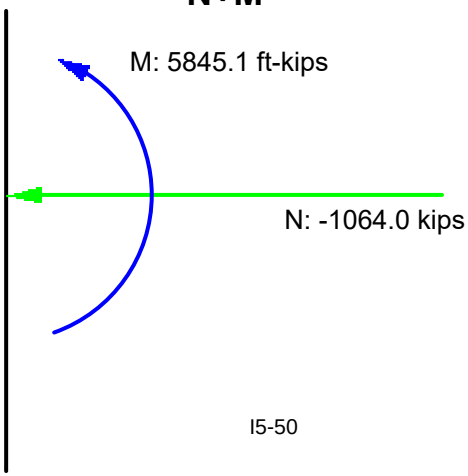
Longitudinal Concrete Stress



Internal Forces



N+M

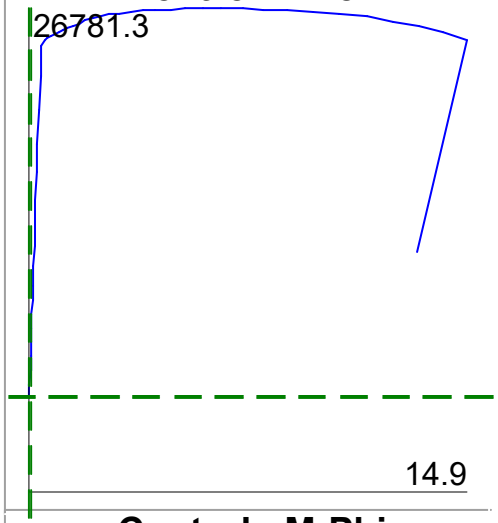


Response-2000 v 1.0.5

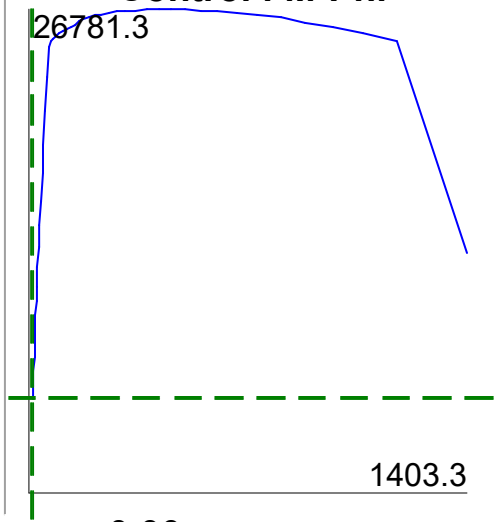
Proposed/Serviceability III/East Span

2021/11/25 - 11:32 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.00 ms

ϕ = 10.37 rad/10^6 in

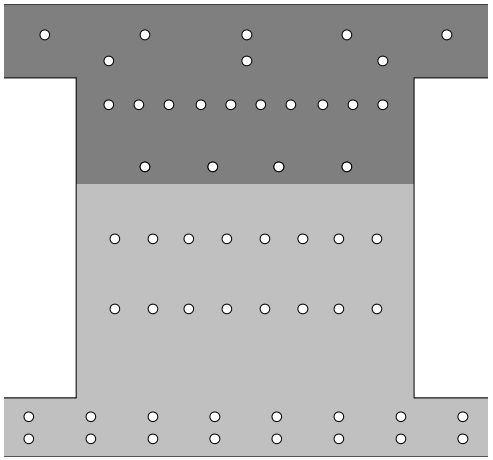
γ_{xy}(avg) = 0.00 ms

Axial Load = -1606.2 kips

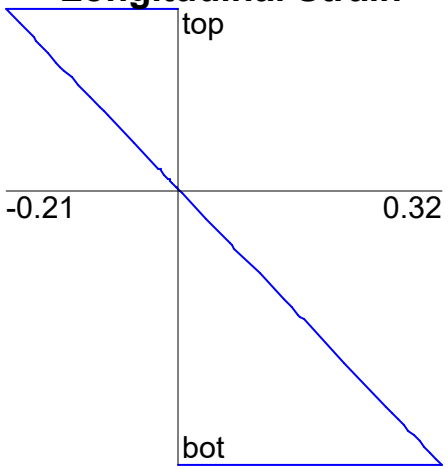
Moment:= 5230.7 ft-kips

Shear = 0.0 kips

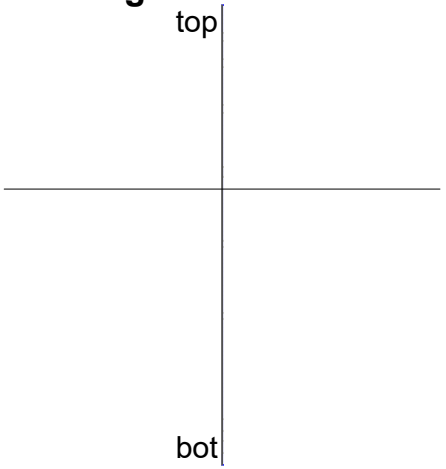
Cross Section



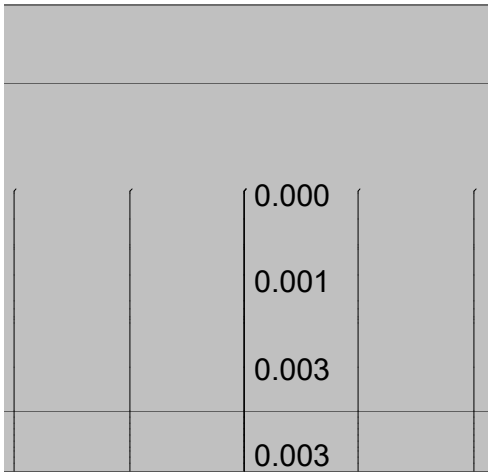
Longitudinal Strain



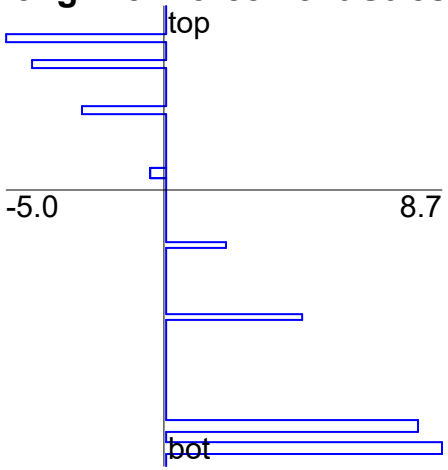
Shrinkage & Thermal Strain



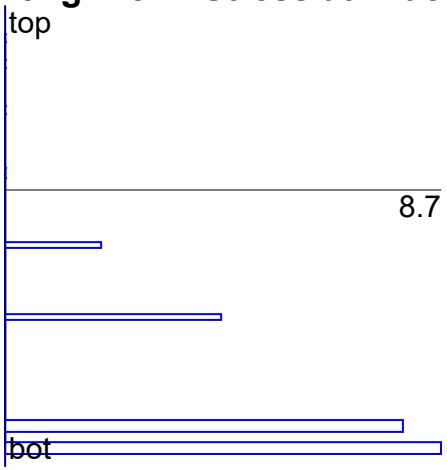
Crack Diagram



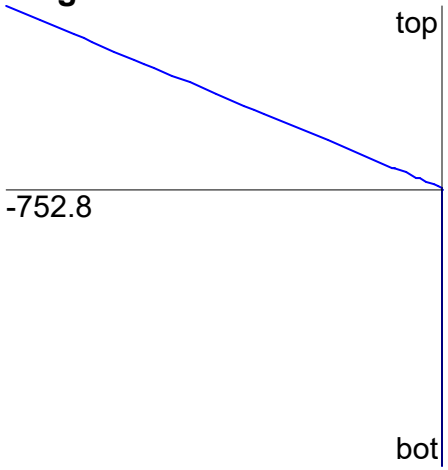
Long. Reinforcement Stress



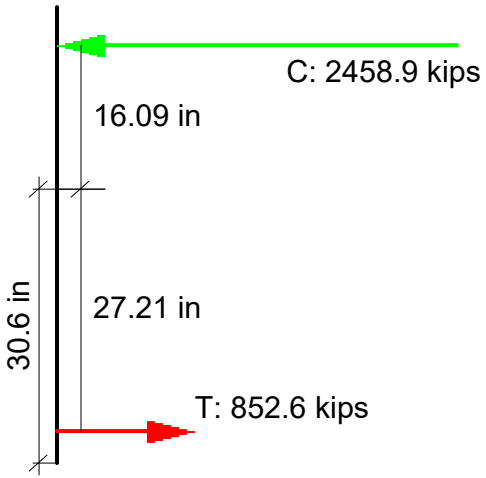
Long. Reinf Stress at Crack



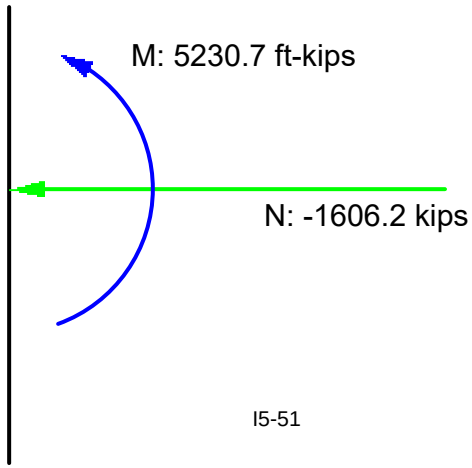
Longitudinal Concrete Stress



Internal Forces



N+M

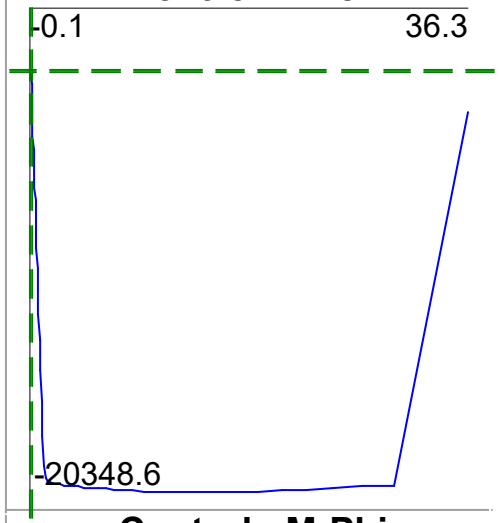


Response-2000 v 1.0.5

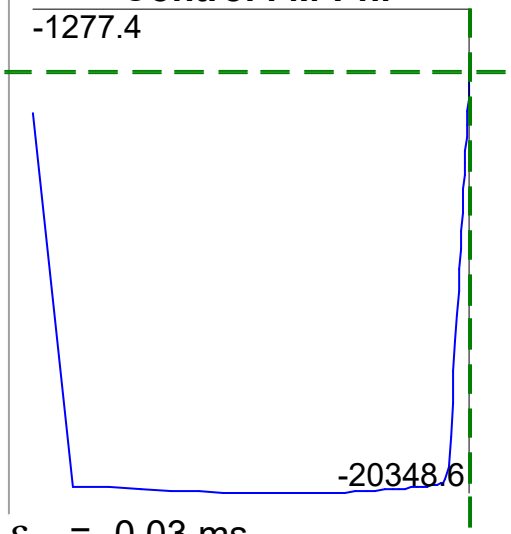
Proposed/Unfactored DC+PT/Pier

2021/11/25 - 11:22 am

Control : M-ex



Control : M-Phi



ε_{x0} = -0.03 ms

ϕ = -4.16 rad/10^6 in

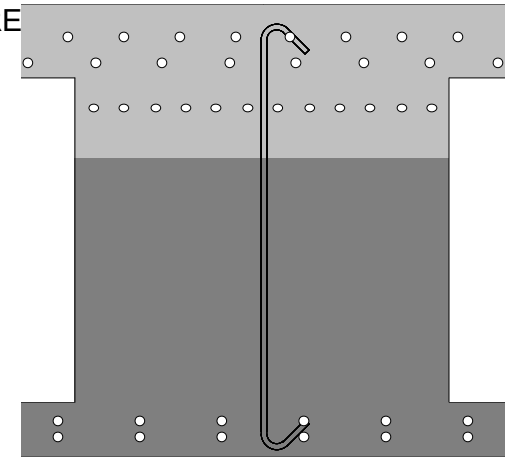
γ_{xy}(avg) = 0.00 ms

Axial Load = -1104.6 kips

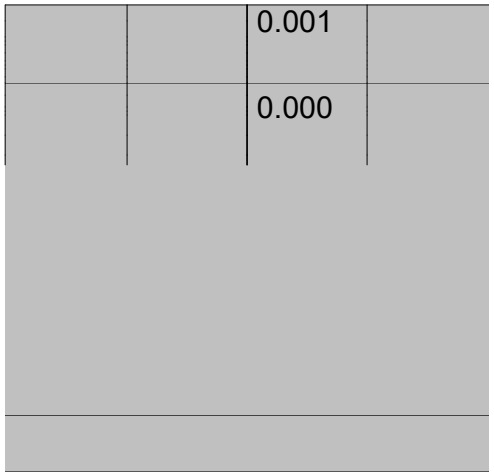
Moment:= -2646.0 ft-kips

Shear = 0.0 kips

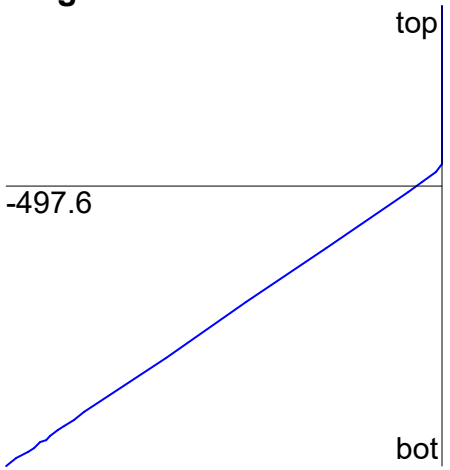
Cross Section



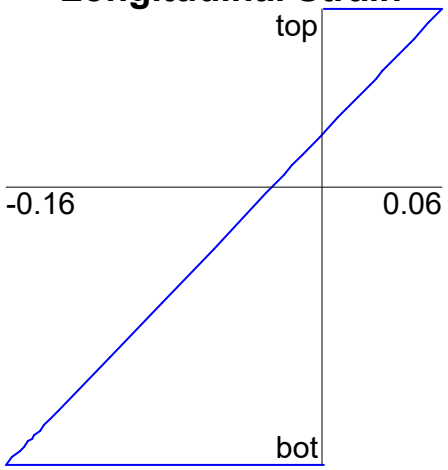
Crack Diagram



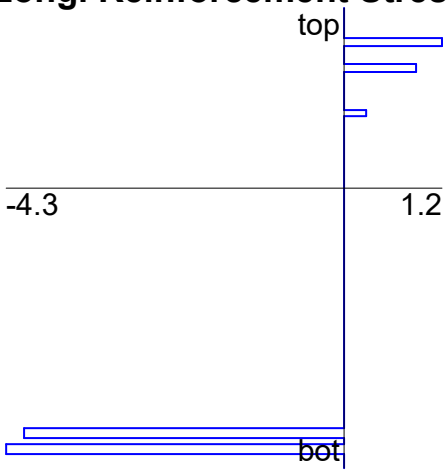
Longitudinal Concrete Stress



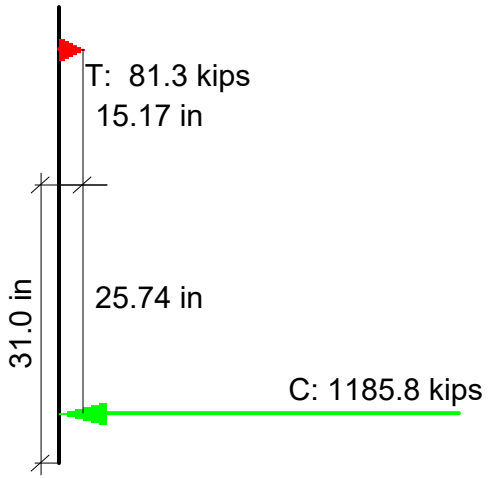
Longitudinal Strain



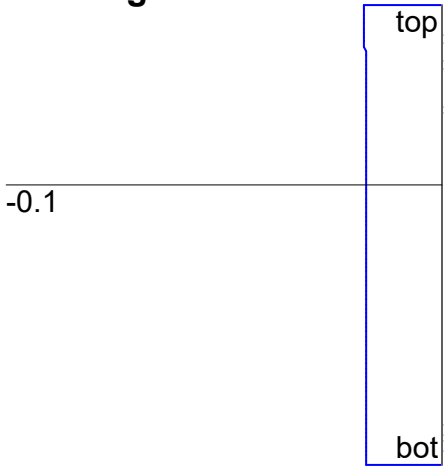
Long. Reinforcement Stress



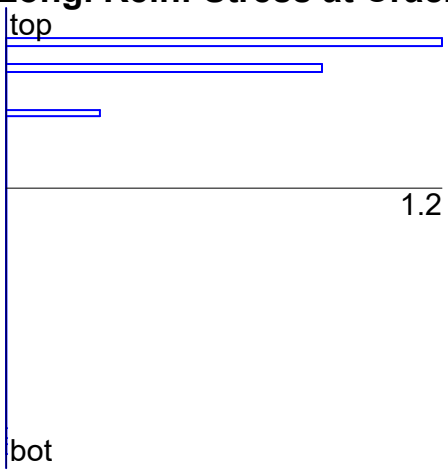
Internal Forces



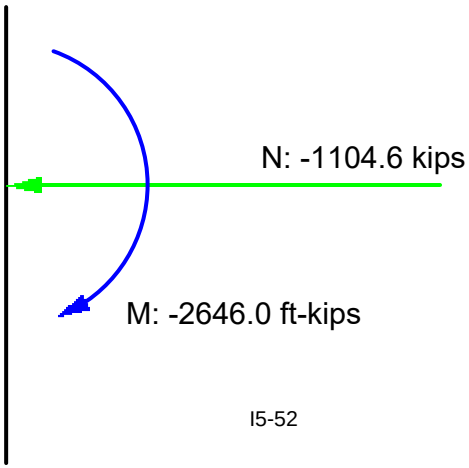
Shrinkage & Thermal Strain



Long. Reinf Stress at Crack



N+M

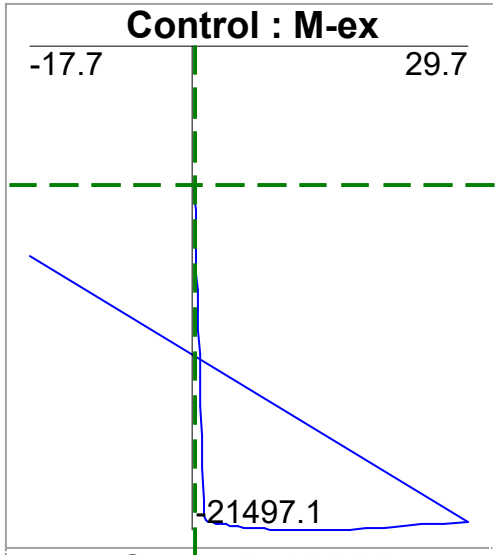


Response-2000 v 1.0.5

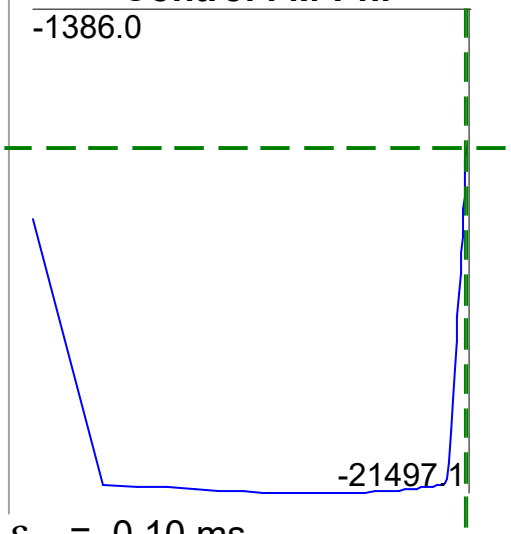
Proposed/Serviceability III/Pier

2021/11/25 - 11:34 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.10 ms

ϕ = -14.01 rad/10^6 in

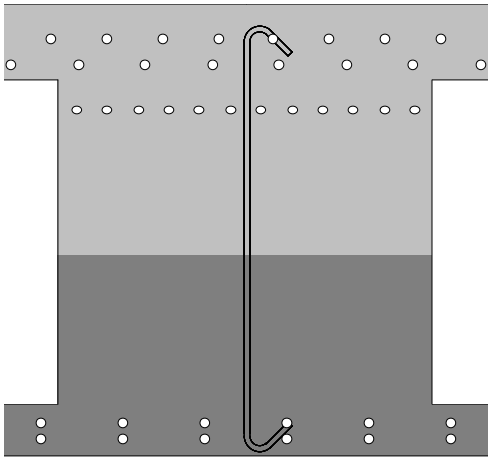
γ_{xy}(avg) = 0.00 ms

Axial Load = -1615.6 kips

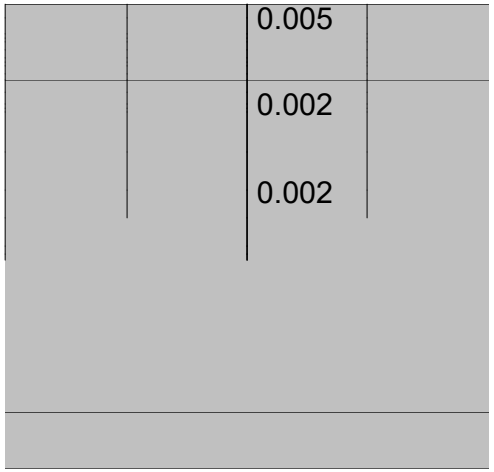
Moment:= -6160.6 ft-kips

Shear = 0.0 kips

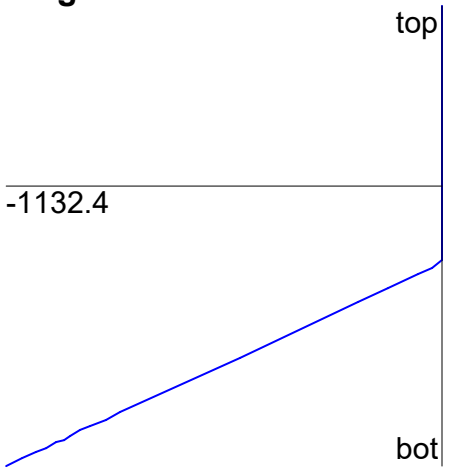
Cross Section



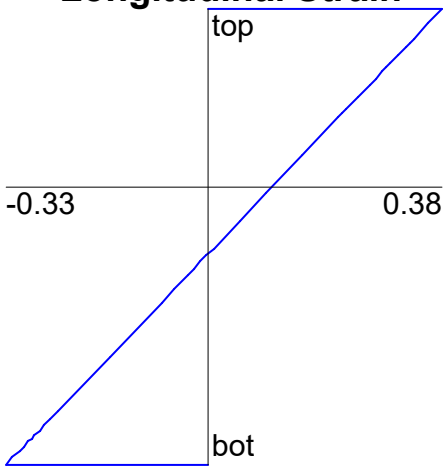
Crack Diagram



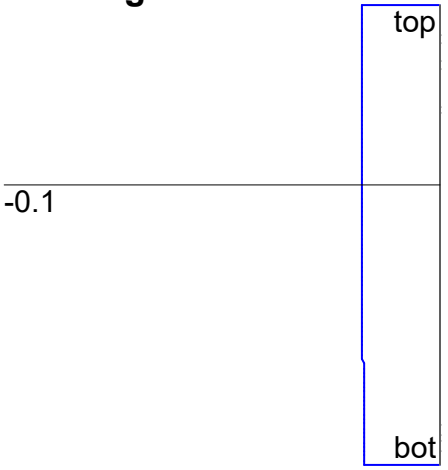
Longitudinal Concrete Stress



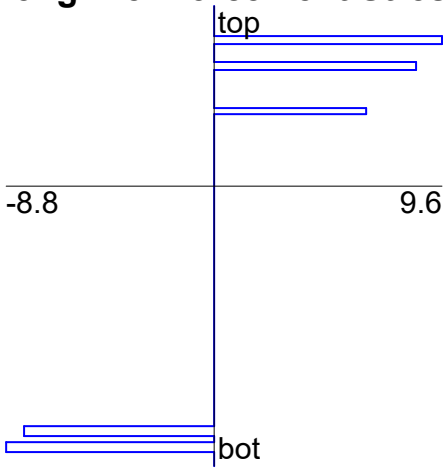
Longitudinal Strain



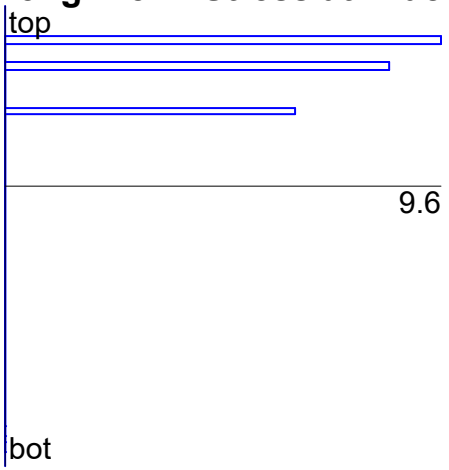
Shrinkage & Thermal Strain



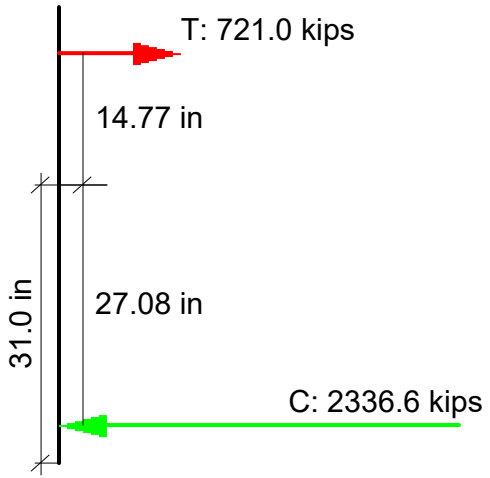
Long. Reinforcement Stress



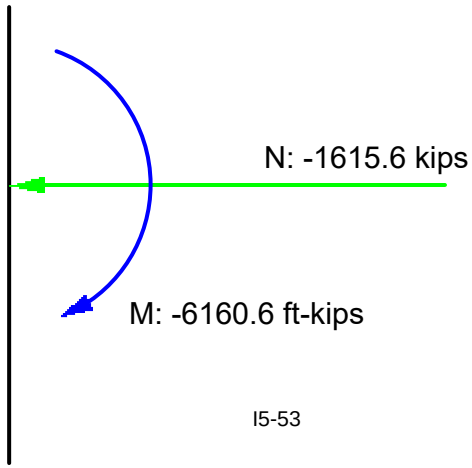
Long. Reinf Stress at Crack



Internal Forces



N+M

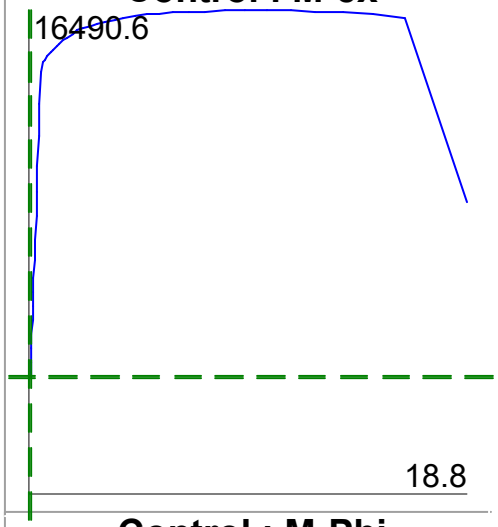


Response-2000 v 1.0.5

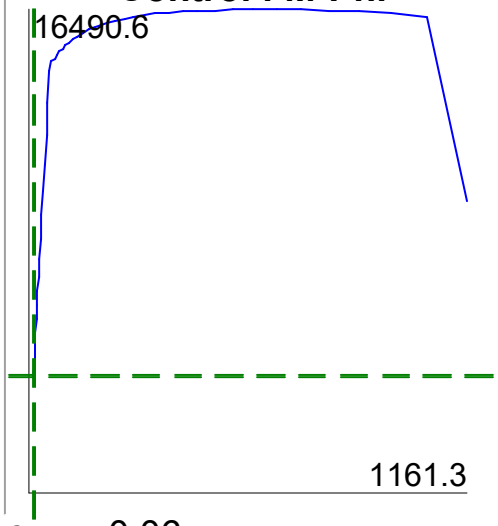
Proposed/Unfactored DC+PT/West Span

2021/11/25 - 11:28 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.06 ms

ϕ = 11.93 rad/10^6 in

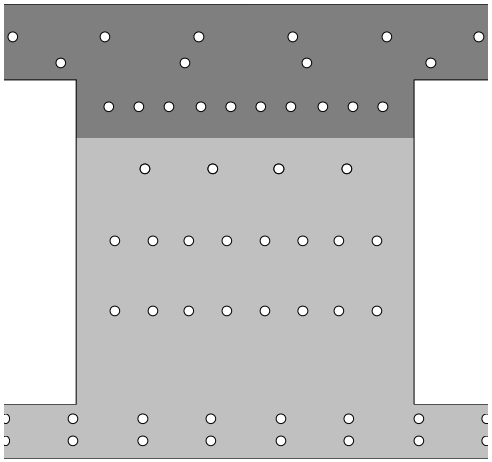
γ_{xy}(avg) = 0.00 ms

Axial Load = -1168.7 kips

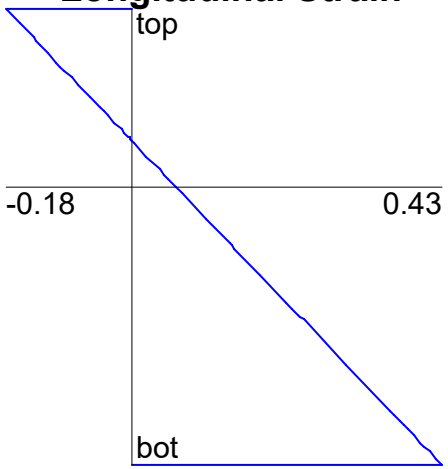
Moment:= 3987.9 ft-kips

Shear = 0.0 kips

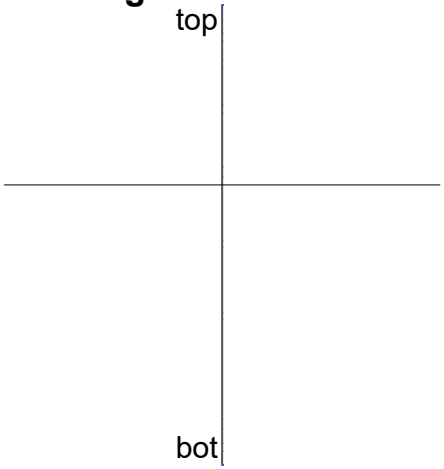
Cross Section



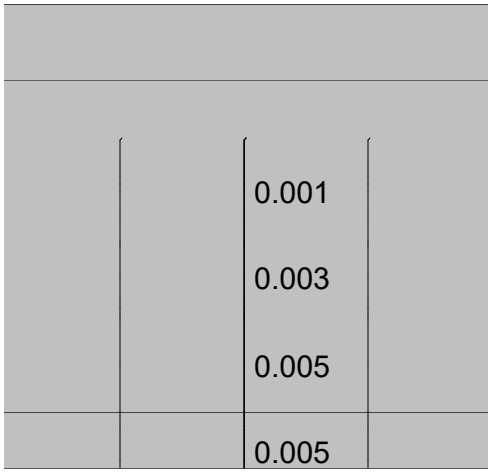
Longitudinal Strain



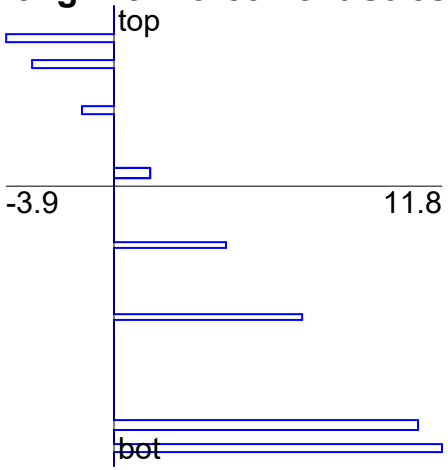
Shrinkage & Thermal Strain



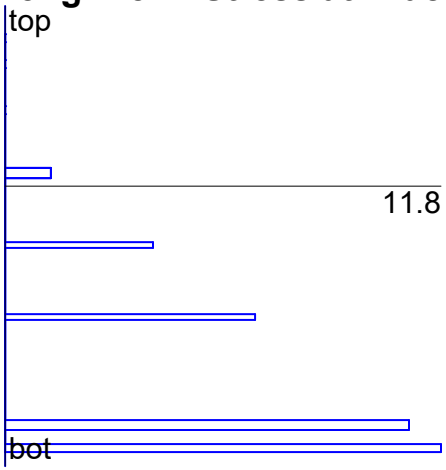
Crack Diagram



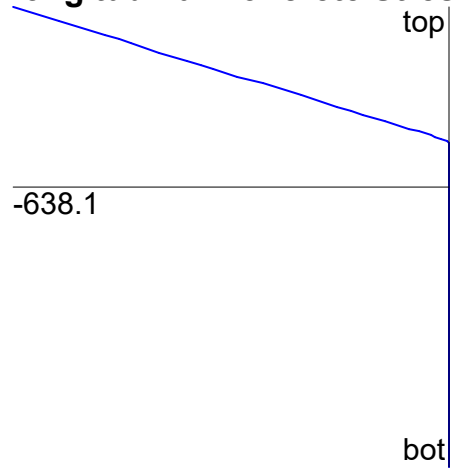
Long. Reinforcement Stress



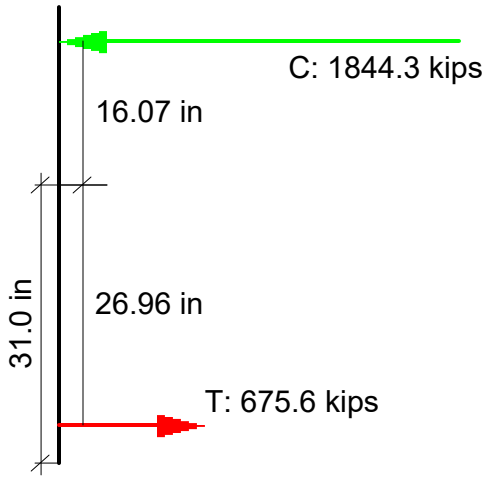
Long. Reinf Stress at Crack



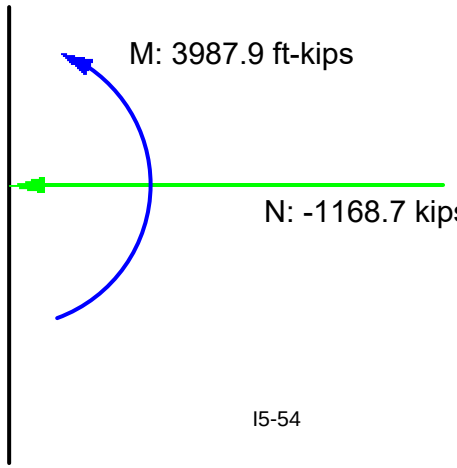
Longitudinal Concrete Stress



Internal Forces



N+M

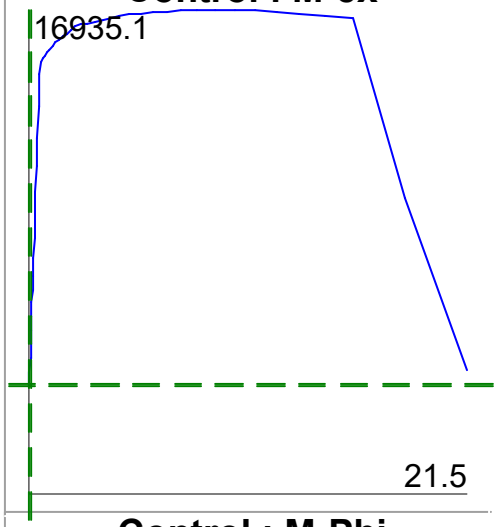


Response-2000 v 1.0.5

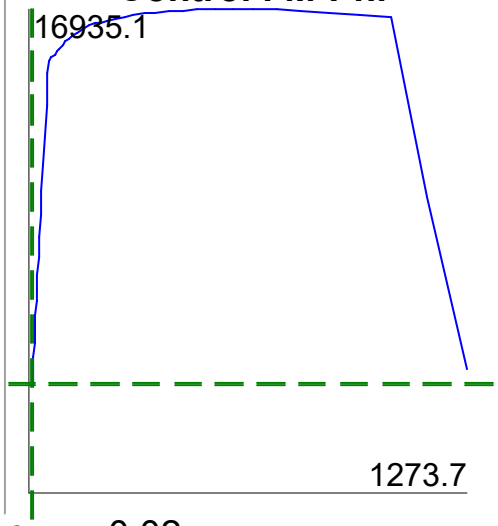
Proposed/Serviceability III/West Span

2021/11/25 - 11:26 am

Control : M-ex



Control : M-Phi



ε_{x0} = 0.02 ms

ϕ = 9.90 rad/10^6 in

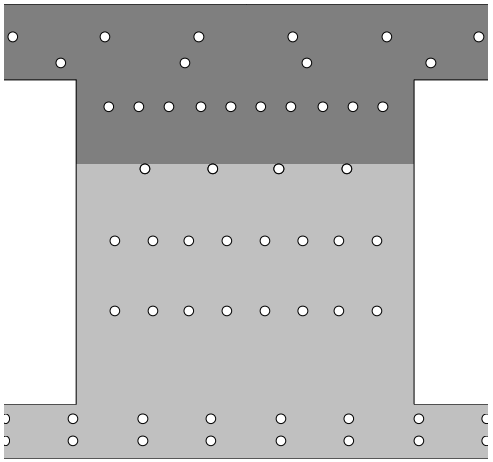
γ_{xy}(avg) = 0.00 ms

Axial Load = -1506.2 kips

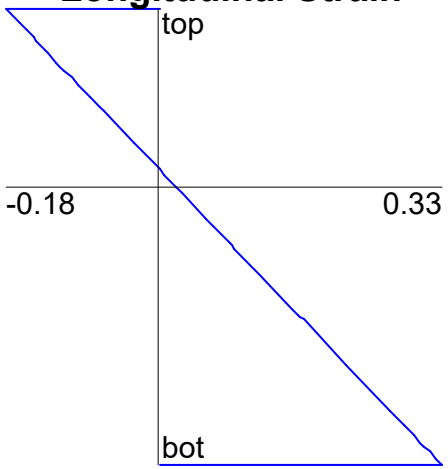
Moment:= 3785.7 ft-kips

Shear = 0.0 kips

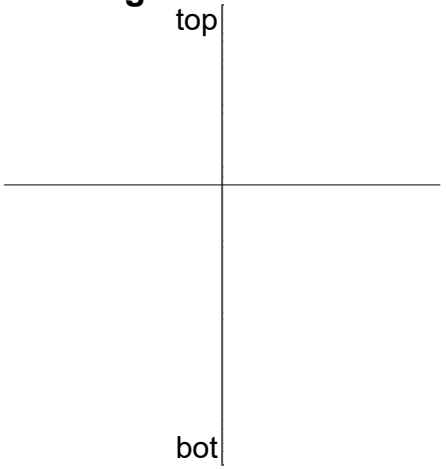
Cross Section



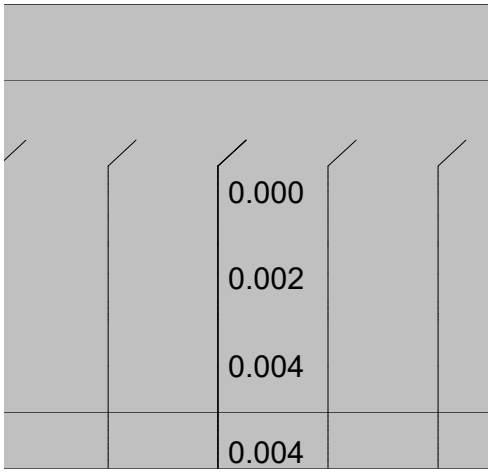
Longitudinal Strain



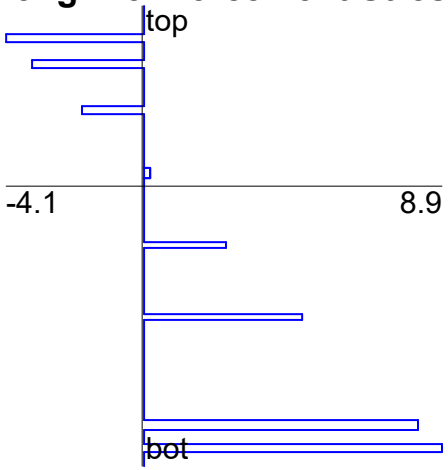
Shrinkage & Thermal Strain



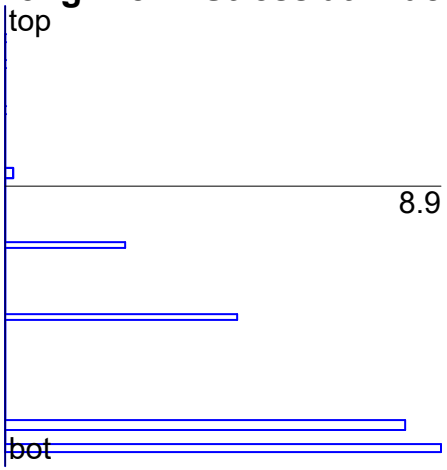
Crack Diagram



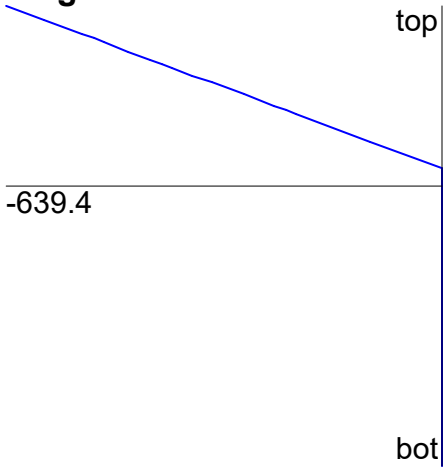
Long. Reinforcement Stress



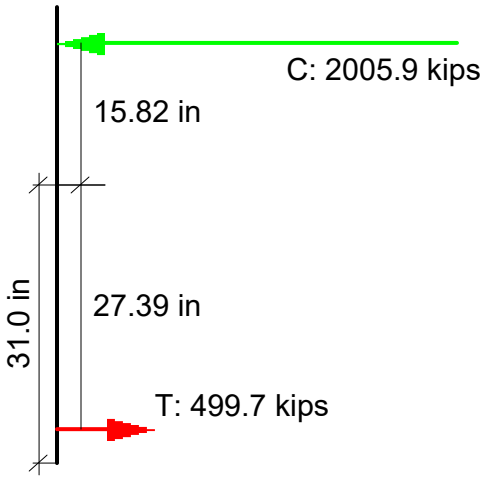
Long. Reinf Stress at Crack



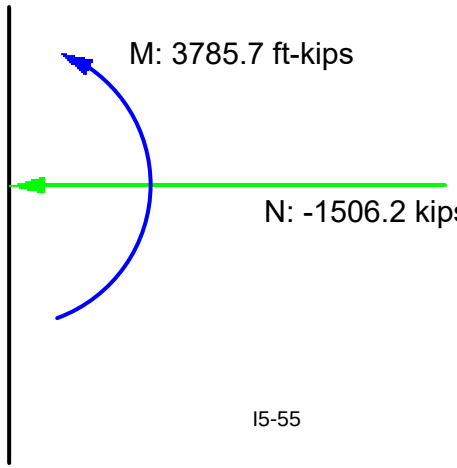
Longitudinal Concrete Stress



Internal Forces



N+M

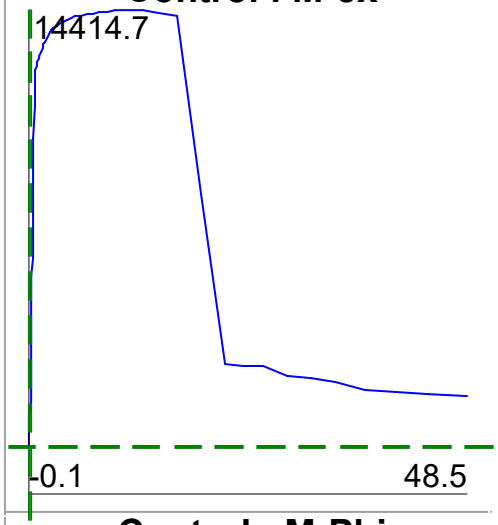


Response-2000 v 1.0.5

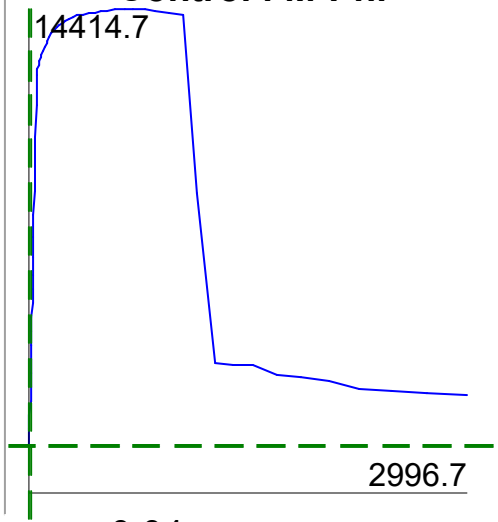
Proposed/Unfactored DC+PT/Interface

2021/11/25 - 11:31 am

Control : M-ex



Control : M-Phi



ε_{x0} = -0.04 ms

ϕ = 2.00 rad/10^6 in

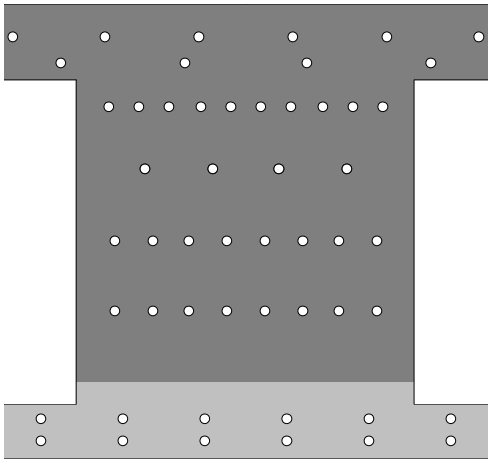
γ_{xy}(avg) = 0.00 ms

Axial Load = -1159.9 kips

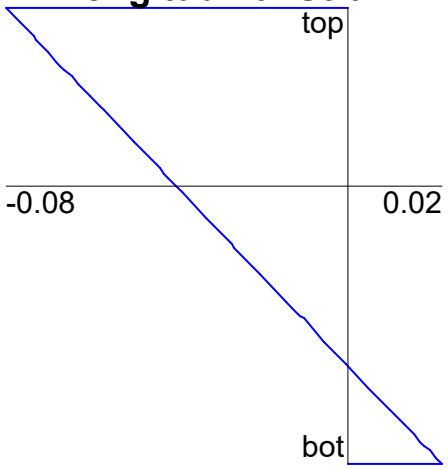
Moment:= 1419.1 ft-kips

Shear = 0.0 kips

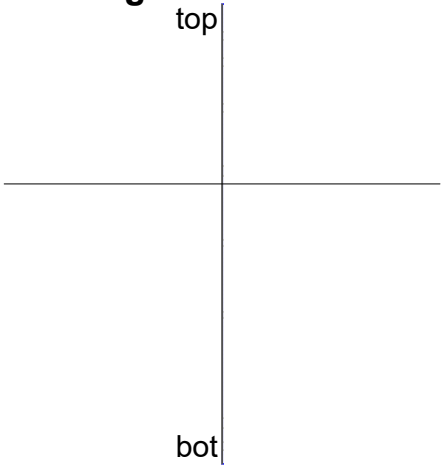
Cross Section



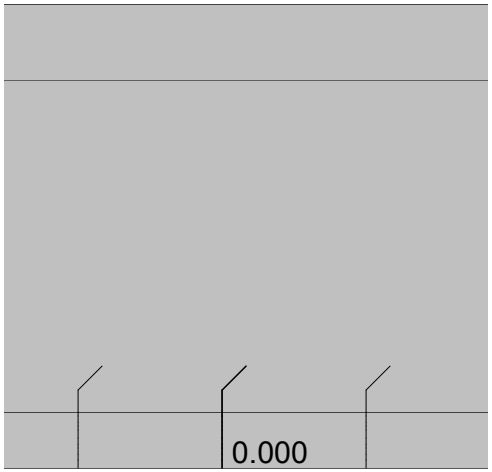
Longitudinal Strain



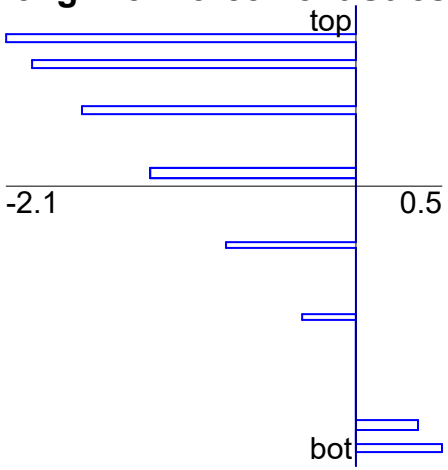
Shrinkage & Thermal Strain



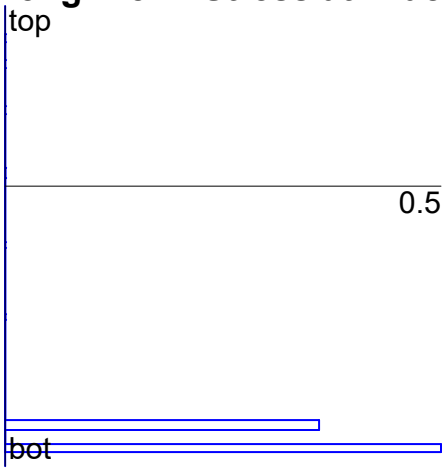
Crack Diagram



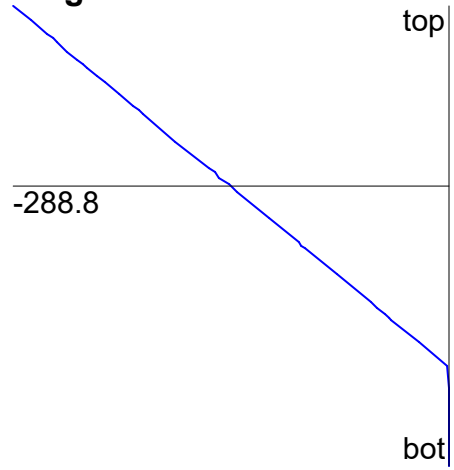
Long. Reinforcement Stress



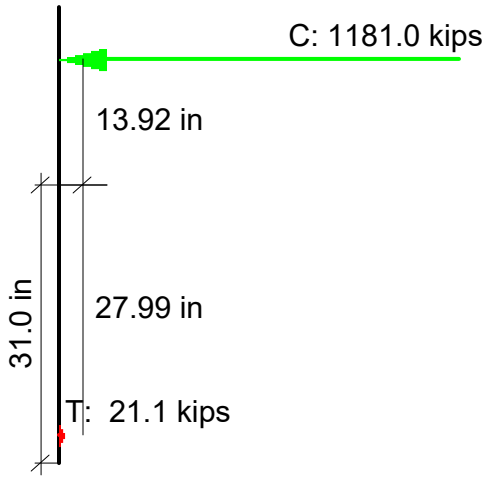
Long. Reinf Stress at Crack



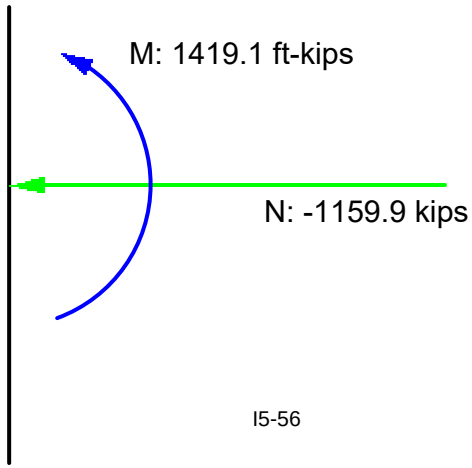
Longitudinal Concrete Stress



Internal Forces



N+M

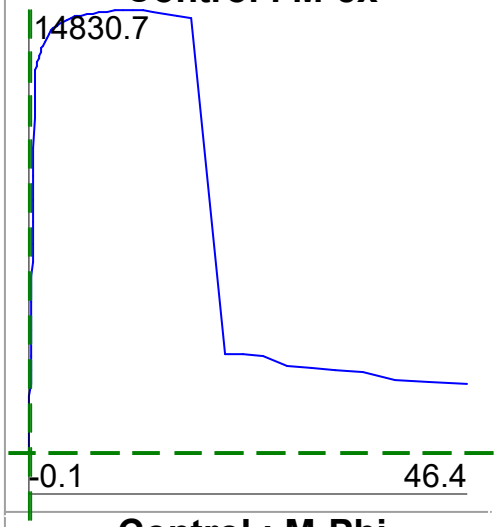


Response-2000 v 1.0.5

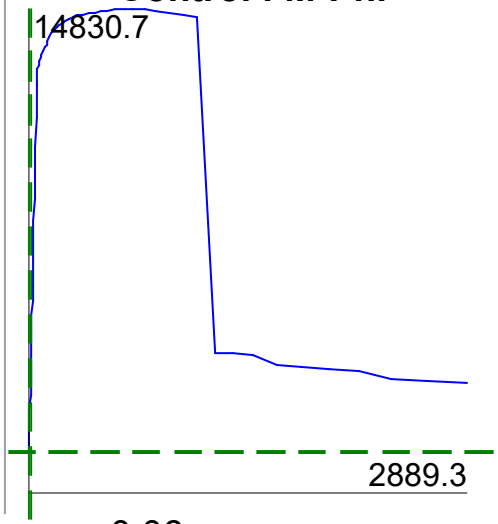
Proposed/Serviceability III/Interface

2021/11/25 - 11:30 am

Control : M-ex



Control : M-Phi



ε_{x0} = -0.06 ms

ϕ = 1.58 rad/10^6 in

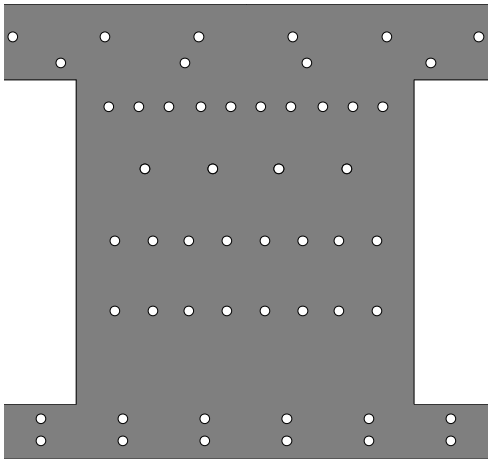
γ_{xy}(avg) = 0.00 ms

Axial Load = -1475.6 kips

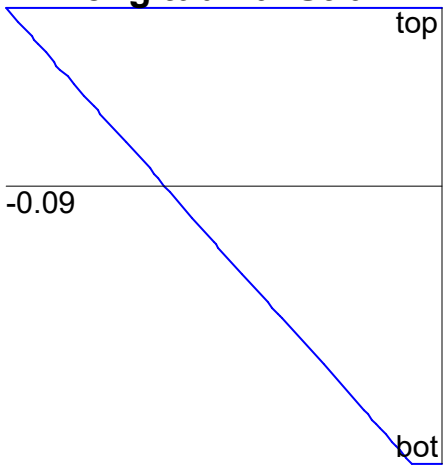
Moment:= 1277.9 ft-kips

Shear = 0.0 kips

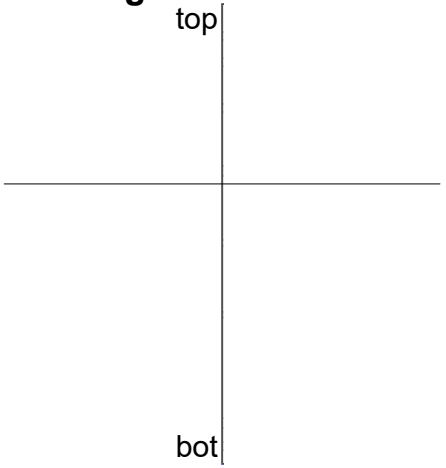
Cross Section



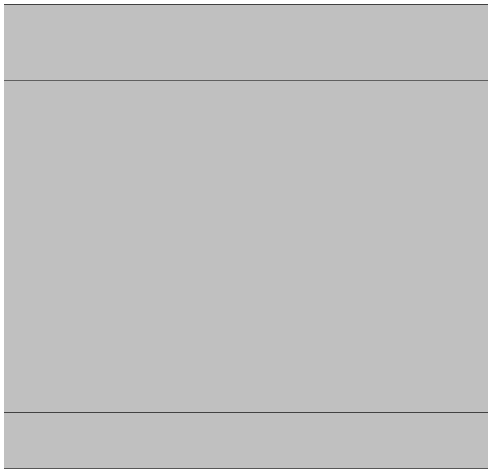
Longitudinal Strain



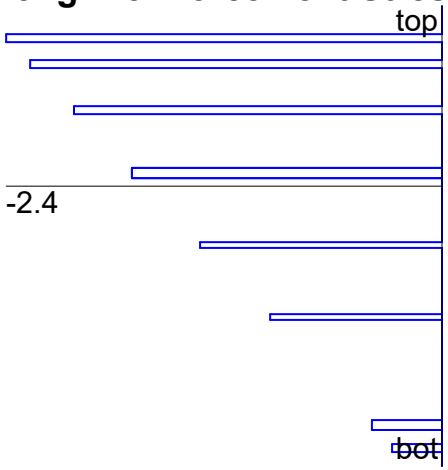
Shrinkage & Thermal Strain



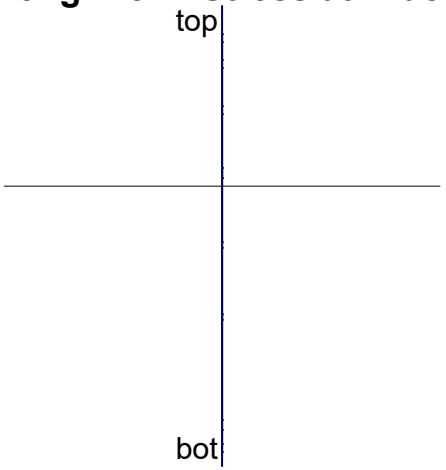
Crack Diagram



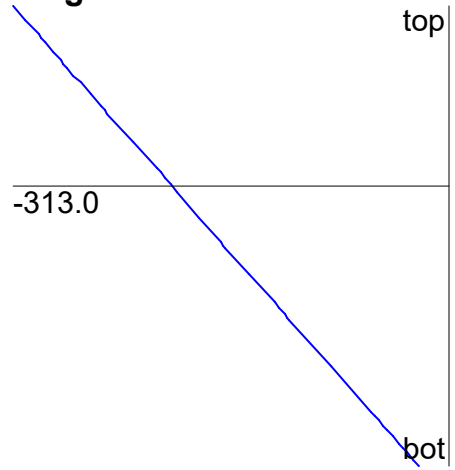
Long. Reinforcement Stress



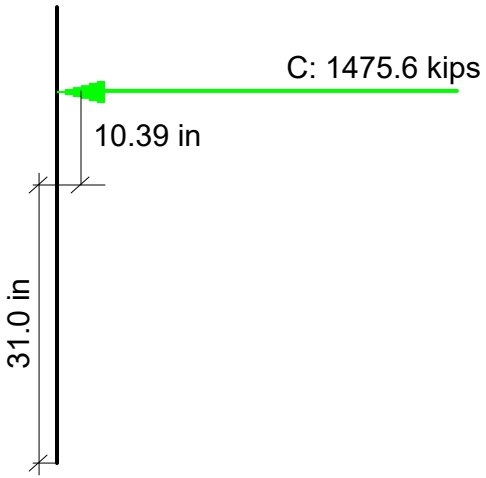
Long. Reinf Stress at Crack



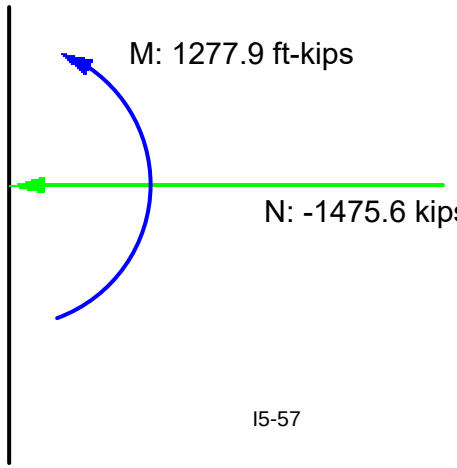
Longitudinal Concrete Stress



Internal Forces



N+M



COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

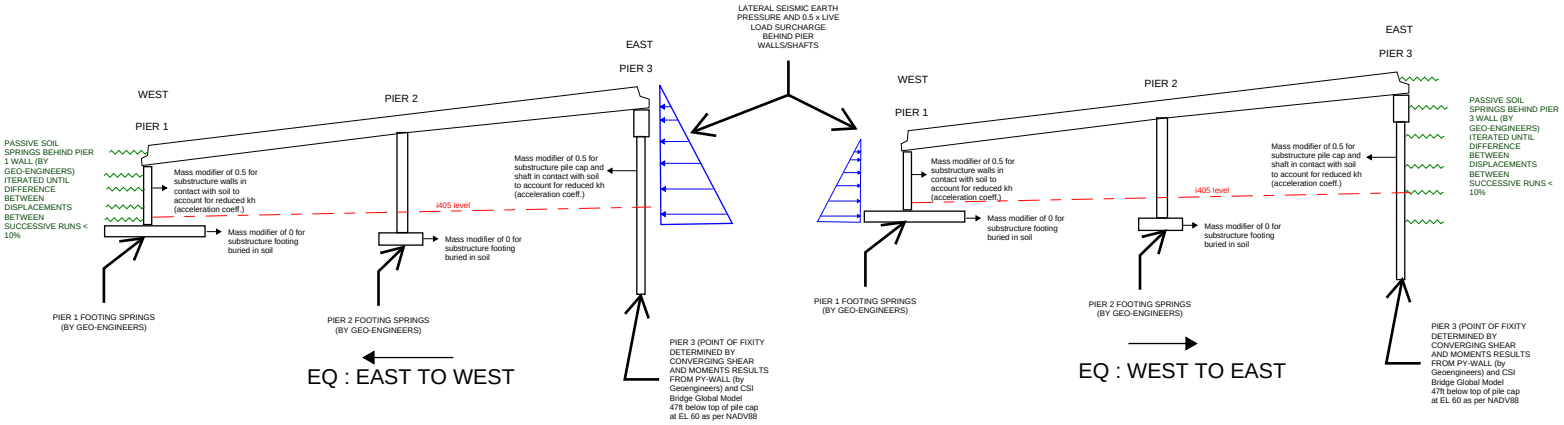
CONT: **A207833**

Book: I6 CALC BOOK COMPLETION DATE: **2021 Dec 05**

Title: Pier 1 (West Abutment)

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I6-0		I6-0	Calc Register (this sheet)	N/A	N/A	
2	I6-1	-	I6-14	Seismic Analysis Results	RSGR	MWBM	
3	I6-15	-	I6-27	Pier 1 Footing Sizing	RSGR	MWBM	
4	I6-28	-	I6-30	Pier 1 : Existing Wall Structural Check	RSGR	MWBM	
6	I6-31	-	I6-38	Pier 1 Wall reinforcement	RSGR	MWBM	
7	I6-39	-	I6-46	Pier 1 Footing reinforcement	RSGR	MWBM	
8	I6-47	-	I6-51	Moment Curvature Analysis	RSGR	MWBM	
9	I6-52	-	I6-56	Pier 1 - Top Shear Key Design (EXT CASE)	RSGR	LDLI	

1. The seismic analysis of the bridge is studied in two direction separately, for earthquake (EQ) in longitudinal direction East to West and West to East. 100% of passive springs behind pier 1 in East to West direction and 100% of passive springs behind Pier 3 in other direction. In both cases, the bridge's response to transverse and vertical EQ is the same between the two models. The following sketches illustrate the boundary conditions of the two directions analyzed:



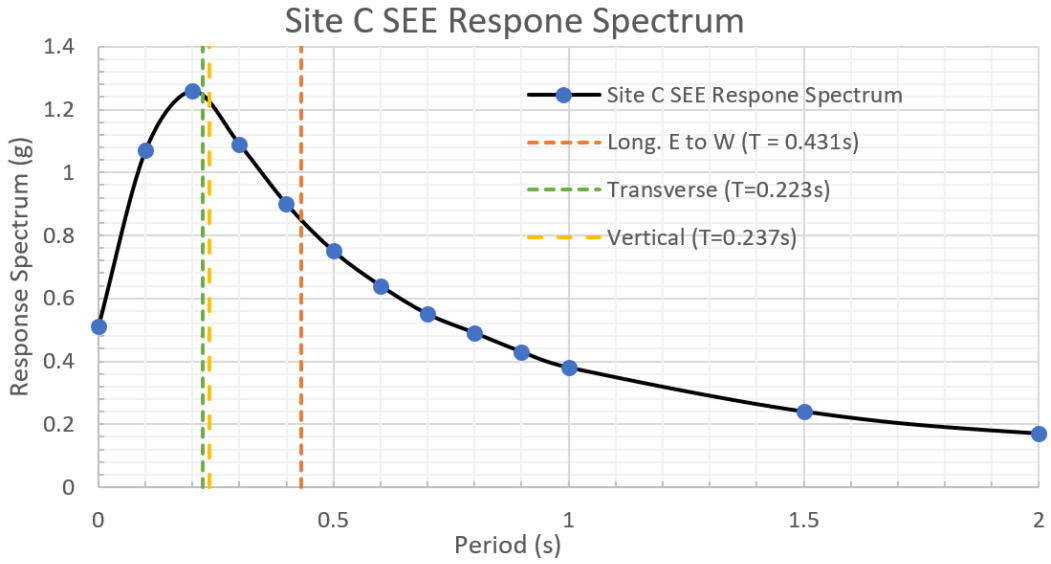
2. The following SEE Response Spectrum is input into CSI Bridge:

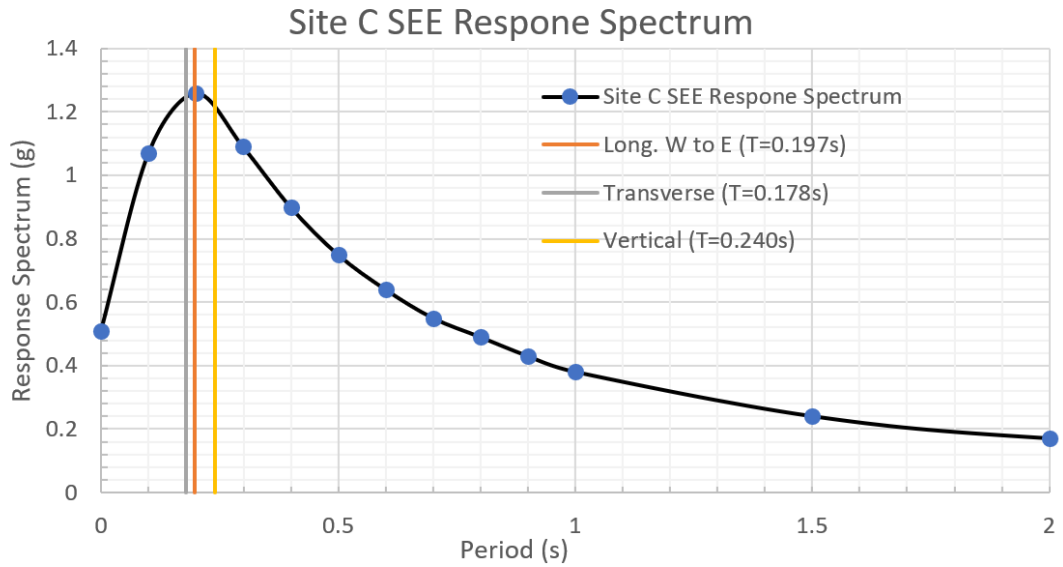
Period (s)	Hart Crowser PSHA Site-Specific (V_{s30} = 1,850 ft/s) 975-year Response Spectrum (g)[1,2] (Geometric Mean)	Basin Amplification Factor[3]	Composite Peak Directional Scaling Factors Weighted by Seattle Fault Contribution[4]	Calculated SEE Response Spectrum with Maximum Component Factor (g)	WSDOT BDM Code-Based SEE Response Spectrum for Site Class C (g)	Ratio of Calculated SEE Response Spectrum to Code-Based SEE Spectrum	**Recommended SEE Response Spectrum (g)5**
0.01	0.48	1.00	1.05	0.51	0.57	0.89	**0.51**
0.1	1.02	1.00	1.05	1.07	1.18	0.91	**1.07**
0.2	1.19	1.00	1.05	1.26	1.18 Sds	1.06	**1.26**
0.3	1.03	1.00	1.06	1.09	1.18	0.92	**1.09**
0.4	0.85	1.00	1.06	0.90	1.06	0.85	**0.90**
0.5	0.71	1.00	1.06	0.75	0.85	0.89	**0.75**
0.6	0.60	1.00	1.07	0.64	0.71	0.90	**0.64**
0.7	0.52	1.00	1.07	0.55	0.60	0.91	**0.55**
0.8	0.45	1.00	1.07	0.49	0.53	0.92	**0.49**
0.9	0.40	1.00	1.07	0.43	0.47	0.91	**0.43**
1	0.36	1.00	1.07	0.38	0.42 Sd1	0.91	**0.38**
1.5	0.23	1.00	1.06	0.24	0.28	0.85	**0.24**
2	0.16	1.00	1.06	0.17	0.21	0.81	**0.17**
3	0.093	1.00	1.06	0.10	0.14	0.70	**0.10**
4	0.060	1.00	1.06	0.064	0.11	0.60	**0.071**
5	0.041	1.00	1.06	0.044	0.085	0.52	**0.057**

1. Highlighted Values were obtained from Hart Crowser's PSHA using V_{S30} = 1,850 ft/s reference default basin depth terms ($Z_{1.0}$ = 0.164 km, $Z_{2.5}$ = 0.854 km)
2. Directivity effects included in PSHA per Bayless and Somerville (2013)
3. Basin amplification factors do not apply to this location.
4. Composite peak directional scaling factors developed by using factors taken from Shahi and Baker (2013) and applying period dependent weighting factors to them based on the Seattle Fault contribution to the total hazar
5. Recommended spectrum obtained by taking the maximum of the calculated basin-amplified response spectrum and 2/3 of the code-based spectrum for all periods.

For the spectrum in vertical direction, 2/3 of the design spectrum is used as per section 2.7 of FWHA (Seismic Retrofitting Manual for Highway Structures) for T,vertical = 0.2s

3. Bridge Fundamental Mode Time periods:





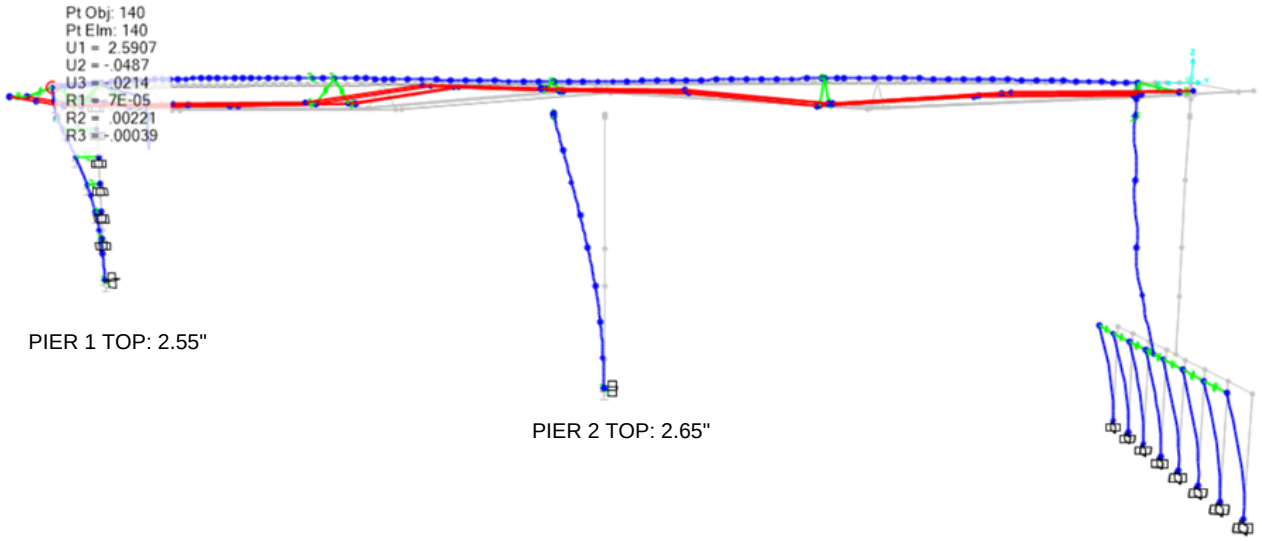
4. Bridge Classification; SDC - C

1-Sec period Spectral acceleration, S_{D1} =	*0.420*	sec	
Fa x 0.2-Sec period Spectral acceleration, S_{Ds} =	*1.180*	sec	
Seismic Design Category, SDC=	*C*		*AASHTO Seismic Table 3.5-1*

5. Seismic Behavior - Pier 1 wall is designed to exhibit in-elastic response at SEE level EQ (About Pier wall weak axis) and Pier 1 foundation is designed to be capacity protected. Pier 2 and Pier 3 are checked against SEE demands and have enough capacity to resist SEE demands elastically with a load factor of 1.2.

6. Displacement Demands Output

Case 1 : Response Spectrum (EtoW) (100%L) + Lateral Earth Pressure Behind Pier 3 + 0.5 x Live Load Surcharge behind Pier 3

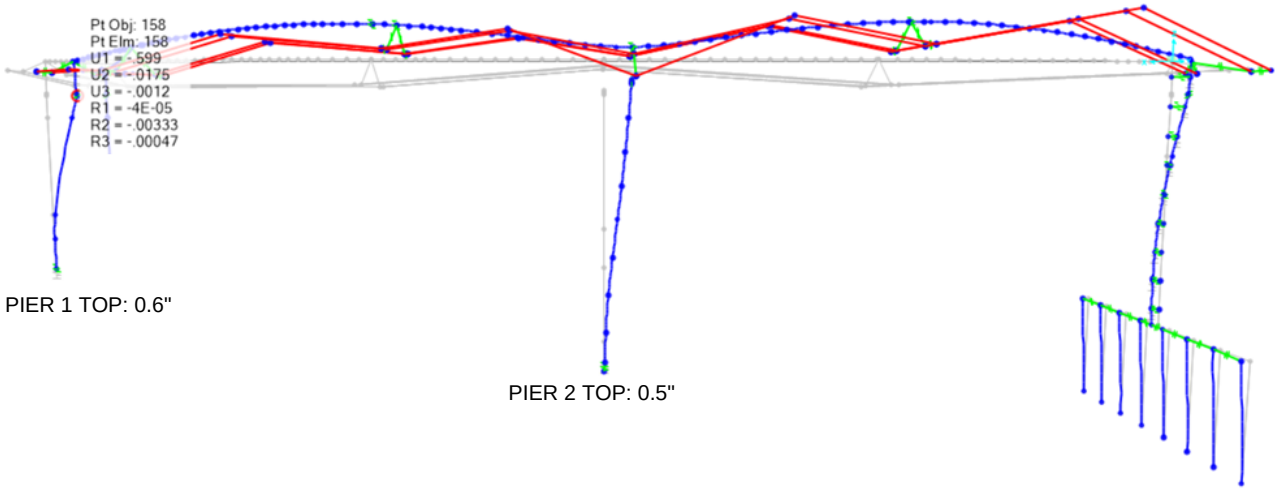


PIER 1 TOP: 2.55"

PIER 2 TOP: 2.65"

PIER 3 TOP: 2.75"

Case 2 : Response Spectrum (WtoE) (100%L) + Lateral Earth Pressure Behind Pier 1 + 0.5 x Live Load Surcharge behind Pier 1

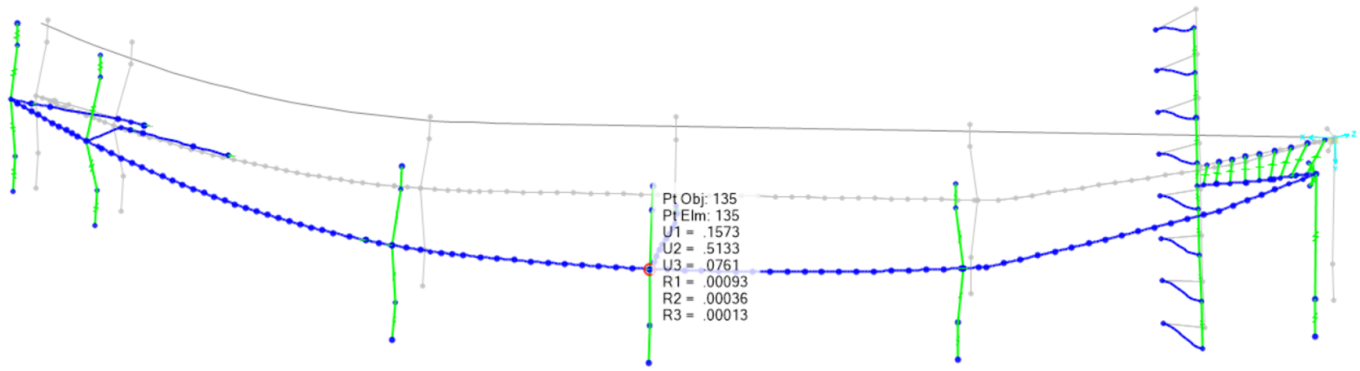


PIER 1 TOP: 0.6"

PIER 2 TOP: 0.5"

PIER 3 TOP: 0.47"

Case 3 : Response Spectrum (100%T) - 0.5" Displacement



7. Summary Longitudinal Displacement Demands at Pier 1

User Input=			
Maximum Dislacement EtoW EQ=	2.6	in	
Time period, T (EtoW)=	0.430	sec	CSI Bridge Output
T_s =	0.356	*sec*	*AASHTO Seismic Fig 3.4.1-1*
1-Sec period Spectral acceleration, S_{D1} =	*0.420*	sec	
Fa x 0.2-Sec period Spectral acceleration, S_{Ds} =	*1.180*	sec	
Seismic Design Category, SDC=	*C*		*AASHTO Seismic Table 3.5-1*
T=*	*0.44*	*sec*	*AASHTO Seismic Eq. 4.3.3-3*
μ_D =	*3*		*AASHTO Seismic Section 4.3.3*
Displacement Magnification factor, R_d =	*1.0*		*AASHTO Seismic Eq. 4.3.3-2*
Maximum Magnified Dislacement EtoW EQ=	***2.6***	***in***	

User Input=			
Maximum Dislacement WtoE EQ=	0.5	in	
Time period, T (WtoE)=	0.200	sec	CSI Bridge Output
T_s =	0.356	*sec*	*AASHTO Seismic Fig 3.4.1-1*
1-Sec period Spectral acceleration, S_{D1} =	*0.420*	sec	
Fa x 0.2-Sec period Spectral acceleration, S_{Ds} =	*1.180*	sec	
Seismic Design Category, SDC=	*C*		*AASHTO Seismic Table 3.5-1*
T=*	*0.44*	*sec*	*AASHTO Seismic Eq. 4.3.3-3*
μ_D =	*3*		*AASHTO Seismic Section 4.3.3*
Displacement Magnification factor, R_d =	*1.8*		*AASHTO Seismic Eq. 4.3.3-2*
Maximum Magnified Dislacement WtoE EQ=	***0.9***	***in***	

8. Displacement capacity at Pier 1 is calculated by performing moment curvature analysis of Pier 1 wall section. Details are in pages 47 to 51 . (per WSDOT BDM 4.2.4)

Based on moment curvature analysis,

Elastic displacement capacity = 2.02"
Plastic displacement capacity = 10.97"
Total displacement demand = 13" > Maximum Displacement demand = 2.55"
Ductility Capacity = 6.44

Plastic displacement demand = 0.53"
Ductility demand = 1.26 < Ductility Capacity
Ductility demand = 1.26 < 5.0 AASHTO SEISMIC GUIDE eq. 4.9-3 (or WSDOT BDM Table 4.1-2)

9. P-Delta not included in model as checked against AASHTO SEISMIC GUIDE 4.11.5

P - Delta Capacity Requirement Check

AASHTO SEISMIC 4.11.5

LONGITUDINAL

	kips	ft	kips-ft	kips-ft	kips-ft
	Pdl	**Δ**	**Mp**	**P x Δ**	**0.25Mp**
Pier 1	900	0.21	16745	191	4186

unfactored DL at the base of Pier 1 wall

Max. Pier 1displacement demand = 2.55"

$P_{dl}\Delta_r \leq 0.25\ M_p$

Plastic Moment Demand Mp at Pier 1 wall base

Page 1

10. Check minimum lateral strength requirement for a ductile member (AASHTO SEISMIC GIUDE 8.7.1)

Min. Lateral Capacity Requirement Check for each column

AASHTO SEISMIC 8.7.1

LONGITUDINAL	Ptrib (kips)	Hh (ft)	Ds (ft)	Λ	Mne=Mp (k-ft)	k-ft
Pier 1	900	19.5	4.25	1	16745	1946

$$M_{ne} \geq 0.1 P_{trib}\left(\frac{H_h + 0.5D_s}{\Lambda}\right)$$

11. Effective Pier Section Properties input for each pier are summarized below:

Pier 1 Wall about its weak axis : Based on Moment-Curvature Analysis

I, Crack = 448959 in4
I, Gross = 1539886 in2

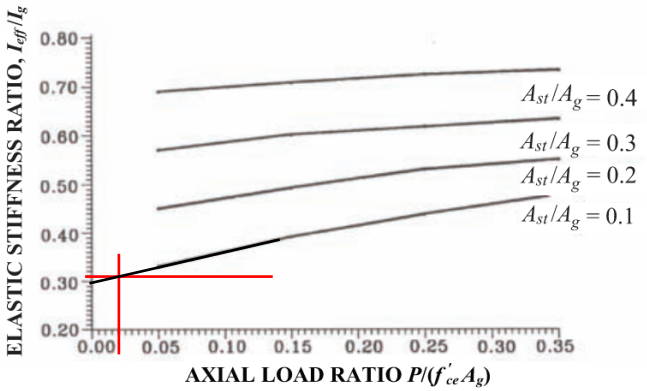
Stiffness modifier of 0.30 used for I value about weak axis and for shear stiffness
Torsional stiffness modifier of 0.2 used per AASHTO SEISIMC GUIDE eq. 5.6.5-1

Pier 2 Wall about its weak axis : Based on Moment-Curvature Analysis
I, Crack = 188101 in4
I, Gross = 648000 in2

Stiffness modifier of 0.30 used for I value about weak axis and for shear stiffness
Torsional stiffness modifier of 0.2 used per AASHTO SEISIMC GUIDE eq. 5.6.5-1

Pier 3 Shafts : Based Figure 5.6.2-1 AASHTO SEISMIC GUIDE

P = unfactored axial load per shaft ~ 130 kips; f'c = 4ksi, Ag = 1809 in2
Ast/Ag = 0.01

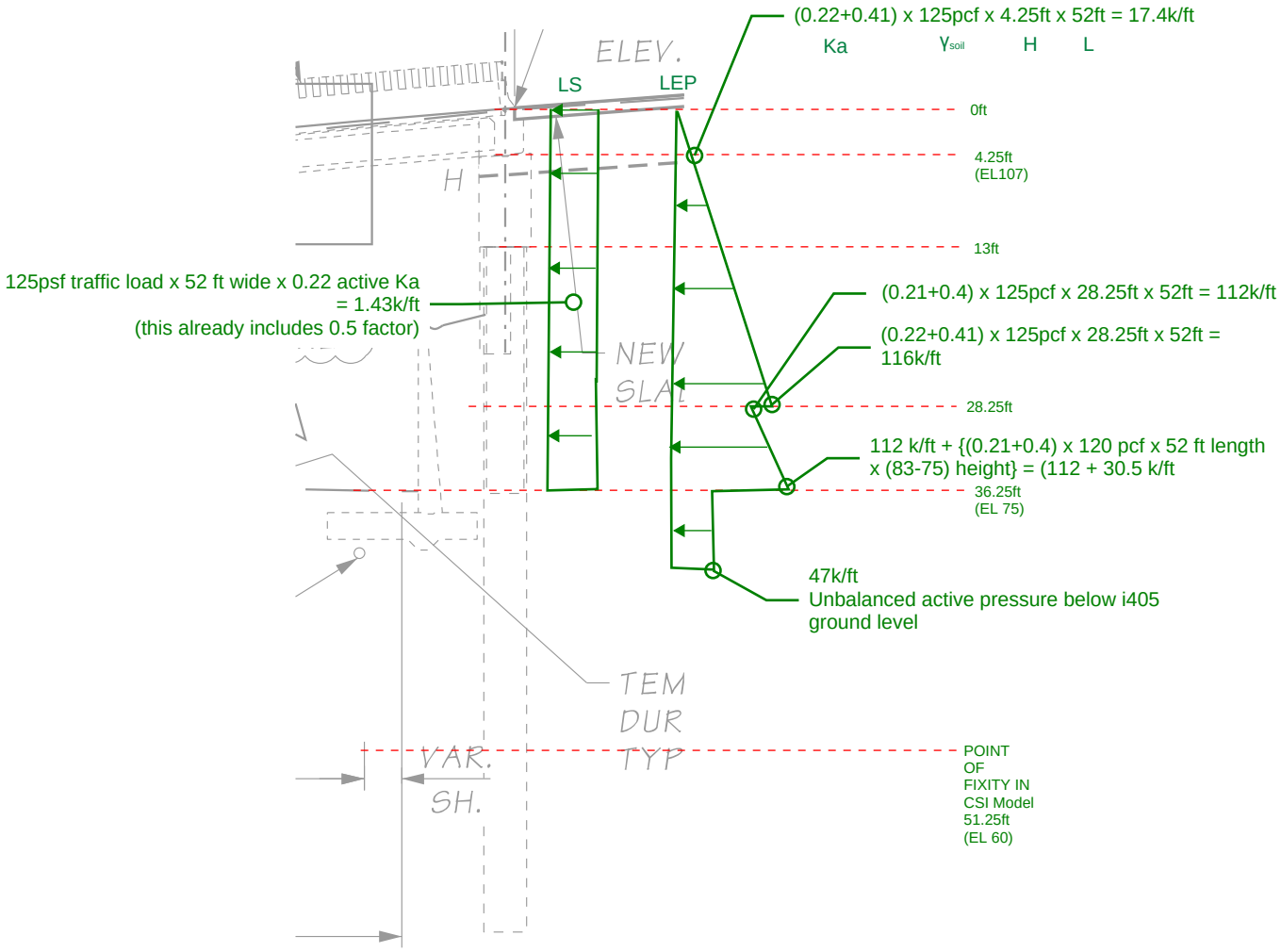


(b) Rectangular Sections

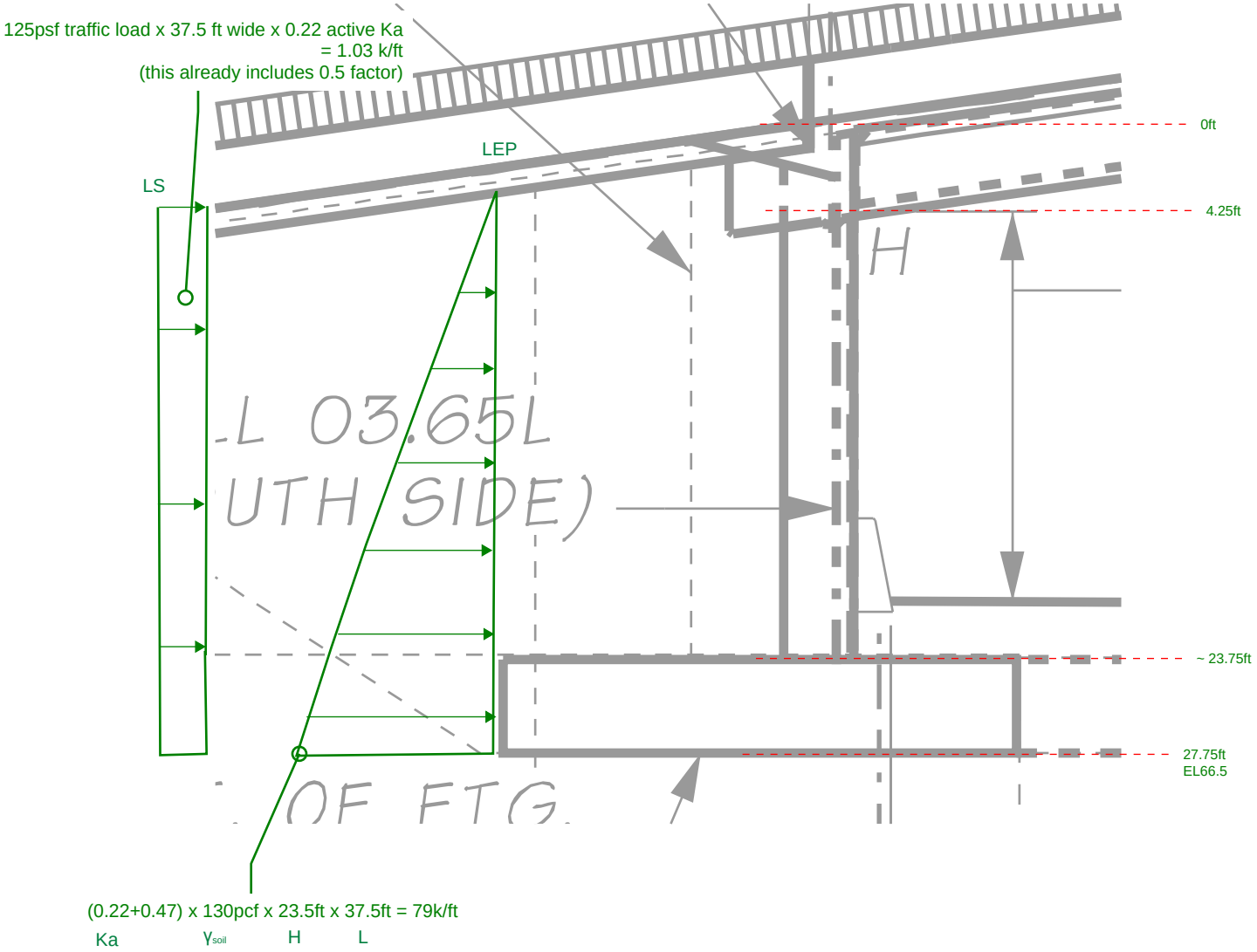
Stiffness modifier of ~0.30 used for I value about weak axis
Torsional stiffness modifier of 0.2 used per AASHTO SEISIMC GUIDE eq. 5.6.5-1

12. Pier Lateral Seismic Earth Pressure and Live Load Surcharge input into the CSI Models are summarized below:

Behind Pier 3 for EtoW EQ Analysis

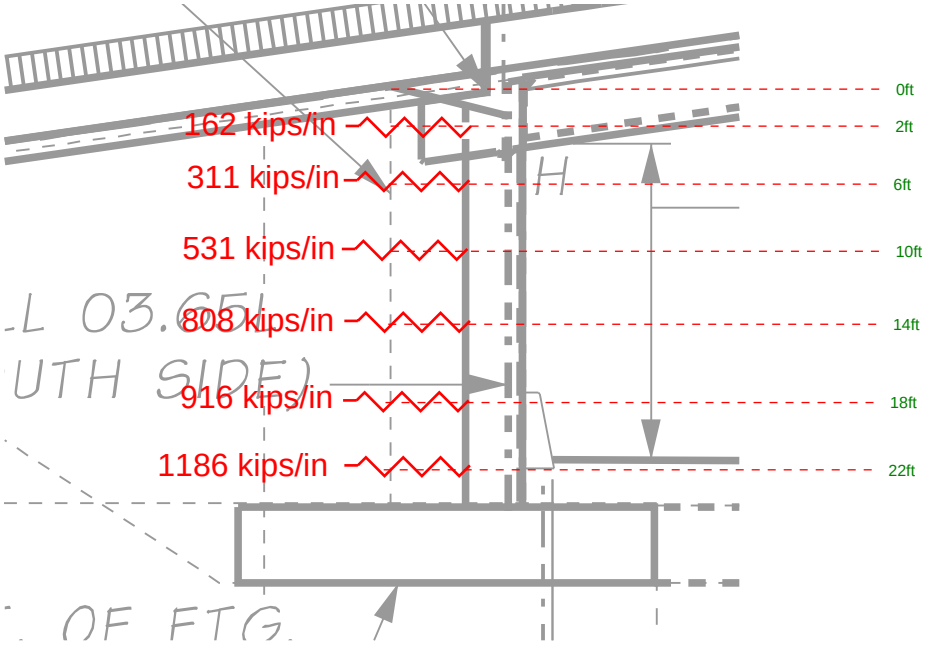


Behind Pier 1 for WtoE EQ Analysis



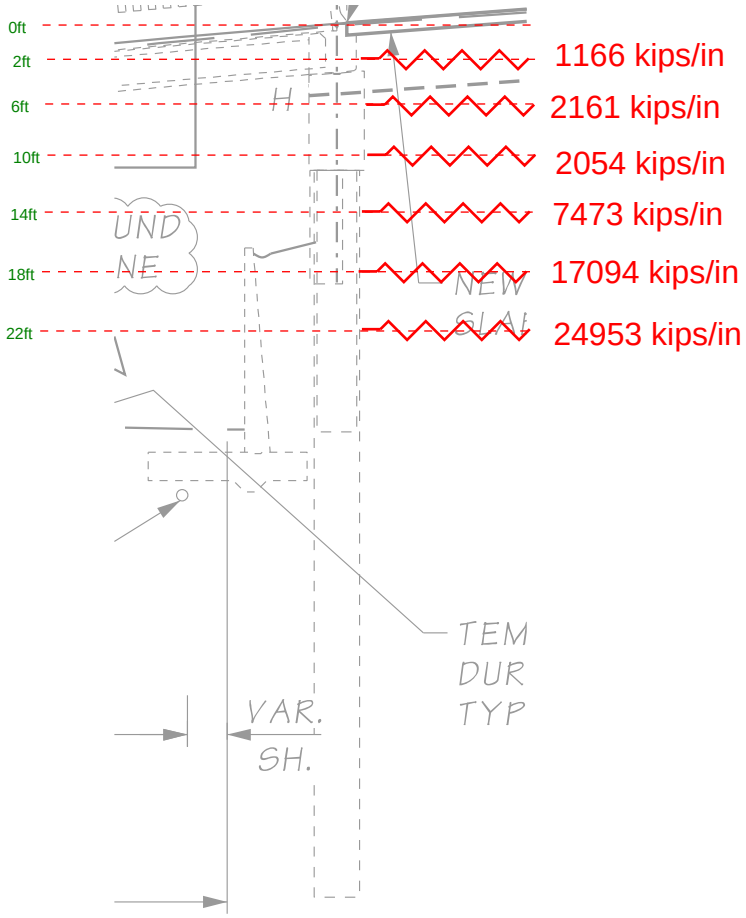
14. Passive springs used behind Pier 1 for EtoW EQ analysis

(From spring data by Geo-engineers for different displacements at different levels below soil. These are final values obtained after several iterations to converge displacements between successive runs of response spectrum case + LEP and LS loads behind Pier 3)



15. Passive springs used behind Pier 3 for WtoE EQ analysis

(From spring data by Geo-engineers for different displacements at different levels below soil. These are final values obtained after several iterations to converge displacements between successive runs of response spectrum case + LEP and LS loads behind Pier 1)



16. Pier 1 footing springs (by Geo-engineers)

Table 14: Summary of the 6-Degree-of-Freedom Structural Seismic Spring Constants Proposed Pier 1

Results Summary[1]							
Pier		$K_{x,emb}$	$K_{y,emb}$	$K_{z,emb}$	$K_{xx,emb}$	$K_{yy,emb}$	$K_{zz,emb}$
		kips/in	kips/in	kips/in	kip-in	kip-in	kip-in
SEE	Pier 1 – Existing (ESU 7A)	13,800	15,350	13,510	138,100,000	647,500,000	730,300,000
	Pier 1 – Proposed (ESU 1B)	5,647	5,870	5,793	160,200,000	320,600,000	357,000,000

17. Pier 2 footing springs (by Geo-engineers)

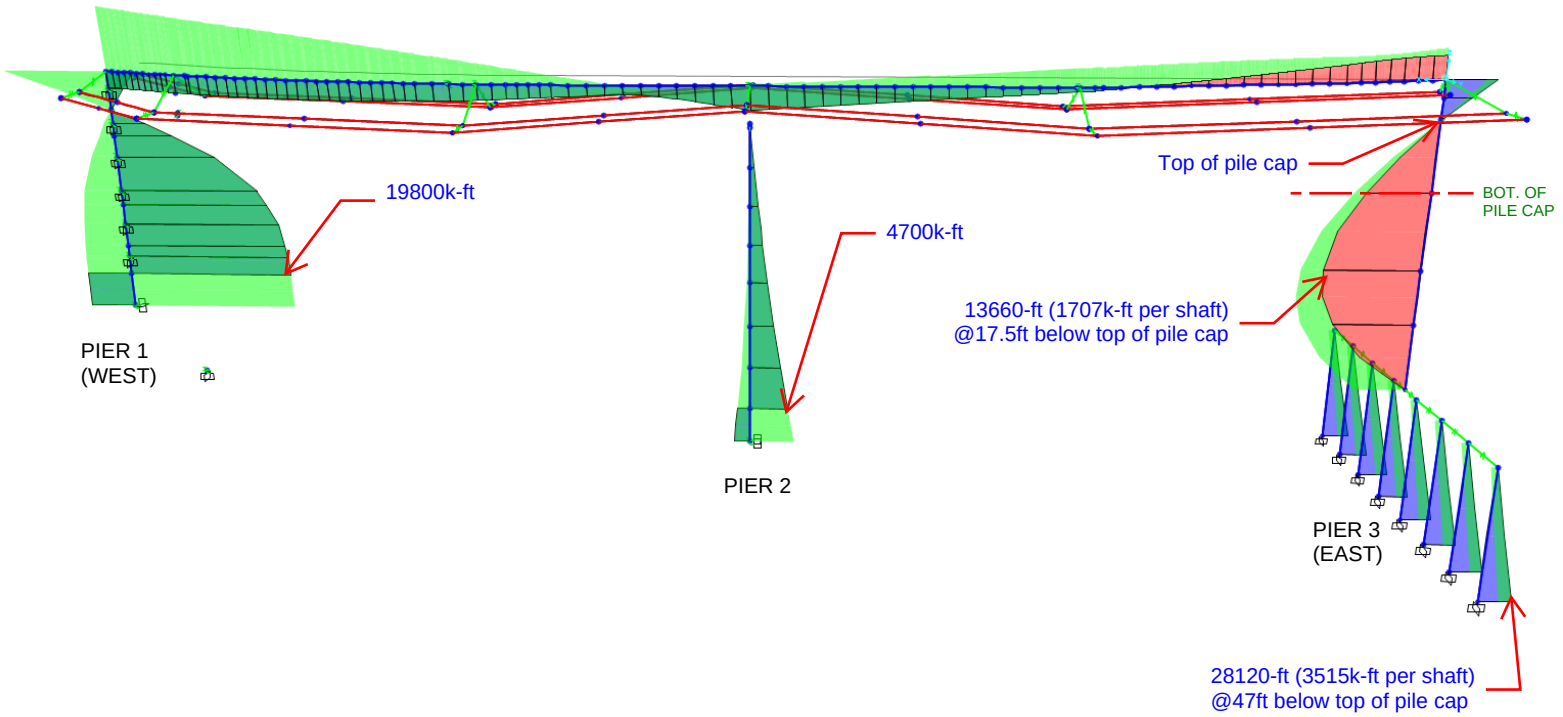
Table 17: Bridge 17.7 Pier 2 Summary of the 6-Degree-of-Freedom Structural Seismic Spring Constants

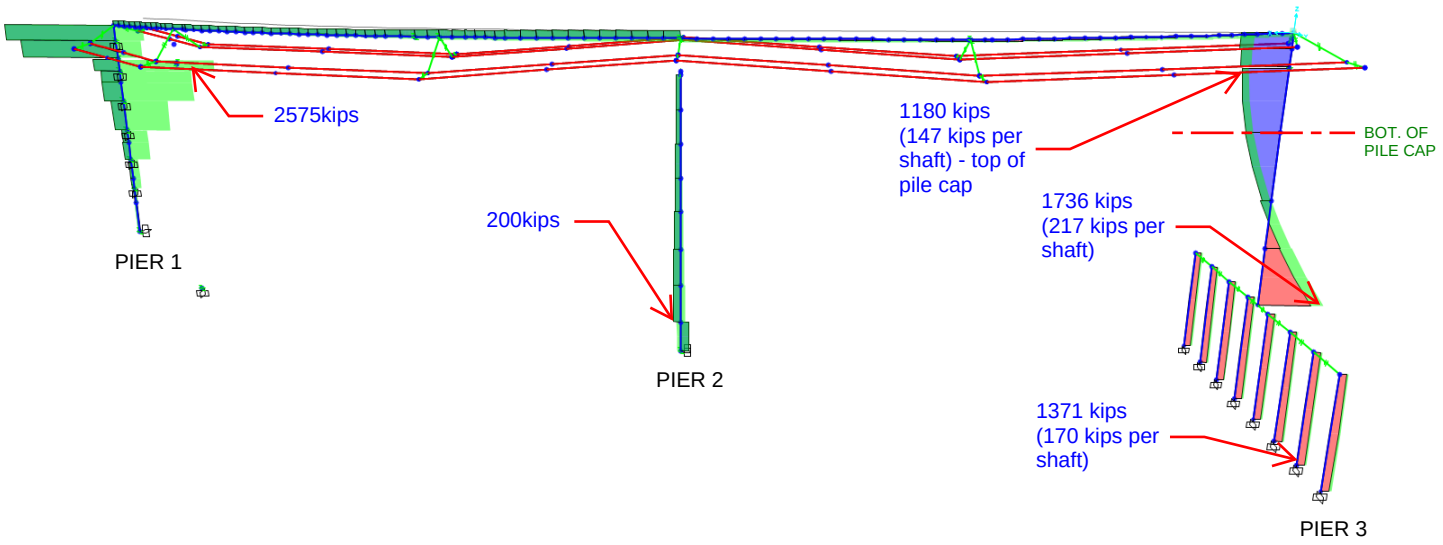
Results Summary[1]							
Pier		$K_{x,emb}$	$K_{y,emb}$	$K_{z,emb}$	$K_{xx,emb}$	$K_{yy,emb}$	$K_{zz,emb}$
		kips/in	kips/in	kips/in	kip-in	kip-in	kip-in
SEE	Pier 2 – Existing (ESU 2A)	11,380	12,070	10,080	119,100,000	278,500,000	333,400,000

Notes:
1. Spring values calculated using equations presented in Figure 4-4 (FEMA)

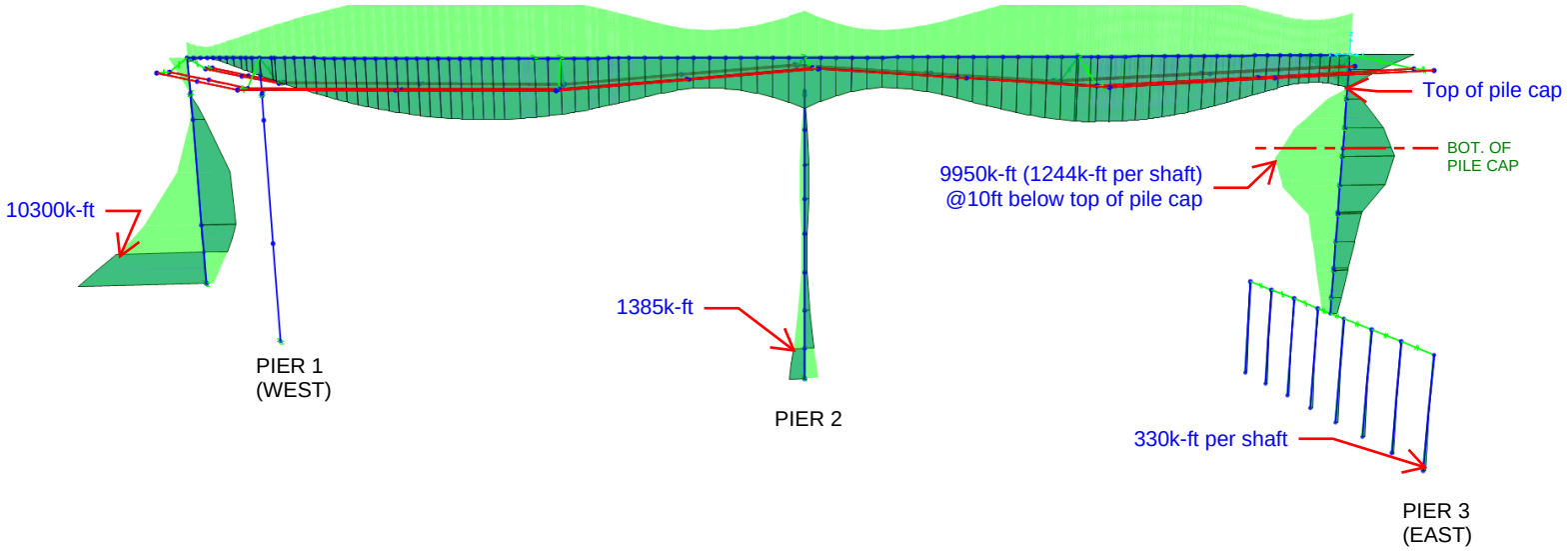
18. Moment and Shear output summarized for EtoW and WtoE analysis

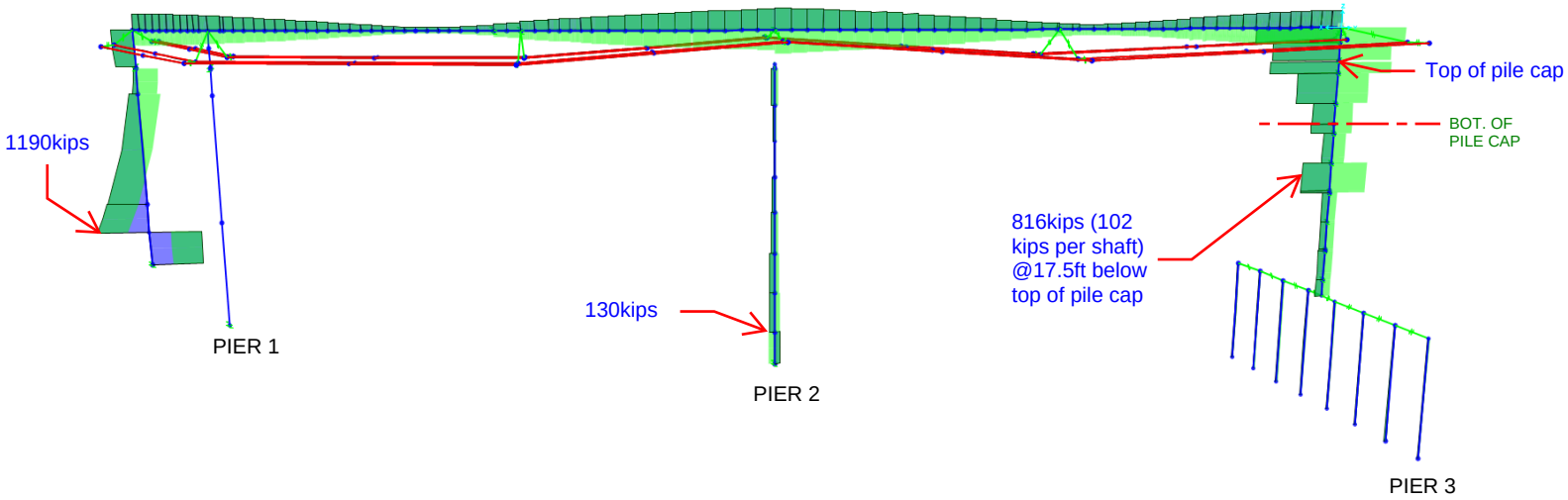
A) Response Spectrum Results EtoW + LEP & LS behind Pier 3





B) Response Spectrum Results WtoE + LEP & LS behind Pier 1





19. MODAL ANALYSIS OUTPUT (East to West Case)

TABLE: Modal Load Participation Ratios

OutputCase	ItemType	Item	Static	Dynamic
Text	Text	Text	Percent	Percent
MODAL	Acceleration	UX	100.00	92.12
MODAL	Acceleration	UY	100.00	96.24
MODAL	Acceleration	UZ	100.00	96.68

TABLE: Modal Participating Mass Ratios

OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
MODAL	Mode	1	1.175059	2.966E-07	2.246E-08	0.00015	2.966E-07	2.246E-08	0.00015	0.0000251	0.00016	1.377E-08	0.0000251	0.00016	1.377E-08
MODAL	Mode	2	1.175051	2.027E-09	0.00015	2.272E-08	2.987E-07	0.00015	0.00015	0.00005581	2.734E-08	0.00016	0.00008091	0.00016	0.00016
MODAL	Mode	3	1.175049	2.638E-11	7.833E-07	5.113E-09	2.987E-07	0.00015	0.00015	0.00006734	5.678E-09	0.00000105	0.00015	0.00016	0.00016
MODAL	Mode	4	1.175048	5.756E-20	3.761E-18	3.467E-20	2.987E-07	0.00015	0.00015	8.51E-18	3.314E-20	2.601E-16	0.00015	0.00016	0.00016
MODAL	Mode	5	1.175048	3.021E-19	6.009E-18	2.371E-20	2.987E-07	0.00015	0.00015	1.357E-17	7.577E-20	3.981E-16	0.00015	0.00016	0.00016
MODAL	Mode	6	1.175048	5.595E-19	1.054E-19	0	2.987E-07	0.00015	0.00015	2.642E-19	1.285E-20	7.489E-18	0.00015	0.00016	0.00016
MODAL	Mode	7	1.175048	1.199E-18	4.289E-20	5.701E-20	2.987E-07	0.00015	0.00015	2.687E-19	5.076E-20	1.07E-17	0.00015	0.00016	0.00016
MODAL	Mode	8	1.175048	0	1.037E-20	0	2.987E-07	0.00015	0.00015	0	0	1.774E-19	0.00015	0.00016	0.00016
MODAL	Mode	9	0.928367	2.187E-07	6.917E-07	0.00006839	5.174E-07	0.00015	0.00021	0.00003157	0.00005245	4.418E-07	0.00018	0.00021	0.00016
MODAL	Mode	10	0.928361	0.00000486	0.00006685	5.725E-07	0.000005377	0.00022	0.00022	0.0000128	8.267E-07	0.00005004	0.00019	0.00022	0.00021
MODAL	Mode	11	0.928357	1.956E-09	1.969E-08	9.369E-09	0.000005379	0.00022	0.00022	6.543E-07	6.603E-09	1.653E-08	0.00019	0.00022	0.00021
MODAL	Mode	12	0.928357	3.941E-12	2.587E-11	4.785E-13	0.000005379	0.00022	0.00022	5.445E-11	6.994E-13	1.725E-09	0.00019	0.00022	0.00021
MODAL	Mode	13	0.925138	8.123E-09	5.178E-07	0.00006798	0.000005387	0.00022	0.00028	0.00004504	0.00005287	2.885E-07	0.00024	0.00027	0.00021
MODAL	Mode	14	0.925131	0.000004762	0.00006693	3.519E-07	0.00001015	0.00028	0.00028	0.00001309	8.085E-08	0.00004253	0.00025	0.00027	0.00025
MODAL	Mode	15	0.925127	3.149E-09	4.757E-08	3.248E-08	0.00001015	0.00028	0.00028	0.000001446	2.696E-08	3.36E-08	0.00025	0.00027	0.00025
MODAL	Mode	16	0.925127	3.698E-12	1.469E-11	1.031E-12	0.00001015	0.00028	0.00028	1.154E-10	1.185E-12	4.301E-09	0.00025	0.00027	0.00025
MODAL	Mode	17	0.832485	0.000001067	4.039E-08	0.00013	0.00001122	0.00028	0.00041	0.00002259	0.00001673	2.172E-08	0.00028	0.00029	0.00025
MODAL	Mode	18	0.832481	1.854E-08	0.00014	4.037E-08	0.00001124	0.00042	0.00041	0.00006497	1.026E-08	0.00001347	0.00034	0.00029	0.00026
MODAL	Mode	19	0.832475	1.15E-10	0.000003276	1.141E-08	0.00001124	0.00042	0.00041	0.00004648	1.493E-09	2.609E-07	0.00039	0.00029	0.00026
MODAL	Mode	20	0.832474	0	0	0	0.00001124	0.00042	0.00041	0	1.077E-20	0	0.00039	0.00029	0.00026
MODAL	Mode	21	0.832474	0	0	4.33E-19	0.00001124	0.00042	0.00041	0	2.293E-19	4.691E-19	0.00039	0.00029	0.00026
MODAL	Mode	22	0.832474	0	0	0	0.00001124	0.00042	0.00041	0	0	0	0.00039	0.00029	0.00026
MODAL	Mode	23	0.832474	0	0	2.466E-20	0.00001124	0.00042	0.00041	0	3.629E-20	3.498E-20	0.00039	0.00029	0.00026
MODAL	Mode	24	0.832474	0	0	2.243E-20	0.00001124	0.00042	0.00041	4.683E-19	0	0	0.00039	0.00029	0.00026
MODAL	Mode	25	0.590637	0.00000497	0.000004146	0.00011	0.00001621	0.00043	0.00052	0.00003343	0.00001065	4.097E-07	0.00042	0.0003	0.00026
MODAL	Mode	26	0.590635	4.733E-07	0.00013	0.000003272	0.00001668	0.00056	0.00053	0.00005514	0.000000248	0.000008006	0.00048	0.0003	0.00027
MODAL	Mode	27	0.590626	6.449E-09	0.000003695	3.475E-08	0.00001669	0.00056	0.00053	0.00003649	2.206E-09	1.366E-07	0.00051	0.0003	0.00027
MODAL	Mode	28	0.590625	2.85E-14	3.305E-13	2.245E-14	0.00001669	0.00056	0.00053	4.467E-13	3.378E-15	4.657E-11	0.00051	0.0003	0.00027
MODAL	Mode	29	0.590625	4.902E-14	5.625E-13	3.869E-14	0.00001669	0.00056	0.00053	7.7E-13	5.028E-15	8.196E-11	0.00051	0.0003	0.00027
MODAL	Mode	30	0.590625	1.369E-15	1.685E-14	1.319E-15	0.00001669	0.00056	0.00053	2.112E-14	1.382E-16	2.498E-12	0.00051	0.0003	0.00027
MODAL	Mode	31	0.590625	4.424E-16	5.244E-15	3.703E-16	0.00001669	0.00056	0.00053	6.806E-15	4.552E-17	7.657E-13	0.00051	0.0003	0.00027
MODAL	Mode	32	0.590625	3.682E-19	3.762E-18	2.297E-19	0.00001669	0.00056	0.00053	6.953E-18	3.258E-20	6.347E-16	0.00051	0.0003	0.00027
MODAL	Mode	33	0.431385	0.80819	0.00045	0.00043	0.80821	0.00102	0.00096	0.00007917	0.00636	0.00071	0.00059	0.00666	0.00099
MODAL	Mode	34	0.237868	0.00237	0.05245	0.00015	0.81058	0.05347	0.00111	0.00955	0.26027	0.00272	0.01015	0.26693	0.0037
MODAL	Mode	35	0.223347	0.00108	0.71171	0.00168	0.81167	0.76517	0.00279	0.218	0.02111	0.01659	0.22815	0.28804	0.02029
MODAL	Mode	36	0.177162	0.00055	0.00278	0.61722	0.81221	0.76795	0.62001	0.01935	0.00044	0.00001306	0.24749	0.28848	0.0203
MODAL	Mode	37	0.133122	0.00023	0.08986	0.00019	0.81244	0.85781	0.62019	0.0000663	0.00005352	0.85082	0.24756	0.28853	0.87112
MODAL	Mode	38	0.09849	0.00478	0.00062	0.00037	0.81722	0.85843	0.62057	0.00043	0.00027	0.00099	0.24799	0.2888	0.87211
MODAL	Mode	39	0.095433	0.04586	0.00252	0.00142	0.86307	0.86095	0.62198	0.00118	0.02106	0.00148	0.24917	0.30986	0.87359
MODAL	Mode	40	0.087533	0.00033	0.00019	0.00097	0.86341	0.86115	0.62295	0.05686	0.05969	0.00024	0.30603	0.36955	0.87383
MODAL	Mode	41	0.082569	0.00258	0.05903	0.00003404	0.86599	0.92017	0.62298	0.0089	0.00141	0.02208	0.31493	0.37096	0.89591
MODAL	Mode	42	0.07341	0.00031	0.000005708	0.07956	0.8663	0.92018	0.70255	0.0003	0.14845	0.00000154	0.31523	0.51941	0.89591
MODAL	Mode	43	0.064781	5.332E-11	5.605E-08	2.338E-08	0.8663	0.92018	0.70255	8.143E-07	1.408E-07	1.394E-07	0.31523	0.51941	0.89591
MODAL	Mode	44	0.064781	4.287E-12	4.849E-11	1.012E-12	0.8663	0.92018	0.70255	3.039E-10	6.42E-12	7.118E-10	0.31523	0.51941	0.89591
MODAL	Mode	45	0.064776	1.667E-07	0.000003275	7.358E-07	0.8663	0.92018	0.70255	1.522E-08	0.000009327	9.614E-08	0.31523	0.51942	0.89591
MODAL	Mode	46	0.064773	1.662E-08	0.000003216	0.000009636	0.8663	0.92018	0.70256	0.00002323	0.00009532	0.000001088	0.31526	0.51951	0.89591
MODAL	Mode	47	0.063862	2.472E-10	2.776E-08	4.44E-09	0.8663	0.92018	0.70256	2.849E-07	1.884E-07	7.982E-08	0.31526	0.51951	0.89591
MODAL	Mode	48	0.063862	1.47E-12	2.846E-11	8.228E-13	0.8663	0.92018	0.70256	1.438E-10	1.91E-12	1.762E-10	0.31526	0.51951	0.89591
MODAL	Mode	49	0.063858	5.031E-09	0.000001156	0.000001672	0.8663	0.92019	0.70256	4.533E-08	0.00003241	0.000001021	0.31526	0.51955	0.89592
MODAL	Mode	50	0.063856	2.891E-07	0.000006092	0.000002379	0.8663	0.92019	0.70256	0.000007003	0.00006009	0.000001286	0.31526	0.51961	0.89592
MODAL	Mode	51	0.056154	0.000006955	0.00003184	0.06102	0.86631	0.92022	0.76358	0.00542	0.00006891	0.00003145	0.32069	0.51967	0.89595
MODAL	Mode	52	0.042394	0.00084	0.00011	0.00899	0.86715	0.92033	0.77257	0.00038	0.03172	1.701E-07	0.32107	0.55139	0.89595
MODAL	Mode	53	0.040301	0.00009227	0.00001735	1.257E-07	0.86724	0.92035	0.77257	0.0023	0.000001611	0.00355	0.32337	0.55139	0.89949
MODAL	Mode	54	0.040007	0.00113	0.00016	0.000002985	0.86837	0.92051	0.77258	0.02452	0.0000162	0.01031	0.34788	0.55141	0.90981
MODAL	Mode	55	0.037131	0.01583	0.00084	0.00027	0.88421	0.92135	0.77285	0.01025	0.00001673	7.497E-07	0.35813	0.55143	0.90981
MODAL	Mode	56	0.034954	0.00854	0.00502	0.00016	0.89275	0.92637	0.77301	0.05788	0.00094	0.00013	0.41602	0.55237	0.90994
MODAL	Mode	57	0.034928	0.00568	0.02255	0.00001261	0.89843	0.94892	0.77302	0.22589	0.00089	0.00025	0.64191	0.55326	0.91019
MODAL	Mode	58	0.033346	0.00211	0.00036	0.00008294	0.90054	0.94928	0.7731	0.00969	0.00022	0.000008243	0.6516	0.55348	0.9102
MODAL	Mode	59	0.029506	0.00134	0.000008876	0.01101	0.90188	0.94929	0.78412	0.00263	0.01782	0.00016	0.65423	0.57129	0.91035
MODAL	Mode	60	0.026645	0.00009534	0.00045	0.00008311	0.90197	0.94974	0.7842	0.05189	0.00005168	0.00111	0.70612	0.57134	0.91147
MODAL	Mode	61	0.024971	0.00571	0.00017	0.00004356	0.90768	0.94991	0.78424	0.00715	0.00477	0.000009318	0.71327	0.57611	0.91148
MODAL	Mode	62	0.024436	0.00465	0.00014	0.00852	0.91233	0.95006	0.79276	0.00071	0.01925	0.00037	0.71398	0.59536	0.91184
MODAL	Mode	63	0.022811	0.0000482	0.00007678	0.00029	0.91238	0.95013	0.79305	0.03695	0.00052	0.00762	0.75093	0.59588	0.91946
MODAL	Mode	64	0.019525	0.00001067	0.00519	0.01974	0.91239	0.95532	0.8128	0.18581	0.03964	0.00008754	0.93674	0.63552	0.91955
MODAL	Mode	65	0.019229	0.00006541	0.00093	0.07319	0.91246	0.95625	0.88599	0.00636	0.14488	0.00001139	0.9431	0.7804	0.91956
MODAL	Mode	66	0.019056	0.00023	0.00094	0.00213	0.91269	0.9572	0.88812	0.00439	0.00282	0.002	0.94748	0.78322	0.92156
MODAL	Mode	67	0.01858	0.00136	0.00136	0.000005002	0.91404	0.95855	0.88812	0.000007507	0.00039	0.00036	0.94749	0.78361	0.92192
MODAL	Mode	68	0.017905	0.00046	0.00011	0.04397	0.9145	0.95866	0.93209	0.00345	0.09045	0.00008509	0.95094	0.87406	0.922
MODAL	Mode	69	0.017153	0.00055	0.00099	0.00901	0.91505	0.95965	0.94111	0.00116	0.01061	0.00005007	0.9521	0.88467	0.92205
MODAL	Mode	70	0.016216	0.00016	0.00071	0.01704	0.91521	0.96036	0.95815	0.00258	0.04242	0.00061	0.95468	0.92709	0.92266
MODAL	Mode	71	0.016114	0.00027	0.00086	0.00192	0.91548	0.96122	0.96007	0.00111	0.00916	0.00043	0.9558	0.93625	0.92309
MODAL	Mode	72	0.014838	0.00002366	9.826E-07	0.00000285	0.91551	0.96122	0.96007	0.00638	0.00002309	0.00044	0.96218	0.93627	0.92353
MODAL	Mode	73	0.014501	0.00142	0.00001156	0.00143	0.91693	0.96124	0.9615	0.00002158	0.00234	1.379E-07	0.9622	0.93861	0.92353
MODAL	Mode	74	0.013461	0.00095	3.003E-07	0.00023	0.91788	0.96124	0.96174	2.369E-07	0.00000833	0.000002273	0.9622	0.93869	0.92353
MODAL	Mode	75	0.012577	0.00024	0.000001771	0.00005946	0.91812	0.96124	0.9618	0.00068	0.00032	4.527E-07	0.96287	0.93901	0.92353
MODAL	Mode	76	0.01234	8.284E-07	0.000004718	0.00082	0.91812	0.96125	0.96262	0.00055	0.00192	0.00001932	0.96343	0.94093	0.92355
MODAL	Mode	77	0.011767	0.00018	0.00001058	0.0019	0.91831	0.96126	0.96452	0.00034	0.00175	0.000001945	0.96377	0.94268	0.92355
MODAL	Mode	78	0.011079	0.00098	1.601E-07	0.000004616	0.91929	0.96126	0.96453	6.491E-09	0.00017	0.000002318	0.96377	0.94285	0.92355
MODAL	Mode	79	0.010994	0.00003521	0.00034	0.000009489	0.91933	0.9616	0.96454	0.01092	0.00004133	4.344E-07	0.97469	0.94289	0.92356
MODAL	Mode	80	0.010802	0.00044	0.00000801	0.000007623	0.91977	0.96161	0.96454	0.00089	0.00017	0.0000153	0.97558	0.94306	0.92357
MODAL	Mode	81	0.010275	9.398E-09	0.00002268	0.000002382	0.91977	0.96163	0.96455	0.0001	0.00002207	0.00005958	0.97568	0.94308	0.92363
MODAL	Mode	82	0.009681	0.000001033	0.000003395	0.00011	0.91977	0.96163	0.96466	0.00002547	0.00016	0.00001752	0.97571	0.94325	0.92365
MODAL	Mode	83	0.009634	0.000004783	0.000004804	0.000004844	0.91978	0.96164	0.96466	0.00001651	0.00015	0.00002591	0.97573	0.9434	0.92367
MODAL	Mode	84	0.009124	0.00006767	3.523E-07	0.00069	0.91985	0.96164	0.96536	0.00017	0.00151	0.00002052	0.9759	0.94491	0.92369
MODAL	Mode	85	0.008961	0.00001832	0.000006061	0.00006445	0.91986	0.96164	0.96542	0.00009861	0.00008076	0.00001699	0.976	0.94499	0.92371
MODAL	Mode	86	0.008427	0.00001413	0.000001714	0.0000659	0.91988	0.96165	0.96549	0.0000961	0.0002	0.00003357	0.9761	0.94519	0.92374
MODAL	Mode	87	0.00818	0.00001293	0.00004234	0.000003886	0.91989	0.96169	0.96549	3.705E-07	0.00024	0.00027	0.9761	0.94544	0.92402
MODAL	Mode	88	0.007901	0.00007014	0.00004241	0.00007605	0.91996	0.96173	0.96557	0.00015	0.00059	0.00003577	0.97625	0.94602	0.92406
MODAL	Mode	89	0.007769	0.000005623	0.00005361	0.00018	0.91997	0.96179	0.96575	0.00015	0.00008774	0.00002539	0.97639	0.94611	0.92408
MODAL	Mode	90	0.007552	0.000008654	0.00001526	0.00066	0.91998	0.9618	0.96641	0.00004282	0.00073	8.168E-08	0.97644	0.94684	0.92408
MODAL	Mode	91	0.007479	0.00004125	0.00028	0.00003489	0.92002	0.96208	0.96644	0.00009446	0.00015	0.00021	0.97653	0.94699	0.92429
MODAL	Mode	92	0.007402	0.00004127	0.00015	0.00002316	0.92006	0.96223	0.96647	0.00002235	0.00002279	0.00008226	0.97655	0.94701	0.92437
MODAL	Mode	93	0.007176	0.00008399	0.00005055	9.923E-07	0.92014	0.96228	0.96647	0.00004344	0.00006	0.00044	0.9766	0.94707	0.92481
MODAL	Mode	94	0.007111	0.00046	3.282E-07	0.00006228	0.9206	0.96228	0.96653	0.000007405	0.000001443	0.000006901	0.9766	0.94708	0.92482
MODAL	Mode	95	0.006773	0.00007668	0.00003844	0.000001738	0.92068	0.96232	0.96653	0.00003442	0.00011	0.0000134	0.97664	0.94719	0.92483
MODAL	Mode	96	0.006712	0.00012	0.000006126	0.00001951	0.9208	0.96233	0.96655	0.00012	0.00018	0.00002276	0.97676	0.94737	0.92486
MODAL	Mode	97	0.006451	0.00009075	0.000001303	0.000001263	0.92089	0.96233	0.96655	0.00001013	0.000001706	0.00000476	0.97677	0.94737	0.92486
MODAL	Mode	98	0.006339	0.00016	0.0000376	0.00018	0.92105	0.96237	0.96674	0.00009726	0.00043	0.00001946	0.97687	0.9478	0.92488
MODAL	Mode	99	0.006223	0.0000172	0.00005939	0.00001836	0.92107	0.96242	0.96676	0.00035	0.0000367	4.323E-08	0.97722	0.94783	0.92488
MODAL	Mode	100	0.006054	0.00002498	0.000007793	2.137E-07	0.92109	0.96243	0.96676	0.000005021	0.000001003	0.000004423	0.97723	0.94784	0.92488

Long.
Vertical
Trans.

20. MODAL ANALYSIS OUTPUT (WEST to EAST Case)

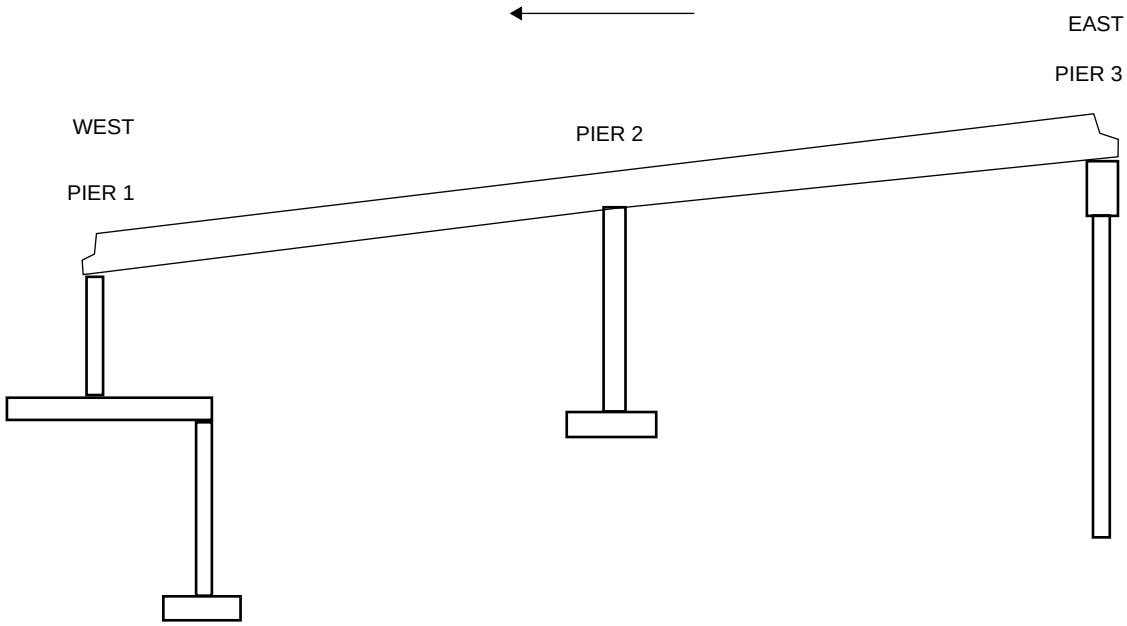
TABLE: Modal Load Participation Ratios

OutputCase	ItemType	Item	Static	Dynamic
Text	Text	Text	Percent	Percent
MODAL	Acceleration	UX	100.00	92.70
MODAL	Acceleration	UY	100.00	96.05
MODAL	Acceleration	UZ	100.00	94.42

TABLE: Modal Participating Mass Ratios

OutputCase	StepType	StepNum	Period	UX	UY	UZ	SumUX	SumUY	SumUZ	RX	RY	RZ	SumRX	SumRY	SumRZ	
Text	Text	Unitless	Sec	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless	
MODAL	Mode	1	1.175059	1.458E-07	4.777E-08	0.0001467	1.458E-07	4.777E-08	0.0001467	0.00002553	0.0001626	3.28E-08	0.00002553	0.0001626	3.28E-08	
MODAL	Mode	2	1.175051	1.574E-11	0.0001488	4.383E-08	1.458E-07	0.0001488	0.0001467	0.00005419	4.847E-08	0.0001622	0.00007973	0.0001626	0.0001622	
MODAL	Mode	3	1.175049	1.715E-12	6.608E-07	6.765E-09	1.458E-07	0.0001495	0.0001467	0.00006803	7.353E-09	9.011E-07	0.0001478	0.0001626	0.0001631	
MODAL	Mode	4	1.175048	0	1.647E-20	0	1.458E-07	0.0001495	0.0001467	0	0	2.964E-19	0.0001478	0.0001626	0.0001631	
MODAL	Mode	5	1.175048	0	0	0	1.458E-07	0.0001495	0.0001467	0	0	3.918E-19	0.0001478	0.0001626	0.0001631	
MODAL	Mode	6	1.175048	0	4.011E-20	0	1.458E-07	0.0001495	0.0001467	9.559E-20	0	5.597E-18	0.0001478	0.0001626	0.0001631	
MODAL	Mode	7	1.175048	0	3.899E-18	0	1.458E-07	0.0001495	0.0001467	9.217E-18	0	5.658E-16	0.0001478	0.0001626	0.0001631	
MODAL	Mode	8	1.175048	0	7.915E-19	0	1.458E-07	0.0001495	0.0001467	1.24E-18	0	5.922E-17	0.0001478	0.0001626	0.0001631	
MODAL	Mode	9	0.928367	3.413E-09	6.254E-10	0.00006813	1.492E-07	0.0001495	0.0002149	0.00002727	0.00005287	1.85E-10	0.000175	0.0002155	0.0001631	
MODAL	Mode	10	0.928358	0.000003259	0.00006299	1.577E-11	0.000003408	0.0002125	0.0002149	0.00001486	3.001E-08	0.00004944	0.0001899	0.0002155	0.0002125	
MODAL	Mode	11	0.928357	6.771E-11	2.229E-09	9.665E-09	0.000003408	0.0002125	0.0002149	8.065E-07	7.742E-09	1.34E-09	0.0001907	0.0002155	0.0002125	
MODAL	Mode	12	0.928357	7.254E-11	1.231E-09	1.901E-14	0.000003408	0.0002125	0.0002149	5.695E-10	7.487E-13	6.092E-09	0.0001907	0.0002155	0.0002125	
MODAL	Mode	13	0.925137	5.622E-09	1.176E-14	0.00006746	0.000003414	0.0002125	0.0002823	0.00004051	0.00005241	7.253E-10	0.0002312	0.0002679	0.0002125	
MODAL	Mode	14	0.925128	0.000003028	0.00006334	2.814E-09	0.000006442	0.0002758	0.0002823	0.00001513	4.699E-08	0.00004199	0.0002463	0.000268	0.0002545	
MODAL	Mode	15	0.925127	8.108E-11	2.217E-09	2.612E-08	0.000006442	0.0002758	0.0002824	0.000001809	1.989E-08	9.505E-10	0.0002481	0.000268	0.0002545	
MODAL	Mode	16	0.925127	1.677E-10	3.134E-09	2.228E-14	0.000006442	0.0002758	0.0002824	1.368E-09	2.249E-12	1.338E-08	0.0002481	0.000268	0.0002546	
MODAL	Mode	17	0.832485	7.747E-07	8.38E-09	0.0001279	0.000007217	0.0002759	0.0004103	0.00002185	0.00001681	1.05E-08	0.00027	0.0002848	0.0002546	
MODAL	Mode	18	0.83248	6.643E-09	0.0001321	8.74E-09	0.000007223	0.0004079	0.0004103	0.00006253	6.74E-10	0.00001517	0.0003325	0.0002848	0.0002697	
MODAL	Mode	19	0.832475	1.69E-10	0.000003059	1.251E-08	0.000007224	0.000411	0.0004103	0.00004716	1.574E-09	2.362E-07	0.0003797	0.0002848	0.00027	
MODAL	Mode	20	0.832474	8.498E-18	2.582E-18	1.824E-18	0.000007224	0.000411	0.0004103	0	0	1.642E-18	0.0003797	0.0002848	0.00027	
MODAL	Mode	21	0.832474	3E-19	2.655E-18	1.36E-18	0.000007224	0.000411	0.0004103	8.296E-19	1.766E-18	6.489E-19	0.0003797	0.0002848	0.00027	
MODAL	Mode	22	0.832474	7.528E-20	4.843E-19	1.625E-19	0.000007224	0.000411	0.0004103	2.737E-19	1.759E-20	0	0.0003797	0.0002848	0.00027	
MODAL	Mode	23	0.832474	2.57E-19	2.627E-19	0	0.000007224	0.000411	0.0004103	3.253E-20	0	1.358E-20	0.0003797	0.0002848	0.00027	
MODAL	Mode	24	0.832474	0	1.497E-19	1.289E-20	0.000007224	0.000411	0.0004103	8.133E-19	0	1.89E-20	0.0003797	0.0002848	0.00027	
MODAL	Mode	25	0.590636	0.000001057	2.042E-07	0.0001135	0.000008281	0.0004112	0.0005237	0.00002243	0.00001135	3.086E-08	0.0004021	0.0002962	0.00027	
MODAL	Mode	26	0.590631	3.006E-08	0.0001223	1.846E-07	0.000008311	0.0005334	0.0005239	0.00005624	2.676E-08	0.000005952	0.0004584	0.0002962	0.000276	
MODAL	Mode	27	0.590626	1.177E-09	0.000003065	3.802E-08	0.000008312	0.0005365	0.000524	0.00003803	3.141E-09	1.149E-07	0.0004964	0.0002962	0.0002761	
MODAL	Mode	28	0.590625	5.086E-17	7.04E-12	3.367E-16	0.000008312	0.0005365	0.000524	4.802E-12	4.032E-18	1.504E-10	0.0004964	0.0002962	0.0002761	
MODAL	Mode	29	0.590625	4.937E-20	1.835E-15	1.447E-19	0.000008312	0.0005365	0.000524	1.278E-15	0	4.008E-14	0.0004964	0.0002962	0.0002761	
MODAL	Mode	30	0.590625	3.399E-20	2.641E-16	0	0.000008312	0.0005365	0.000524	1.897E-16	0	5.712E-15	0.0004964	0.0002962	0.0002761	
MODAL	Mode	31	0.590625	0	1.782E-18	0	0.000008312	0.0005365	0.000524	9.065E-19	2.65E-20	3.147E-17	0.0004964	0.0002962	0.0002761	
MODAL	Mode	32	0.590625	0	3.141E-17	0	0.000008312	0.0005365	0.000524	2.078E-17	0	6.621E-16	0.0004964	0.0002962	0.0002761	
MODAL	Mode	33	0.240001	0.1246	0.0009097	0.0048	0.1246	0.0014	0.0053	0.0001135	0.1686	0.0001412	0.0006099	0.1689	0.0004173	→ Vertical
MODAL	Mode	34	0.197533	0.5269	0.0566	0.0316	0.6515	0.0581	0.0369	0.0086	0.0874	0.0012	0.0092	0.2563	0.0017	→ Long.
MODAL	Mode	35	0.17855	0.0153	0.6201	0.1088	0.6667	0.6782	0.1457	0.1962	0.0014	0.0803	0.2055	0.2577	0.082	→ Trans.
MODAL	Mode	36	0.169896	0.0868	0.1063	0.4454	0.7535	0.7844	0.5911	0.0004297	0.009	0.0211	0.2059	0.2667	0.1031	
MODAL	Mode	37	0.09996	0.0036	0.0142	0.00014	0.7571	0.7986	0.5912	0.0472	0.000007695	0.6105	0.2531	0.2667	0.7136	
MODAL	Mode	38	0.097617	0.0152	0.00001142	0.00005949	0.7723	0.7986	0.5913	0.0003346	0.0041	0.0067	0.2534	0.2708	0.7203	
MODAL	Mode	39	0.081067	0.0002894	1.206E-07	0.0385	0.7726	0.7986	0.6298	0.0276	0.0001939	0.00000969	0.2811	0.271	0.7203	
MODAL	Mode	40	0.064785	1.741E-07	0.000000816	0.00001576	0.7726	0.7986	0.6298	0.000002644	0.0001603	9.557E-07	0.2811	0.2712	0.7203	
MODAL	Mode	41	0.064782	1.369E-07	0.00002891	2.107E-07	0.7726	0.7987	0.6298	0.000003984	0.00000188	0.00003479	0.2811	0.2712	0.7204	
MODAL	Mode	42	0.064781	6.804E-10	1.047E-09	3.344E-08	0.7726	0.7987	0.6298	4.344E-07	2.735E-07	1.06E-08	0.2811	0.2712	0.7204	
MODAL	Mode	43	0.064781	2.76E-11	2.447E-10	5.459E-15	0.7726	0.7987	0.6298	1.193E-09	7.384E-12	3.44E-09	0.2811	0.2712	0.7204	
MODAL	Mode	44	0.063868	6.035E-07	0.000000307	0.00001568	0.7726	0.7987	0.6298	0.00002433	0.0002461	2.61E-09	0.2811	0.2714	0.7204	
MODAL	Mode	45	0.063864	6.389E-08	0.00003377	3.54E-09	0.7726	0.7987	0.6298	0.00000614	2.164E-07	0.00004476	0.2811	0.2714	0.7204	
MODAL	Mode	46	0.063862	3.28E-10	2.995E-09	9.923E-09	0.7726	0.7987	0.6298	9.413E-08	2.036E-07	8.552E-09	0.2811	0.2714	0.7204	
MODAL	Mode	47	0.063862	1.079E-11	9.287E-13	2.709E-16	0.7726	0.7987	0.6298	2.684E-10	3.706E-12	4.314E-10	0.2811	0.2714	0.7204	
MODAL	Mode	48	0.061529	0.0003482	1.985E-08	0.0001301	0.773	0.7987	0.6299	0.0002	0.0758	0.00001475	0.2813	0.3473	0.7204	
MODAL	Mode	49	0.048584	0.0002454	0.0708	0.0000325	0.7732	0.8695	0.63	0.0016	0.00001356	0.0604	0.2829	0.3473	0.7808	
MODAL	Mode	50	0.044199	0.00006126	0.0004295	0.0536	0.7733	0.8699	0.6835	0.0051	0.0012	0.00007126	0.288	0.3485	0.7809	
MODAL	Mode	51	0.036411	3.498E-07	6.001E-07	4.479E-07	0.7733	0.8699	0.6835	0.0005264	6.458E-08	0.0015	0.2886	0.3485	0.7824	
MODAL	Mode	52	0.035517	0.00009459	0.0097	0.0002227	0.7734	0.8796	0.6838	0.3039	0.0001045	0.0092	0.5925	0.3486	0.7916	
MODAL	Mode	53	0.035262	0.0106	0.0006795	5.528E-08	0.7839	0.8802	0.6838	0.0288	0.0018	0.0106	0.6213	0.3504	0.8022	
MODAL	Mode	54	0.034914	0.0094	0.001	0.00003724	0.7933	0.8813	0.6838	0.0073	0.0004886	0.00002033	0.6285	0.3509	0.8022	
MODAL	Mode	55	0.033155	0.0804	0.0026	0.0000903	0.8737	0.8838	0.6839	0.000003724	0.0104	0.0016	0.6285	0.3613	0.8038	
MODAL	Mode	56	0.030282	0.0011	0.0037	0.0012	0.8748	0.8875	0.6851	0.000004022	0.0398	0.0053	0.6285	0.401	0.8091	
MODAL	Mode	57	0.029811	0.001	0.041	0.000003371	0.8758	0.9285	0.6851	0.0067	0.0042	0.0671	0.6352	0.4052	0.8762	
MODAL	Mode	58	0.025234	0.0000685	0.000377	0.0163	0.8759	0.9289	0.7014	0.0039	0.0055	0.0013	0.6391	0.4107	0.8774	
MODAL	Mode	59	0.023093	0.0005244	0.0111	0.0014	0.8764	0.94	0.7028	0.0464	0.00005713	0.0103	0.6855	0.4107	0.8877	
MODAL	Mode	60	0.020506	0.0083	0.0001665	0.0003213	0.8848	0.9401	0.7031	0.0027	0.00004398	0.0000013	0.6882	0.4108	0.8877	
MODAL	Mode	61	0.019648	0.0002255	0.0008358	0.0112	0.885	0.941	0.7142	0.118	0.021	0.0049	0.8062	0.4318	0.8926	
MODAL	Mode	62	0.019091	0.0014	0.0001445	0.0532	0.8864	0.9411	0.7674	0.0077	0.1744	0.0012	0.8139	0.6062	0.8938	
MODAL	Mode	63	0.01864	0.0091	0.0009488	0.0123	0.8955	0.942	0.7797	0.0327	0.0215	0.0035	0.8466	0.6277	0.8974	
MODAL	Mode	64	0.018414	0.0027	0.0021	0.0035	0.8982	0.9441	0.7832	0.0153	0.0102	0.0054	0.8619	0.6378	0.9027	
MODAL	Mode	65	0.018238	0.0014	9.651E-07	0.0107	0.8996	0.9441	0.7939	0.0159	0.0359	0.0013	0.8777	0.6737	0.904	
MODAL	Mode	66	0.017379	0.0006183	0.0012	0.0607	0.9002	0.9453	0.8546	0.0029	0.0218	0.0004062	0.8806	0.6956	0.9044	
MODAL	Mode	67	0.016355	0.0005668	0.0038	0.0099	0.9008	0.9491	0.8645	0.0003393	0.0044	0.00004518	0.881	0.6999	0.9044	
MODAL	Mode	68	0.015758	0.0014	0.00002838	0.0162	0.9022	0.9491	0.8807	0.0046	0.0853	0.000002844	0.8856	0.7853	0.9044	
MODAL	Mode	69	0.015163	0.0019	0.00001561	0.0007561	0.9041	0.9492	0.8814	0.0001437	0.0048	0.0008657	0.8857	0.7901	0.9053	
MODAL	Mode	70	0.014752	0.00002727	0.0041	0.00009603	0.9041	0.9533	0.8815	0.0136	0.0009505	0.0039	0.8993	0.791	0.9092	
MODAL	Mode	71	0.012906	0.000006298	0.00005451	0.0017	0.9041	0.9533	0.8832	0.0013	0.0000664	0.0002949	0.9006	0.7911	0.9095	
MODAL	Mode	72	0.012662	0.000002415	0.0015	0.0023	0.9041	0.9548	0.8856	0.0001704	0.0005395	0.0026	0.9008	0.7916	0.9122	
MODAL	Mode	73	0.012308	0.00001787	0.000302	0.0026	0.9041	0.9552	0.8882	0.0011	0.0047	0.000623	0.9018	0.7963	0.9128	
MODAL	Mode	74	0.01159	0.0000339	0.00006499	0.0011	0.9042	0.9552	0.8893	0.00004726	0.0062	0.0002435	0.9019	0.8025	0.913	
MODAL	Mode	75	0.011059	0.0007787	0.00002087	0.0002224	0.9049	0.9552	0.8895	4.912E-07	0.0002115	0.0000154	0.9019	0.8027	0.9131	
MODAL	Mode	76	0.011004	0.000004073	0.000009265	2.885E-08	0.9049	0.9552	0.8895	0.0089	0.00001991	0.0008147	0.9108	0.8028	0.9139	
MODAL	Mode	77	0.010328	0.00003399	0.0002076	0.00006131	0.905	0.9555	0.8896	0.00001457	0.0011	0.0001557	0.9108	0.8039	0.914	
MODAL	Mode	78	0.010252	0.0002431	0.000968	0.0001252	0.9052	0.9564	0.8897	0.0039	0.0001543	0.0012	0.9147	0.804	0.9152	
MODAL	Mode	79	0.009783	0.0001456	0.00002041	0.0051	0.9054	0.9564	0.8948	0.0014	0.0066	0.00004185	0.9161	0.8106	0.9153	
MODAL	Mode	80	0.009444	0.000005717	0.000006216	0.0003443	0.9054	0.9564	0.8951	0.000004135	0.0022	0.000004693	0.9161	0.8128	0.9153	
MODAL	Mode	81	0.008999	0.00002343	0.0007227	0.00002379	0.9054	0.9572	0.8951	0.0003451	0.0006043	0.001	0.9165	0.8134	0.9163	
MODAL	Mode	82	0.008635	0.0037	0.000008636	0.000003792	0.9091	0.9572	0.8951	0.000147	0.0016	0.0000259	0.9166	0.815	0.9163	
MODAL	Mode	83	0.008251	0.0097	8.093E-07	0.0009554	0.9188	0.9572	0.8961	0.000004102	0.0004444	0.0002505	0.9166	0.8155	0.9166	
MODAL	Mode	84	0.008165	0.0006713	0.0001764	0.000003125	0.9195	0.9574	0.8961	0.000615	0.0002036	0.0004448	0.9173	0.8157	0.917	
MODAL	Mode	85	0.008068	0.0009188	0.000001583	0.0077	0.9204	0.9574	0.9038	0.0021	0.0157	0.000004461	0.9193	0.8313	0.917	
MODAL	Mode	86	0.007749	0.0009436	0.0002294	0.0017	0.9214	0.9576	0.9054	0.0001698	0.0007314	0.0003114	0.9195	0.8321	0.9173	
MODAL	Mode	87	0.007581	0.0007042	0.00001761	0.0056	0.9221	0.9576	0.911	0.0016	0.0008621	0.000008566	0.9211	0.8329	0.9173	
MODAL	Mode	88	0.007487	0.0002527	0.00007596	0.00005982	0.9223	0.9577	0.911	0.00001834	0.0012	0.00004311	0.9212	0.8341	0.9174	
MODAL	Mode	89	0.00742	0.00004245	0.000002675	0.00007873	0.9224	0.9577	0.9111	0.0001168	6.671E-07	0.0000959	0.9213	0.8341	0.9175	
MODAL	Mode	90	0.007109	0.00004535	0.0005249	0.00001945	0.9224	0.9582	0.9111	0.0001333	0.00001874	0.002	0.9214	0.8341	0.9195	
MODAL	Mode	91	0.006964	1.321E-07	0.0000306	0.0126	0.9224	0.9582	0.9237	0.0036	0.0344	0.00005436	0.925	0.8685	0.9195	
MODAL	Mode	92	0.006632	8.167E-07	0.00007773	0.0025	0.9224	0.9583	0.9262	0.0014	0.0045	0.0001619	0.9264	0.873	0.9197	
MODAL	Mode	93	0.006567	0.00000992	0.0002649	0.0026	0.9224	0.9586	0.9288	0.0004686	0.0027	0.0002991	0.9269	0.8758	0.92	
MODAL	Mode	94	0.006381	0.0009008	0.0004303	0.0014	0.9233	0.959	0.9302	0.0015	0.0004627	0.0004139	0.9284	0.8762	0.9204	
MODAL	Mode	95	0.006262	0.00004846	0.0008197	0.000001316	0.9234	0.9598	0.9302	0.0021	0.0002184	0.0008473	0.9305	0.8765	0.9213	
MODAL	Mode	96	0.006213	0.0004231	0.00001052	0.0101	0.9238	0.9598	0.9403	0.003	0.0252	0.00004389	0.9335	0.9017	0.9213	
MODAL	Mode	97	0.006062	0.00001936	0.0006609	3.159E-07	0.9238	0.9605	0.9403	0.0006358	3.051E-09	0.0015	0.9342	0.9017	0.9228	
MODAL	Mode	98	0.005952	0.0029	0.000001153	0.0001012	0.9267	0.9605	0.9404	0.000003004	0.000001293	0.00003661	0.9342	0.9017	0.9229	
MODAL	Mode	99	0.005851	0.00009017	2.284E-07	0.000002353	0.9268	0.9605	0.9404	0.000007333	0.00007851	3.363E-07	0.9342	0.9018	0.9229	
MODAL	Mode	100	0.005759	0.0001597	0.000008663	0.0038	0.927	0.9605	0.9442	0.0012	0.0039	0.000007501	0.9354	0.9056	0.9229	

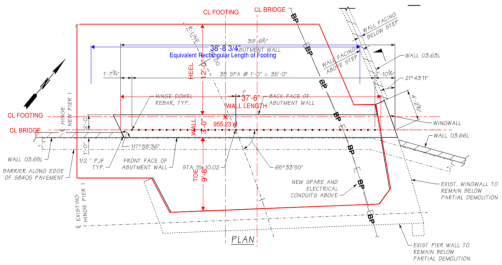
PIER 1 - FOOTING SIZING
EXT CASE (EAST TO WEST)



COWI	PROJECT	BR 17.7													A207833
	SUBJECT	Pier 1 Abutment Footing Sizing EQ LOADS (East to West)													
CHECKED BY MWBM	DATE	2021-11-09	CALCS BY RSGR		DATE	2021-11-09					PAGE				

Purpose:
Check bearing, sliding and overturning of the footing
Notes:
- The wall is assumed to be pinned at top and fixed at bottom in bridge longitudnal direction. Hence, moment due to lateral loads in long. direction are less than a pure cantilevered wall case.
- The wall is to be treated as cantilevered in transverse direction
- Load inputs for DL, LL, CR, SH, PS, SE and EQ are from the CSi model at the base of wall and the shears are multiplied by half the footing depth to be added to wall base moments
- The wall is designed to develop a plastic hinge at the base about the bridge transverse direction. Hence, the longitudinal moment (M3) and shear (V2) for EQ case is plastic overstrength capacity of wall about its weak axis.
- The wall will stay elastic about its strong axis. EQ transverse demands are taken from RSA of the bridge and a factor of 1.0 is used
- Assumed there is no friction between back of wall and fill
- DL has been maximized in bearing check and minimized in sliding/eccentricity check as per AASHTO combos
- The new footing is tied to existing pier wall to reduce the overturning moments at the new footing and to reduce the bearing stress under new footing
- AASHTO SEISMIC 6.3.2, (L - Dc)/(2Hf) = 2.7 > 2.5 in bridge longitudinal direction for the footing. But the footing is analysed and designed by assuming it to be rigid conservatively. This ignores the flexible behaviour of footing and hence, the footing demands are high.

User input=				**References**
Manual Load input=				
Unit Weight of Soil, γ_{soil}=	130	pcf		Geotech info
Width of Wall, L_w=	37.50	ft		
Length of Footing, L_f=	38.72	ft		Length of footing is measured in transverse direction; this value is chosen such that L X B is same as actual area of footing
Width of Footing, B_f=	24.67	ft	=toe+heel+wall	Calculated
Depth of Footing, D_f=	4	ft		Designed
Concrete unit weight =	155	pcf		



Step 1: Bearing Capacities

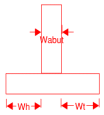
a) Bearing Capacities

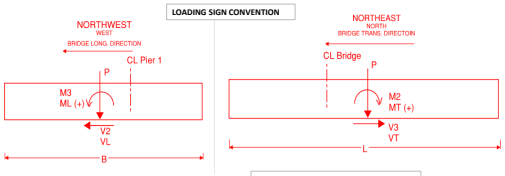
FOOTING BEARING RESISTANCE, Φq_n (KSF)			
Footing Width (Ft)	Footing Length (Ft)		Extreme
ANY	ANY		**12.0**

From Geotech Report

Step 2: a) Demands (WHOLE STRUCTURE +LIVE LOAD AND SUPER IMPOSED LOADS)

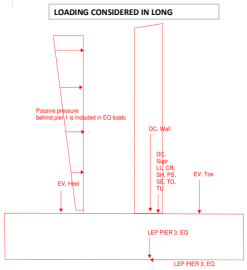
Average Height of pier wall, H_{abut}=	19.50	ft		Designed
Height of Abutment+footing, $H_{abut+footing}$=	23.50	ft	=Habut + Df	Added footing height
Height of Superstructure, H_s=	4.25	ft		
Total Height of Abutment, H=	27.75	ft		Wall + Footing + Superstructure
Width of the abutment stem at base, W_{abut}=	3.0	ft		Designed
Width of the Toe @ abutment, W_t=	9.7	ft		Designed
Width of the Heel @ abutment, W_h=	12.0	ft		Designed





V2= VL: Shear in longitudinal direction
V3= VT: Shear in transverse direction

M2= MT: Moment about stong axis
M3= ML: Moment about weak axis



Load modifications to account for eccentricities CL Footing and CL Superstructure

Eccentricity between CL Superstructure and CL Footing in longitudinal direction =	-1.75	ft	-ve means towards east of footing CL
Eccentricity between CL Superstructure and CL Footing in transverse direction =	-4.54	ft	-ve means towards north of footing CL
Eccentricity between CL wall and CL Footing in longitudinal direction =	-1.27	ft	-ve means towards east of footing CL
Eccentricity between CL wall and CL Footing in transverse direction =	-4.54	ft	-ve means towards north of footing CL

Notes:
1. Values in orange shade below are the final values used for design check
2. The moment demands in orange shade below also include moment from shear at base of wall to centre of footing and eccentric offsets

	P	V2	V3		M2	M3
	Kip	Kip	Kip		Kip-ft	Kip-ft
DC, supr	510	7	4		1190	-139
DC, supr	0	0	0		-2316	-893
DC, Supr	510	7	4		-1118	-1018
DC, wall	340	0	0		-1544	-432
CR&SH	55	-13	-4		-410	250
CR&SH	0	0	0		-250	-96
CR&SH,	55	-13	-4		-668	128
LL MAX	-21	-148	-67		-1338	-3216
LL MAX	0	0	0		95	37
LL MIN,	-21	-148	-67		-1377	-3475
LL MAX	169	164	28		1573	2397
LL MAX	0	0	0		-768	-296
LL MAX,	169	164	28		861	2429
PS	16	-14	-5		-64	279
PS	0	0	0		-73	-28
PS,	16	-14	-5		-147	223
SE	-13	-5	0		-109	98
SE	0	0	0		59	23
SE,	-13	-5	0		-50	111
TG-	-15	-3	2		-55	52
TG-	0	0	0		68	26
TG- ,	-15	-3	2		17	72
TG+	51	9	-7		183	-174
TG+	0	0	0		-232	-89
TG+,	51	9	-7		-63	-245
TU-	-7	-125	-16		211	755
TU-	0	0	0		32	12
TU-,	-7	-125	-16		211	517
TU+	5	0	10		-132	7
TU+	0	0	0		-23	-9
TU+,	5	0	10		-135	-2
LS_Pier3	1	0	1		6	179
LS_Pier3	0	0	0		-5	-2
LS_Pier3	1	0	1		3	177
LEP_Pier3	53	11	36		218	6861
LEP_Pier3	0	0	0		-241	-93
LEP_Pier3	53	11	36		49	6790
1.0X+0.3Y+0.3Z	202	42	-524		-10871	12772
1.0X+0.3Y+0.3Z	0	0	0		-917	-257
1.0X+0.3Y+0.3Z	202	42	-524		-12836	12599
0.3X+1.0Y+0.3Z	226	18	-1374		-30393	4673
0.3X+1.0Y+0.3Z	0	0	0		-1026	-287
0.3X+1.0Y+0.3Z	226	18	-1374		-34167	4422

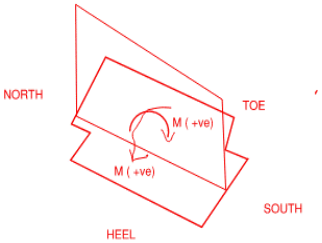
S behind Pier 3 already includes 0.5 factor

Use 30% of LEP values when combining with 100% Transverse loads from EQ (RSA)

EQ (RSA)

EQ (RSA)

V2= VL: Shear in longitudinal direction
V3= VT: Shear in transverse direction
M2= MT: Moment about stong axis
M3= ML: Moment about weak axis



For 100% Longitudinal Case the following overstrength V2 and M3 values shall be used to replace the values from combination of LEP_Pier3 + LS_Pier3 + EQ(RSA)

V2	M3
Kip	Kip-ft
1030	20094

Rashim Garg: From Moment curvature analysis. See separate calc

For 30% Longitudinal Case, these values obtained from RSA are to be used in combination with 30% LEP_Pier3 and LS_Pier3 with a 1.2 factor

Earth Loads: (EV)

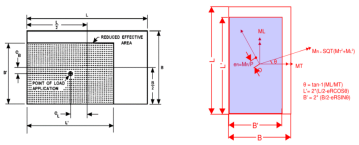
Vertical,EV:

Dead Load, Evheel=	1624	kips	includes the soil behind superstructure too	Using actual footing dimension = 0.130 x (23.75+2)ft x 485 sq. ft
Moment Arm from CL FOOTING=	6.3	ft		
Moment, M_{Ev}, heel=	10282	k-ft		
Dead Load, E_{vtoe}=	99.84	kips	Assuming 2ft of soil above toe (below i405 level)	Using actual footing dimension = 0.130 x 2ft x 384 sq. ft= 100 kips
Moment Arm from CL FOOTING=	-7.5	ft		
Moment at Point O, M_{Evtoe}=	-749	k-ft	Heel Stabilizing moment (M-ve)	
Net Moment due to EV about CL Footing, M(ev)L=	9534	k-ft		

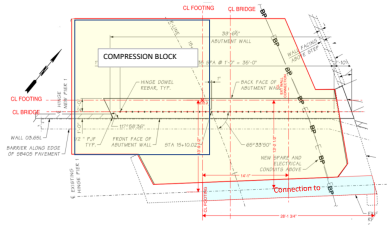
Weight of footing:

Axial load of footing, P=	592	kips
Moment due to footing forces, M_L=	0	k-ft

Step 3: Bearing Stress about footing center (considering effective footing Area):



Tie-in Hold Back Moment: For longitudinal EtoW and Transverse **NtoS**



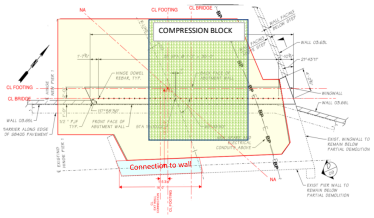
NOTE:

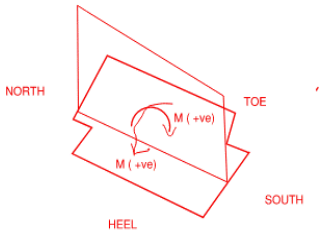
The new footing is tied to existing pier wall, which will remain in place after partial demolition to 2' below the finished grade (top of new footing). Only the tensile resistance from existing pier wall is used to resist overturning moments. This values is limted by the wall uplift capacity. Wall uplift capacity is determined from the net DL of existing wall and soil over its footing.

The existing pier wall and footing has plenty of compression capacity to handle compression loads transferred from the new footing, Hence, this load path is conservatively ignored for design of the new footing for West to East direction. See WtoE direction calcs separately

Length of connection, L=	28.00	ft	from centroid of footing to north end of wall
Total Rebars, n	59.00	28 bar each face + 3 end face	
F'ye=	68.00	ksi	
Area of each rebar, Adb=	1.27	in2	#10 rebar
Total area of rebars Adb x n, As=	74.93	in2	
Max. Tension Capacity=	5095	kips	= As x Fye; However, limited by existing wall's uplift capacity, which is 1931 kips (see below)
Moment arm for longitudinal moment=	13.21	ft	from centroid of footing to middle of rebar
Moment arm for transverse moment=	14.08	ft	from centroid of footing to middle of rebar
Moment arm for bi-axial loading=	19.31	ft	
Moment Arm angle Tan $^{-1}$ (L/T)=	43	degrees	
Max. Resisting moment for long. loading=	27527	k-ft	= 2084 x 13.21 ft; at centroid of footing
	27527	k-ft	Use this value to resist overturning longitudinal moment if overturinng moment from load combination > 27,000k-ft; For overturning values lower than 27,000 k-ft, Use 0 k-ft as overturning moment
Max. Resisting moment for trans. loading=	29350	k-ft	= 2084 x 14.08 ft; at centroid of footing
	29350	k-ft	Use this value to resist overturning transverse moment if overturinng moment from load combination > 29000k-ft; For overturning values lower than 29000k-ft k-ft, Use 0 k-ft as overturning moment

Tie-in Hold Back Moment: For longitudinal EtoW and Transverse **StoN**
consider the connection in highlighted area to the south of footing CL



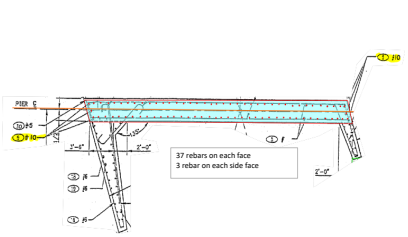


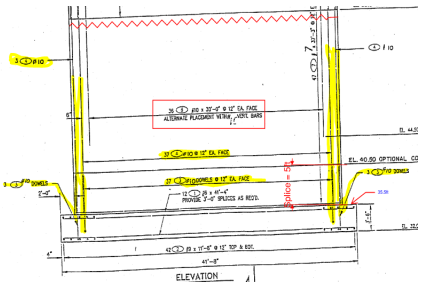
Length of connection, L=	16.000	ft	16ft of wall connection below NA axis of equilibirum
Total Rebars, n=	35	16 bar each face + 3 end bars	
F'ye=	68	ksi	
Area of each rebar, Adb=	1.27	in2	
Total area of rebars Adb x n, As=	44.45	in2	
Max. Tension Capacity=	3023	kips	= As x Fye; However, limited by existing wall's uplift capacity = 1931 kips; See calculation of this value below
Moment arm for longitudinal moment=	13.79	ft	from centroid of footing to middle of rebar
Moment arm for transverse moment=	1.48	ft	from centroid of footing to middle of rebar
Moment arm for bi-axial loading=	13.87	ft	
Max. Counter moment for long. loading=	28742	k-ft	= 2084 x 13.78 ft; at centroid of footing
	28742	k-ft	Use this value to resist overturning longitudinal moment if overturinng moment from load combination > 29000k-ft; For overturning values lower than 29000 k-ft, Use 0 k-ft as overturning moment
Max. Counter moment for trans. loading=	3083	k-ft	= 2084 x 1.48 ft; at centroid of footing
	3083	k-ft	Use this value to resist overturning transverse moment if overturinng moment from load combination > 3000k-ft; For overturning values lower than 3000 k-ft, Use 0 k-ft as overturning moment

Notes on capacity of rebar considered from existing Pier wall:

Total from each face	74	#10
Total from each edge	6	#10
Total	80	#10 rebar in the wall that are spliced with Pier Wall started bars

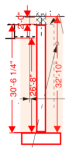
Notes:
1.0 There are additional 72#10 rebars (36 in each face (as marked)) that are excluded from capacity calculations as they do not extend to the footing
2.0 Vertical rebar in wings are conservatively not included
3.0 Footing #10 dowel (starter bar) and Pier wall #10 rebar are spliced. Splice length = 5ft which is adequate for #10 rebar in 4ksi concrete. Minimum required splice length = 4ft
4.0 Footing #10 dowel (starter bar) is hooked at 90 degree. The provided hook development length is 2.5' with 2' tail which meets the minimum requirement of 1.7' development and 2' hook tail





Self Weight of existing Pier and Soil:

Height of pier wall intact after conneting to new footing=	31.50	ft	
thickness of wall=	2.75	ft	
Length of wall=	37.50	ft	ignoring wing wall
Concrete Weight=	504	kips	0.155 kcf unit weight used
Height of footing=	3.50	ft	
width of footing=	12.00	ft	
Length of footing=	41.67	ft	
Concrete Weight=	271	kips	
Avg Height of soil above existing heel and toe below the new footing=	30.75	ft	
Length of heel and toe=	41.67	ft	
Width of heel and toe=	9.25	ft	
Soil Weight=	1541	kips	0.130 kcf unit weight used



Total Concrete + Soil Self weight=	2316	kips	Use 90% of this value for bridge foundation moment resistance. Check of existing pier footing reinforcement is done separately for Flexure and Shear strength using the full value
Max. Wall Tension Capacity for NtoS or StoN earthquake=	2084	kips	=90% of self weight of existing wall + footing + soil above footing

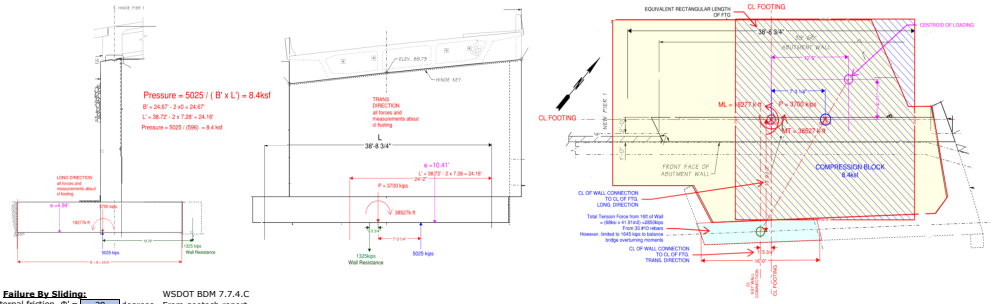
Bearing Stress about footing center (considering effective footing Area):

Rashim Garg: Here C/D < 1. i.e. Maximum bearing stress induced by footing exceeds allowable 12ksf limit. Hence, the footing is tied-in to existing wall to control the bearing stress by pulling onto existing wall. This patch of compression force is at North - West corner of footing and is checked against the tie in forces to ensure that bearing stress is always less than 12ksf

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 3)	TU, TG, SE	LS,H (Behind Pier 3)	EQ (RSA) =1.0 or EQ (Overstrength) = 1.2	Factored Axial Load,P (kips)	Factored Moments,M_L Longitudinal Direction (k-ft)	Factored Moments,M_T Transverse Direction (k-ft)	Factored Effective Moments, M_o(k-ft)	Eccentricity eo=M_o/P (FT)	Resultant angle,θ=Atan (M_L/M_T)	Effective Length, L' (ft)	Effective Width, B' (ft)	Maximum Soil Stress, σ (KSF)	Factored Soil Bearing Resistance, Φq_n	C/D	
Extreme I Load Case: -100%T + 30% L + 30% Z	1	0.5	1	1.2	1	1	1.2	3700	18277	-38527	42643	12	-0.443	17.89	15	**14**	12	**0.9**	~~StoN for 100%T~~ **(Max. Tensile resistance limited used= 1725 kips instead of max. 1931 kips to balance overturning moments)**
Extreme I Load Case: -100%T + 30% L - 30% Z	1	0.5	1	1.2	1	1	1.2	2984	18277	-38527	42643	14	-0.443	13	12	**19**	12	**0.6**	
Extreme I Load Case: -100%T + 30% L + 30% Z **+ tie-in**	1	0.5	1	1.2	1	1	1.2	5025	0	-36567	36567	7.28	0.000	24.16	24.7	**8.4**	12	**1.4**	
Extreme I Load Case: -100%T + 30% L - 30% Z **+ tie-in**	1	0.5	1	1.2	1	1	1.2	4310	0	-36567	36567	8.5	0.000	21.7	24.7	**8.0**	12	**1.5**	
Extreme I Load Case: 100%T + 30% L + 30% Z	1	0.5	1	1.2	1	1	1.2	3700	18277	31352	36291	10	0.528	22	15	**11**	12	**1.0**	
Extreme I Load Case: 100%T + 30% L - 30% Z	1	0.5	1	1.2	1	1	1.2	3091	18277	31352	36291	12	0.528	18.4	12.8	**13**	12	**0.9**	**NtoS for 100%T**
Extreme I Load Case: 100%T + 30% L + 30% Z **+ tie-in**	1	0.5	1	1.2	1	1	1.2	5718	0	2002	2002	0	0.000	38	25	**6**	12	**2.0**	
Extreme I Load Case: 100%T + 30% L - 30% Z **+ tie-in**	1	0.5	1	1.2	1	1	1.2	5175	0	2002	2002	0	0.000	37.9	25	**6**	12	**2.2**	
Extreme I Load Case: +30%Trans + 100% Long + 30% Vertical	1	0.5	1	0	1	1	1	3500	31985	8556	33109	9	1.309	34	6	**16**	12	**0.7**	
Extreme I Load Case: +30%Trans + 100% Long - 30% Vertical	1	0.5	1	0	1	1	1	3001	31985	8556	33109	11	1.309	33	3	**27**	12	**0.4**	**NtoS for 30%T**
Extreme I Load Case: +30%Trans + 100% Long + 30% Vertical **+ Tie-in**	1	0.5	1	0	1	1	1	5584	4458	0	4458	1	1.571	39	23	**6**	12	**1.9**	
Extreme I Load Case: +30%Trans + 100% Long - 30% Vertical **+ Tie in**	1	0.5	1	0	1	1	1	5085	4458	0	4458	1	1.571	39	23	**6**	12	**2.1**	
Extreme I Load Case:-30%Trans + 100% Long + 30% Vertical	1	0.5	1	0	1	1	1	3488	32504	-17196	36772	11	-1.084	29	6	**20**	12	**0.6**	
Extreme I Load Case: -30%Trans + 100% Long - 30% Vertical	1	0.5	1	0	1	1	1	2989	32504	-17196	36772	12	-1.084	27	3	**38**	12	**0.3**	**StoN for 30%T**
Extreme I Load Case:-30%Trans + 100% Long + 30% Vertical **+ tie-in**	1	0.5	1	0	1	1	1	5477	3761	-14114	14606	3	-0.260	34	23	**7**	12	**1.7**	
+ 100% Long - 30% Vertical **+ tie-in**	1	0.5	1	0	1	1	1	5061	3761	-14114	14606	3	-0.260	33	23	**7**	12	**1.8**	

Notes:
1. Earthquake demands EQ and EH from lateral pressure behind Pier 3 are used with a factor of 1.2 as per WSDOT BDM 15.4.2 B for 30% Longitudinal Case only
2. Earthquake demands for 100% Longitudinal case are limited to overstrength forces of wall (moment and shear) and are combined EDA demands from 30%T direction without using 1.2 factor
3. All EQ forces in transverse direction (Wall strong direction) are used without 1.2 factor

Check equilibrium of critical case: S to N Trans EQ (100%T + 30%L)



c) Failure By Sliding: WSDOT BDM 7.7.4.C

Effective angle of internal friction, Φ'_f = 38 degrees From geotech report

TAN δ=TAN Φ'_f = 0.78 (For cast-in place concrete against soil)

Note: We are conservatively ignoring any resistance from existing wall for sliding resistance

$$Q_R = \phi_T Q_T + \phi_{ep} Q_{ep}$$

x Resistance Factor (0.9 for the extreme case)

AASHTO BDS 3.4.1-1&2

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 3)	TU, TG, SE	LS,H	EQ (RSA)=1.2 or EQ (Overstrength)=1.0	Factored Horizontal loads, V2	Factored Horizontal loads, V3	SRSS V2 and V3	Factored sliding resistance, Q_R (kips)	C/D
100%L + 30%T	EXT	0.9	0.5	0.9	1.2	1	1	1103	-540	1228	2016	**1.6**
100%T + 30%L	EXT	0.9	0.5	0.9	1.2	1	1	1.2	82	-1399	1401	1895	**1.4**

Table 7.7.2-1 Load Factors

Sliding and Overturning, η_S	Bearing Stress (σ_n, η_S)
$LL_{min} = 0$	LL_{max}
DC_{min}, DW_{min} for resisting forces, DC_{max}, DW_{max} for causing forces,	DC_{max}, DW_{max} for causing forces DC_{min}, DW_{min} for resisting forces
EV_{min}	EV_{max}
EH_{min}	EH_{max}
LS	LS

d) Overturning Stability: LONGITUDNAL DIRECTION, ABOUT FOOTING WIDTH

LOAD CASE	DC, CR SH, PS	LL	Ev	TU, TG, SE	LS,H	Mpo	Factored Axial Load,P (kips)	Moments, Mo Longitudinal Direction (k-ft)	Eccentricity Xo=Mo/P (FT)
100%L + 30%T	EXT	0.9	0	0.9	1	0	1	4962	2919	0.59

Eccentricity Limits (100%L + 30%T): WSDOT BDM 7.7.4.D/AASHTO 11.6.3.3 AASHTO BDS 11.6.3.3

EXTREME CASE

Resultant force shall be within the middle 2/3 of the base width for LL=0.0 AASHTO BDS 11.6.5

Lower Limit= 4.111 ft

Upper Limit= 20.56 ft

(Extreme Case) B/2-Xo= 12.92 ft **OK within 2/3 limit**

(Extreme Case) B/2+Xo= 11.75 ft

Also check AASHTO SEISMIC 6.3.4-1 ignoring passive resistance from surrounding soil

Mpo=	20094	k-ft
Vpo=	1030	kips
Hf=	4	ft
Pu=	2989	kips
L=	24.67	ft
B=	38.72	ft
qn=	12	ksf
a=	6.43	ft
$M_{pr} + V_{pr} H_f$ =	24214	k-ft
$\phi P_r \left(\frac{L-a}{2}\right)$ =	27249	k-ft

Min. vertical load from 100%L case above

length in direction of loading

length perpendicular to direction of loading

by geotech

d) Overturning Stability: TRANSVERSE DIRECTION, ABOUT FOOTING LENGTH

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 3)	TU, TG, SE	LS,H	EQ (RSA)	Factored Axial Load,P (kips)	Factored Moments,Mo Trans Direction (k-ft)	Eccentricity Xo=Mo/P (FT)
100%T + 30%L	EXT	0.9	0	0.9	1	1	0	1	2705	-37495	13.86
100%T + 30%L + Tie-in	EXT	0.9	0	0.9	1	1	0	1	4789	-34412	7.19

Eccentricity Limits (100%L + 30%T) + tie-in : WSDOT BDM 7.7.4.D/AASHTO 11.6.3.3 AASHTO BDS 11.6.3.3

EXTREME CASE

Resultant force shall be within the middle 2/3 of the base width for LL=0 AASHTO BDS 11.6.5

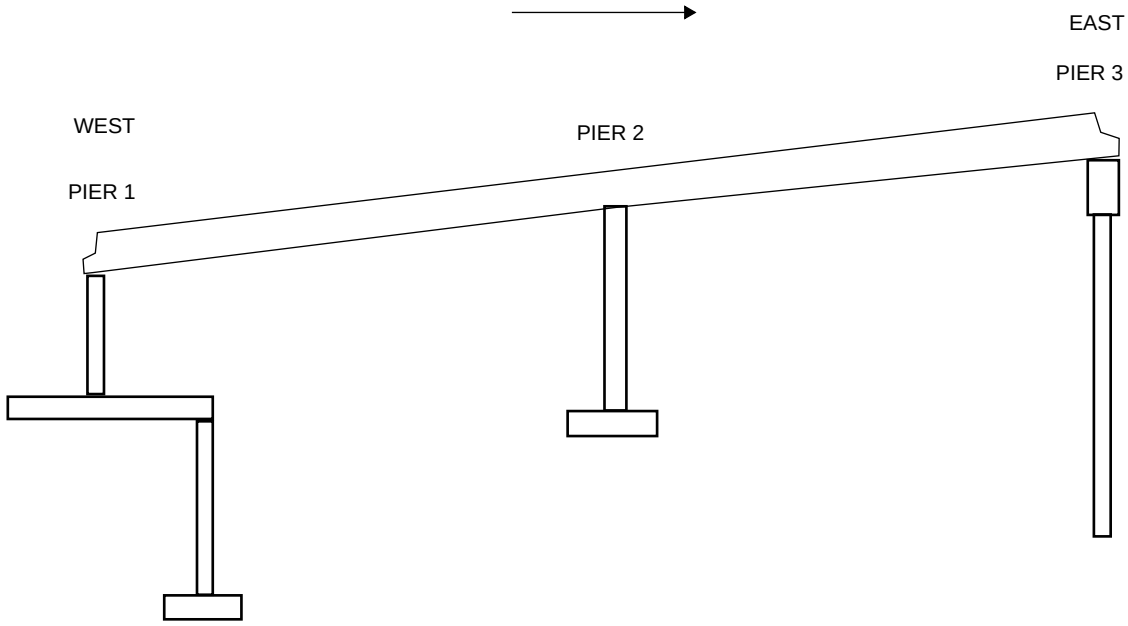
Lower Limit= 6.453 ft

Upper Limit= 32.26 ft

(Extreme Case) L/2-Xo= 12.17 ft **OK within 2/3 limit** We rely on connection to existing wall to control overturning limits

(Extreme Case) L/2+Xo= 26.54 **OK within 2/3 limit** We rely on connection to existing wall to control overturning limits

PIER 1 - FOOTING SIZING
EXT CASE (WEST TO EAST)



COWI	PROJECT	BR 17.7							CONT					A207833
	SUBJECT	Pier 1 Abutment Footing Sizing EQ LOADS (West to East)							PAGE					
CHECKED BY MWBM	DATE	2021-11-09	CALCS BY	RSGR	DATE	2021-11-09								

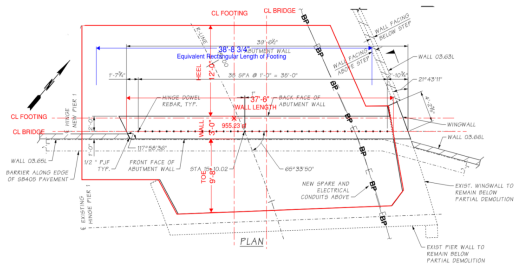
Purpose:
Check bearing, sliding and overturning of the footing
Notes:
- The wall is assumed to be pinned at top and fixed at bottom in bridge longitudnal direction. Hence, moment due to lateral loads in long. direction are less than a pure cantilevered wall case.
- The wall is to be treated as cantilevered in transverse direction
- Load inputs for DL, LL, CR, SH, PS, SE and EQ are from the CSI model at the base of wall and the shears are multiplied by half the footing depth to be added to wall base moments
- The wall is designed to develop a plastic hinge at the base about the bridge transverse direction. Hence, the longitudinal moment (M3) and shear (V2) for EQ case is plastic overstrength capacity of wall about its weak axis.
- The wall will stay elastic about its strong axis. EQ transverse demands are taken from RSA of the bridge and a factor of 1.0 is used
- Assumed there is no friction between back of wall and fill
- DL has been maximized in bearing check and minimized in sliding/eccentricity check as per AASHTO combos
- The new footing is tied to existing pier wall however, this connection is not relied upon for any compression resistance to reduce the bearing stress caused by West to East EQ

User input=				
Manual Load input=				
Unit Weight of Soil, γ_{soil}=	130	pcf		
Width of Wall, L_w=	37.50	ft		
Length of Footing, L_f=	38.72	ft		
Width of Footing, B_f=	24.67	ft	=toe+heel+wall	
Depth of Footing, D_f=	4	ft		
Concrete unit weight =	155	pcf		

References
Geotech info

Length of footing is measured in transverse direction; this value is chosen such that L X B is same as actual area of footing

Calculated
Designed



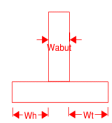
Step 1: Bearing Capacities
a) Bearing Capacities

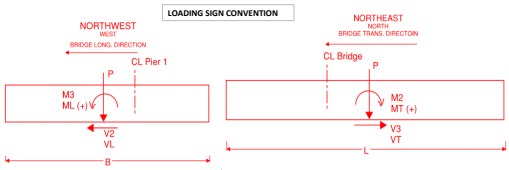
FOOTING BEARING RESISTANCE, Φq_n (KSF)			
Footing Width (Ft)	Footing Length (Ft)		Extreme
ANY	ANY		**12.0**

From Geotech Report

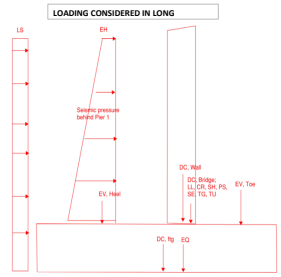
Step 2: a) Demands (WHOLE STRUCTURE +LIVE LOAD AND SUPER IMPOSED LOADS)

Average Height of pier wall, H_{abut}=	19.50	ft		Designed
Height of Abutment+footing, $H_{abut+footing}$=	23.50	ft	=Habut + Df	Added footing height
Height of Superstructure, H_s=	4.25	ft		
Total Height of Abutment, H=	27.75	ft		Wall + Footing + Superstructure
Width of the abutment stem at base, W_{abut}=	3.0	ft		Designed
Width of the Toe @ abutment, W_t=	9.7	ft		Designed
Width of the Heel @ abutment, W_h=	12.0	ft		Designed





V2= VL: Shear in longitudinal direction
V3= VT: Shear in transverse direction
M2= MT: Moment about stong axis
M3= ML: Moment about weak axis



Load modifications to account for eccentricities CL Footing and CL Superstructure

Eccentricity between CL Superstructure and CL Footing in longitudinal direction =	-1.75	ft	-ve means towards east of footing CL
Eccentricity between CL Superstructure and CL Footing in transverse direction =	-4.54	ft	-ve means towards north of footing CL
Eccentricity between CL wall and CL Footing in longitudinal direction =	-1.27	ft	-ve means towards east of footing CL
Eccentricity between CL wall and CL Footing in transverse direction =	-4.54	ft	-ve means towards north of footing CL

Notes:
1. Values in orange shade below are the final values used for design check
2. The moment demands in orange shade below also include moment from shear at base of wall to centre of footing and eccentric offsets

	P	V2	V3		M2	M3
	Kip	Kip	Kip		Kip-ft	Kip-ft
DC, supr	510	7	4	0	1190	-139
DC, supr	0	0	0		-2316	-893
DC, Supr	510	7	4		-1118	-1018
DC, wall	340	0	0		-1544	-432
CR&SH	55	-13	-4	0	-410	250
CR&SH	0	0	0		-250	-96
CR&SH,	55	-13	-4		-668	128
LL MAX	-21	-148	-67	0	-1338	-3216
LL MAX	0	0	0		95	37
LL MIN,	-21	-148	-67		-1377	-3475
LL MAX	169	164	28	0	1573	2397
LL MAX	0	0	0		-768	-296
LL MAX,	169	164	28		861	2429
PS	16	-14	-5	0	-64	279
PS	0	0	0		-73	-28
PS,	16	-14	-5		-147	223
SE	-13	-5	0	0	-109	98
SE	0	0	0		59	23
SE,	-13	-5	0		-50	111
TG-	-15	-3	2	0	-55	52
TG-	0	0	0		68	26
TG-,	-15	-3	2		17	-72
TG+	51	9	-7	0	183	-174
TG+	0	0	0		-232	-89
TG+,	51	9	-7		-63	-245
TU-	-7	-125	-16	0	211	755
TU-	0	0	0		32	12
TU-,	-7	-125	-16		211	517
TU+	5	0	10	0	-132	7
TU+	0	0	0		-23	-9
TU+,	5	0	10		-135	-2
LS_Pier1	-1	-26	-1		-7	-157
LS_Pier1	0	0	0		5	2
LS_Pier1	-1	-26	-1		-4	-207
LEP_Pier1	-12	-813	-9		-73	-3573
LEP_Pier1	0	0	0		55	21
LEP_Pier1	-12	-813	-9		-37	-5178
1.0X+0.3Y+0.3Z	364	-353	-633		-15027	-6613
1.0X+0.3Y+0.3Z	0	0	0		-1653	-463
1.0X+0.3Y+0.3Z	364	-353	-633		-17946	-7782
0.3X+1.0Y+0.3Z	220	-230	-995		-24351	-4319
0.3X+1.0Y+0.3Z	0	0	0		-999	-280
0.3X+1.0Y+0.3Z	220	-230	-995		-27340	-5059

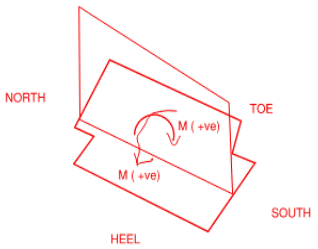
LS behind Pier 1 already includes 0.5 factor

Use 30% of LEP values when combining with 100% Transverse loads from EQ (RSA)

EQ (RSA)

EQ (RSA)

V2= VL: Shear in longitudinal direction
V3= VT: Shear in transverse direction
M2= MT: Moment about stong axis
M3= ML: Moment about weak axis



For 100% Longitudinal Case the following overstrength V2 and M3 values shall be used to replace the values from combination of LEP_Pier1 + LS_Pier1 + EQ(RSA)

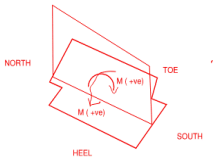
V2	M3
Kip	Kip-ft
-1030	-20094

For 30% Longitudinal Case, these values obtained from RSA are to be used in combination with 30% LEP_Pier1 and LS_Pier1 with a 1.2 factor

Earth Loads: (EV)

Vertical EV:

Dead Load, Evheel=	1624	kips	includes the soil behind superstructure too	Using actual footing dimension = 0.130 x (23.75+2)ft x 485 sq. ft
Moment Arm from CL FOOTING=	6.3	ft		
Moment,M_{EV} heel=	10282	k-ft		
Dead Load, E_{vtoe}=	99.84	kips	Assuming 2ft of soil above toe (below I405 level)	Using actual footing dimension = 0.130 x 2ft x 384 sq. ft= 100 kips
Moment Arm from CL FOOTING=	-7.5	ft		
Moment at Point O,M_{Evtoe}=	-749	k-ft	Heel Stabilizing moment (M-ve)	

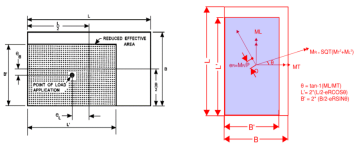


Net Moment due to EV about CL Footing, M(ev)L= 9534 k-ft

Weight of footing:

Axial load of footing, P= 592 kips
Moment due to footing forces,M_f= 0 k-ft

Step 3: Bearing Stress about footing center (considering effective footing Area):

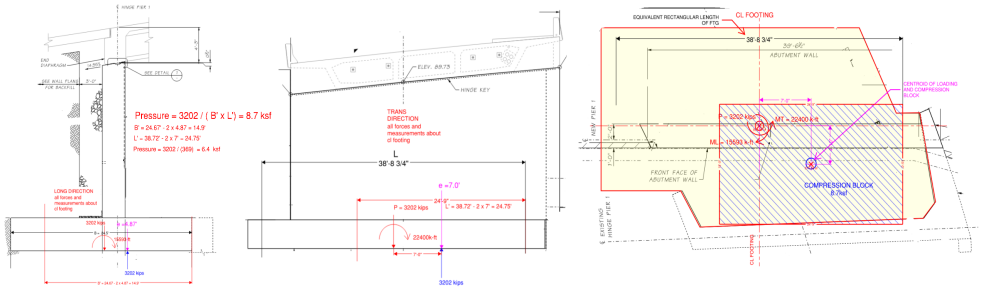


Bearing Stress about footing center (considering effective footing Area):

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 1)	TU, TG, SE	LS,H	EQ (RSA) or EQ (Overstrength)	Factored Axial Load,P (kips)	Factored Moments,M_L Longitudinal Direction (k-ft)	Factored Moments,M_T Transverse Direction (k-ft)	Factored Effective Moments, M_R(k-ft)	Eccentricity $e_O=M_R/P$ (FT)	Resultant angle,θ=Atan (M_L/M_T)	Effective Length, L' (ft)	Effective Width, B' (ft)	Maximum Soil Stress, σ (KSF)	Factored Soil Bearing Resistance, Φq_n	C/D	
Extreme I Load Case: -100%T - 30% L + 30% Z	1	0.5	1	1.2	1	1	1.2	3518	-1581	-30675	30715	9	0.051	21	23.77	**7.0**	12	1.7	**StoN for 100%T (Pressure on Toe, No Tie-in resistance required)**
Extreme I Load Case: -100%T - 30% L - 30% Z	1	0.5	1	1.2	1	1	1.2	2990	-1581	-30675	30715	10	0.051	18	24	**7.0**	12	1.7	
Extreme I Load Case: 100%T - 30% L + 30% Z	1	0.5	1	1.2	1	1	1.2	3440	-744	23312	23324	7	-0.032	25	24	**6**	12	2.1	**NtoS for 100%T (Pressure on Toe, No Tie-in resistance required)**
Extreme I Load Case: 100%T - 30% L - 30% Z	1	0.5	1	1.2	1	1	1.2	2924	-1263	22966	23001	8	-0.055	23.0	23.8	**5**	12	2.2	
Extreme I Load Case: +30%Trans 100% Long + 30% Vertical	1	0.5	1	0	1	1	1	3363	-15593	15037	21662	6	-0.804	30	15	**7**	12	1.6	**NtoS for 30%T (Pressure on Toe side)**
Extreme I Load Case: +30%Trans 100% Long - 30% Vertical	1	0.5	1	0	1	1	1	3202	-15593	15037	21662	7	-0.804	29	15	**7**	12	1.6	
Extreme I Load Case:-30%Trans - 100% Long + 30% Vertical	1	0.5	1	0	1	1	1	3363	-15593	-22400	27293	8	0.608	25	15	**8.6**	12	1.4	**StoN for 30%T (Pressure on Toe side)**
Extreme I Load Case: -30%Trans - 100% Long - 30% Vertical	1	0.5	1	0	1	1	1	3202	-15593	-22400	27293	9	0.608	25	15	**8.7**	12	1.4	

Notes:
1. Earthquake demands EQ and EH from lateral pressure behind Pier 1 are used with a factor of 1.2 as per WSDOT BDM 15.4.2 B for 30% Longitudinal Case only
2. Earthquake demands for 100% Longitudinal case are limited to overstrength forces of wall (moment and shear) and are combined EDA demands from 30%T direction without using 1.2 factor
3. All EQ forces in transverse direction (Wall strong direction) are used without 1.2 factor

Check equilibrium of critical case: S to N Trans EQ (100%T - 30%L)



c) Failure By Sliding: WSDOT BDM 7.7.4.C

Effective angle of internal friction, Φ'f= 38 degrees From geotech report

TAN δ=TAN Φ'f= 0.78 (For cast-in place concrete against soil)

Note: We are conservatively ignoring any passive resistance from surrounding soil for sliding resistance

$$Q_R = \phi_T Q_T + \phi_{ep} Q_{ep}$$

x Resistance Factor (0.9 for the extreme case)

AASHTO BDS 3.4.1-1&2

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 1)	TU, TG, SE	LS,H	EQ (RSA)	Factored Horizontal loads, V2	Factored Horizontal loads, V3	SRSS V2 and V3	Factored sliding resistance, Q_R (kips)	C/D
100%L + 30%T	EXT	0.9	0	0.9	1.2	1	1	1.2	-1176	-661	1349	1706	**1.3**
100%T + 30%L	EXT	0.9	0	0.9	1.2	1	1	1.2	-741	-1032	1270	1860	**1.5**

Table 7.7.2-1 Load Factors

Sliding and Overturning, ϕ_s	Bearing Stress (ϕ_b, ϕ_{ep})
	LL_{MAX} = 0
DC_{min}, DW_{min} for resisting forces, DC_{max}, DW_{max} for causing forces.	DC_{max}, DW_{max} for causing forces DC_{min}, DW_{min} for resisting forces
EV_{RES}	EV_{MAX}
EH_{MAX}	EH_{MIN}
LS	LS

d) Overturning Stability: LONGITUDNAL DIRECTION, ABOUT FOOTING WIDTH

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 1)	TU, TG, SE	LS,H	EQ (RSA)	Factored Axial Load,P (kips)	Factored Moments,M_o Longitudinal Direction (k-ft)	Eccentricity Xo=Mo/P (FT)
100%L + 30%T	EXT	0.9	0	0.9	1.2	1	0	1.2	2505	-14699	-5.87

Eccentricity Limits (100%L + 30%T): WSDOT BDM 7.7.4.D/AASHTO 11.6.3.3; AASHTO BDS 11.6.3.3

EXTREME CASE
Resultant force shall be within the middle 2/3 of the base width for LL=0.0 AASHTO BDS 11.6.5

Lower Limit= 4.111 ft
Upper Limit= 20.56 ft
(Extreme Case) B/2-Xo= 6.46 ft **OK within 2/3 limit**
(Extreme Case) B/2+Xo= 18.20 ft

d) Overturning Stability: TRANSVERSE DIRECTION, ABOUT FOOTING LENGTH

LOAD CASE	DC, CR SH, PS	LL	Ev	EH (LEP behind Pier 1)	TU, TG, SE	LS,H	EQ (RSA)	Factored Axial Load,P (kips)	Factored Moments,M_o Trans Direction (k-ft)	Eccentricity Xo=Mo/P (FT)
100%T + 30%L	EXT	0.9	0	0.9	1	1	0	1	2658	-30674	11.54

Eccentricity Limits (100%L + 30%T): WSDOT BDM 7.7.4.D/AASHTO 11.6.3.3; AASHTO BDS 11.6.3.3

EXTREME CASE
Resultant force shall be within the middle 2/3 of the base width for LL=0 AASHTO BDS 11.6.5

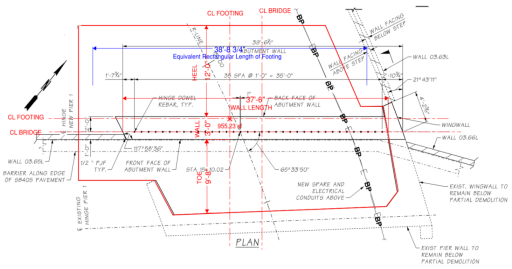
Lower Limit= 6.5 ft
Upper Limit= 32.3 ft
(Extreme Case) L/2-Xo= 7.8 ft **OK within 2/3 limit**
(Extreme Case) L/2+Xo= 30.9 **OK within 2/3 limit**

PIER 1 - FOOTING SIZING
(SER and STR CASE)

COWI	PROJECT BR 17.7					CONT						A207833
	SUBJECT Pier 1 Abutment Footing Sizing STR/SER LOADS					PAGE						
CHECKED BY MWBM	DATE 2021-11-15	CALCS BY RSGR		DATE 2021-11-15								

Purpose:
Check bearing, sliding and overturning of the footing
Notes:
- The wall is assumed to be pinned at top and fixed at bottom in bridge longitudnal direction. Hence, moment due to lateral loads in long. direction are less than a pure cantilevered wall case.
- The wall is to be treated as cantilevered in transverse direction
- Load inputs for DL, LL, CR, SH, PS, SE, TU, TG are from the CSI model at the base of wall and the shears are multiplied by half the footing depth to be added to wall base moments
- Assumed there is no friction between back of wall and fill
- DL has been maximized in bearing check and minimized in sliding/eccentricity check as per AASHTO combos
- The new footing is tied to existing pier wall. The exisitng wall is not relied upon for any resistance that would help in bearing, overturning or sliding for STR and SER case checks

User input=				
Manual Load input=				
Unit Weight of Soil,γ_{soil}=	130	pcf		**References** Geotech info
Width of Wall,L_w=	37.50	ft		
Length of Footing,L_f=	38.72	ft		Length of footing is measured in transverse direction; this value is chosen such that L X B is same as actual area of footing
Width of Footing,B_f=	24.67	ft	=toe+heel+wall	Calculated
Depth of Footing, D_f=	4	ft		Designed
Concrete unit weight =	155	pcf		



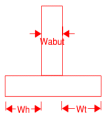
Step 1: Bearing Capacities
a) Bearing Capacities

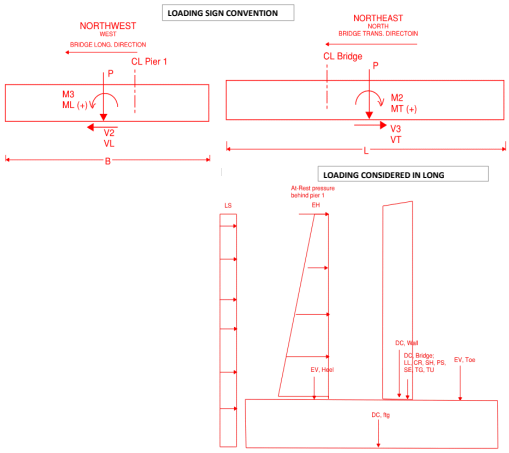
FOOTING BEARING RESISTANCE,Φq_n (KSF)			
Footing Width (Ft)	Footing Length (Ft)	SER	STR
ANY	ANY	**5.4**	**7.0**

From Geotech Report

Step 2: a) Demands (WHOLE STRUCTURE +LIVE LOAD AND SUPER IMPOSED LOADS)

Average Height of pier wall,H_{abut}=	19.50	ft		Average height of wall
Height of Abutment+footing,$H_{abut+footing}$=	23.50	ft	=Habut + Df	Added footing height
Height of Superstructure, H_s=	4.25	ft		
Total Height of Abutment, H=	27.75	ft		Wall + Footing + Superstructure
Width of the abutment stem at base, W_{abut}=	3.0	ft		Designed
Width of the Toe @ abutment, W_t=	9.7	ft		Designed
Width of the Heel @ abutment, W_h=	12.0	ft		Designed





V2= VL: Shear in longitudinal direction
V3= VT: Shear in transverse direction
M2= MT: Moment about stong axis
M3= ML: Moment about weak axis

Load modifications to account for eccentricities CL Footing and CL Superstructure

Eccentricity between CL Superstructure and CL Footing in longitudinal direction =	-1.75	ft	-ve means towards east of footing CL
Eccentricity between CL Superstructure and CL Footing in transverse direction =	-4.54	ft	-ve means towards north of footing CL
Eccentricity between CL wall and CL Footing in longitudinal direction =	-1.27	ft	-ve means towards east of footing CL
Eccentricity between CL wall and CL Footing in transverse direction =	-4.54	ft	-ve means towards north of footing CL

Notes:
1. Values in orange shade below are the final values used for design check
2. The moment demands in orange shade below also include moment from shear at base of wall to centre of footing and eccentric offsets

	P	V2	V3	M2	M3
	Kip	Kip	Kip	Kip-ft	Kip-ft
DC, supr	510	7	4	1190	-139
DC, supr	0	0	0	-2316	-893
DC, Supr	510	7	4	-1118	-1018
DC, wall	340	0	0	-1544	-432
CR&SH	55	-13	-4	-410	250
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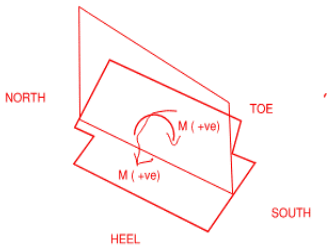
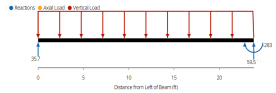
LL MAX	-21	-148	-67	-1338	-3216
LL MAX	0	0	0	95	37
LL MIN,	-21	-148	-67	-1377	-3475
LL MAX	169	164	28	1573	2397
LL MAX	0	0	0	-768	-296
LL MAX,	169	164	28	861	2429
PS	16	-14	-5	-64	279
PS	0	0	0	-73	-28
PS,	16	-14	-5	-147	223
SE	-13	-5	0	-109	98
SE	0	0	0	59	23
SE,	-13	-5	0	-50	111
TG-	-15	-3	2	-55	52
TG-	0	0	0	68	26
TG- ,	-15	-3	2	17	72
TG+	51	9	-7	183	-174
TG+	0	0	0	-232	-89
TG+,	51	9	-7	-63	-245
TU-	-7	-125	-16	211	755
TU-	0	0	0	32	12
TU-,	-7	-125	-16	211	517
TU+	5	0	10	-132	7
TU+	0	0	0	-23	-9
TU+,	5	0	10	-135	-2
WL	0	3	22	502	60
WL	0	0	0	0	0
WL	0	3	22	546	66
WS (SER)	0	18	9	196	295
WS (SER)	0	0	0	0	0
WS (SER)	0	18	9	214	331
WS (STRIII)	16	32	19	654	520
WS (STRIII)	0	0	0	-73	-28
WS (STRIII)	16	32	19	619	556
WS (STRV)	1	24	11	258	385
WS (STRV)	0	0	0	0	0
WS(STRV)	1	24	11	280	433

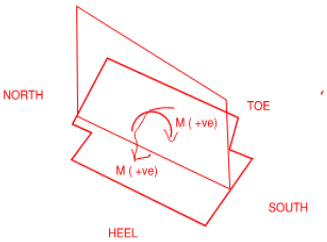
V2= VL: Shear in longitudinal direction
V3= VT: Shear in transverse direction
M2= MT: Moment about stong axis
M3= ML: Moment about weak axis



Live Load Surcharge (LS) Pier 1:
Not to be used with a combination that reduces pressure on heel

k	0.38			lateral earth pressure coefficient; from geotech info
γ_s	130	pcf		unit weight of soil
h_{eq}	2	ft		Table 3.11.6.4-1 - AASHTO LRFD
LS=	0.099	ksf		
LS Force=	4	k/ft	LS x width of wall	
Moment at Point O, M_{LS}=	-285	k-ft	Assuming Pin top -Fix bottom	Heel Resisting moment (M-ve)
Horizontal Reaction Top of wall	-36	kips		
Horizontal Reaction bottom of footing	-60	kips		





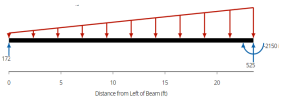
Earth Loads: (EV)

Vertical,EV:

Dead Load, Evheel=	1634	kips	includes the soil behind superstructure too	Using actual footing dimension = 0.130 x (23.75+2)ft x 488 sq. ft
Moment Arm from CL FOOTING=	6.3	ft		
Moment, M_{Ev} heel=	10346	k-ft		
Dead Load, E_{vtoe}=	99.84	kips	Assuming 2ft of soil above toe (below i405 level)	Using actual footing dimension = 0.130 x 2ft x 384 sq. ft= 100 kips
Moment Arm from CL FOOTING=	-7.5	ft		
Moment at Point O, M_{Evtoe}=	-749	k-ft	Heel Stabilizing moment (M-ve)	
Net Moment due to EV about CL Footing, M(ev)L=	9597	k-ft		

Horizontal,EV:
At Rest EP Lateral,EH:

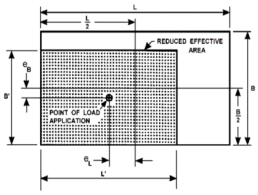
Effective angle of internal friction, Φ'=	38	degrees		WSDOT Gravel Borrow
Angle of Fill, β=	0	degrees		AASHTO BDS 3.11.5.3
Angle of back face of wall to horizontal, θ=	90	degrees		AASHTO BDS 3.11.5.3
Friction angle between fill and wall, δ=0.33Φ'=	0	degrees		AASHTO BDS C3.11.5.3; but considered 0 for actual condit
Coefficient of lateral Earth pressure, at rest = K_o=1-sinΦ'=	0.384			AASHTO BDS Eq. 3.11.5.2-1; At Rest
Γ= (1+√((sin(Φ'+δ)*sin(Φ'-β))/((sin(θ-δ)*sin(θ+β))^2	2.610			AASHTO BDS Eq. 3.11.5.3-2
pressure=K =(sin^2(θ+Φ'-)/(Γ(sin^2θsin(θ-δ))	0.238			AASHTO BDS Eq. 3.11.5.3-1 ; For comparison
Ko at rest (Geotech information)=	0.380			From Geotech
Coefficient of passive lateral Earth pressure=K_p=	4.569			AASHTO BDS Fig. 3.11.5.4-1
Total depth of soil, z/H=	27.750	ft	UDL	
Lateral Earth Pressure top of wall , pt=	0.210	ksf	7.87 k/ft	Pressure at top of the wall is taken at top of end diaphragm
Lateral Earth Pressure bottom of footing, pb=	1.37	ksf	51.41 k/ft	Pressure at bottom of footing which is 27' below ground
Lateral Earth force,P=	-697	kips		Force on the wall and footing; trapezoidal
Moment at Point x, M_{Oth}=	-2150	k-ft		Assuming Pin at top and fixed at base,

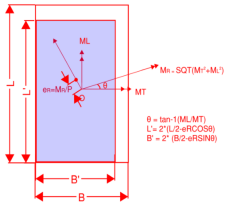


Weight of footing:

Axial load of footing, P=	592	kips	
Moment due to footing forces, M_f=	0	k-ft	

Step 3: Bearing Stress about footing center (considering effective footing Area):





Bearing Stress about footing center (considering effective footing Area):

LOAD CASE	DC	LL	Ev	EH	TU = 0.5; TG = 0.5 SER, 0 STR; SE = 1.0	LS,H	WL	WS	CR SH, PS	Factored Axial Load,P (kips)	Factored Moments,Mt, Longitudinal Direction (k-ft)	Factored Moments,ML, Transverse Direction (k-ft)	Factored Effective Moments, Mn(k-ft)	Eccentricity eo=Mn/P (FT)	Resultant angle,θ=Ata n(ML/Mt)	Effective Length, L' (ft)	Effective Width, B' (ft)	Maximum Soil Stress, σ (KSF)	Factored Soil Bearing Resistance, ΦqR	C/D
SER	1	1	1	1.35	γ	0	1	1	0.5	3166	8652	-5355	10175	3	-1.017	35	19	**4.7**	5.4	1.2
STR I	1.25	1.75	1.35	1.35	γ	0	0	0	0.5	4125	13038	-6262	14463	4	-1.123	36	18	**6.3**	7.0	1.1
STR III	1.25	0	1.35	1.35	γ	0	0	1	0.5	4138	9342	-4472	10358	2.5	-1.124	36.55	20.2	**5.6**	7.0	1.2
STR V	1.25	1.35	1.35	1.35	γ	0	1	1	0.5	4125	12565	-6537	14164	3.4	-1.091	35.5	18.6	**6.2**	7.0	1.1

c) Failure By Sliding:

Effective angle of internal friction, Φ'f,= 38 degrees — From geotech report
TAN δ=TAN Φ'f,= 0.78 — (For cast-in place concrete against soil)

WSDOT BDM 7.7.4.C

Note: We are conservatively ignoring any passive resistance from surrounding soil for sliding resistance

$$Q_R = \phi_T Q_T + \phi_{ep} Q_{ep}$$

x Resistance Factor
(0.8 for the STR and SER case)

AASHTO BDS 3.4.1-1&2 and WSDOT BDM Table 7.7.2-1

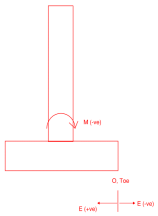
LOAD CASE	DC	LL	Ev	EH	TU = 0.5; TG = 0.5 SER, 0 STR; SE = 1.0	LS,H	WL	WS	CR SH, PS	Factored Horizontal loads, V2	Factored Horizontal loads, V3	SRSS V2 and V3	Factored sliding resistance, Q_R (kips)	C/D
SER	0.9	0	0.9	1	γ	1	1	1	0.5	-682	-39	683	1794	**2.6**
STR I	0.9	0	0.9	1.35	γ	1.75	0	0	0.5	-888	-9	888	1798	**2.0**
STR III	0.9	0	0.9	1.35	γ	0	0	1	0.5	-815	-28	816	1788	**2.2**
STR V	0.9	0	0.9	1.35	γ	1.35	1	1	0.5	-891	-42	892	1798	**2.0**

Table 7.7.2-1 Load Factors

Sliding and Overturning, ϕ_s	Bearing Stress (ϕ_b, ϕ_s)
	LL_{max} = 0
DC_{max}, DW_{max} for resisting forces, DC_{min}, DW_{min} for causing forces,	DC_{max}, DW_{max} for causing forces DC_{min}, DW_{min} for resisting forces
EV_{min}	EV_{max}
EH_{min}	EH_{max}
LS	LS

d) Overturning Stability: LONGITUDNAL DIRECTION, ABOUT FOOTING WIDTH - Check this about the Toe

LOAD CASE	DC	LL	Ev	EH	TU = 0.5; TG = 0.5 SER, 0 STR; SE = 1.0	LS,H	WL	WS	CR SH, PS	Factored Axial Load,P (kips)	Factored Moments,Mo Longitudinal Direction (k-ft)	Eccentricity Xo=Mo/P (FT)
SER	0.9	0	0.9	1	γ	1	1	1	0.5	2908	4663	1.60
STR I	0.9	0	0.9	1.35	γ	1.75	0	0	0.5	2883	4217	1.46
STR III	0.9	0	0.9	1.35	γ	0	0	1	0.5	2867	4160	1.45
STR V	0.9	0	0.9	1.35	γ	1.35	1	1	0.5	2882	3832	1.33



Eccentricity Limits

Resultant force shall be within the middle 2/3 of the base width
Lower Limit= 4.111 ft
Upper Limit= 20.56 ft
B/2+Xo= 13.94 ft — **OK within 2/3 limit**
B/2-Xo= 10.73 ft — **OK within 2/3 limit**

d) Overturning Stability: TRANSVERSE DIRECTION, ABOUT FOOTING LENGTH

LOAD CASE	DC	LL	Ev	EH	TU = 0.5; TG = 0.5 SER, 0 STR; SE = 1.0	LS,H	WL	WS	CR SH, PS	Factored Axial Load,P (kips)	Factored Moments,Mo Trans Direction (k-ft)	Eccentricity Xo=Mo/P (FT)
SER	0.9	0	0.9	1	γ	1	1	1	0.5	2908	-3712	1.28
STR I	0.9	0	0.9	1.35	γ	1.75	0	0	0.5	2883	-2921	1.01
STR III	0.9	0	0.9	1.35	γ	0	0	1	0.5	2867	-3540	1.23
STR V	0.9	0	0.9	1.35	γ	1.35	1	1	0.5	2882	-3747	1.30

Eccentricity Limits

Resultant force shall be within the middle 2/3 of the base width for LL=0
Lower Limit= 6.453 ft
Upper Limit= 32.26 ft
L/2-Xo= 18.06 ft — **OK within 2/3 limit**
L/2+Xo= 20.66 — **OK within 2/3 limit**

EXISTING PIER 1 DESIGN CHECKS

(For geotechnical check - Refer to Book I10 Page 29 to 34)

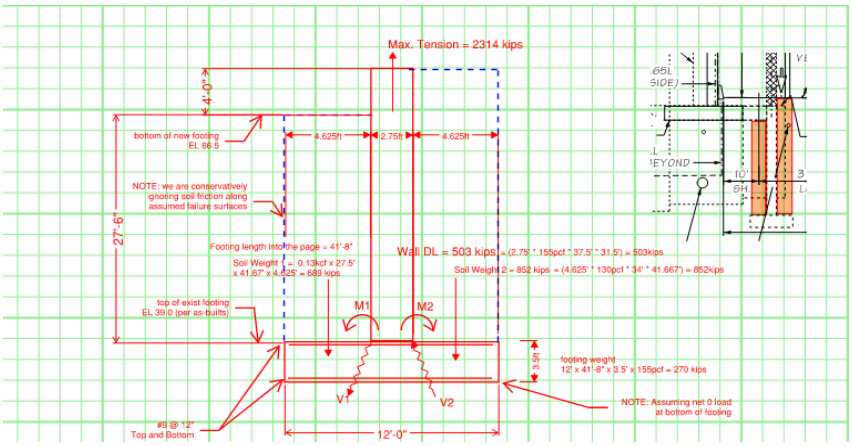
For Structural check - Refer to the following pages

Aim : To check the existing pier wall footing structurally for the loads induced by new pier 1 footing for EXT case

Assumptions/Methodologies:
1. The new pier 1 footing is designed to pull maximum self weight of existing wall and footing + weight of soil over the footing to resist overturning moments.
2. Another case is checked where maximum compression from the new pier 1 footing is transferred entirely to the existing pier footing assuming there is no soil under the new footing.

Case 1 **Tension from New footing**

LOADING



V1 = (wt. of soil + 0.5 footing DL) = 852 + 0.5x270 = 987kips / wall length 37.5' = 26k/ft

V2 = (wt. of soil + 0.5 footing DL) = 689 + 0.5x270 = 924kips / wall length 37.5' = 22k/ft

M2 = (wt. of soil + 0.5 footing DL) x (footing width under soil x 0.5)=
(852 + 0.5 x 270) x (0.5 x 4.625') = 2282 k/ft
Over wall length = 2282/37.5 = 61 k-ft/ft

M1 = (wt. of soil + 0.5 footing DL) x (footing width under soil x 0.5)=
(689 + 0.5 x 270) x (0.5 x 4.625') = 1905 k/ft
Over wall length = 1905/37.5 = 51 k-ft/ft

Design Moment	Mu=	61	k-ft/ft
Design Shear	Vu=	26	k/ft

These are resisted by top bar

input

DIMENSIONS & PARAMETERS

f'ce	5.20	ksi	F'c = 4ksi used in shear	Expected concrete compressive strength
wc =	0.155	kcf		Unit weight of weight concrete
K1 =	1			Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4592	ksi		Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
f'ye =	68	ksi		Expected Yield strength of reinforcement
Es =	29000	ksi		Modulus of elasticity of reinforcement
η =	6.32			Modular ratio
β_1 =	0.85			Compression Zone Factor
φf=	1			EXT case flexural resistance factor
φv=	0.9			factor for shear resistance
Design Width, b=	1	ft		design for unit width
Footing Depth=Df=	3.5	ft		

FLEXURE

Cover top bar, ct=	2	in	AASHTO Table 5.10.1-1
Cover bottom bar, cb=	3	in	
bar size=	9	#	
spacing=	12	in	
bar diam=	1.128	in	
bar area=	1.000	in2	
Effective depth, de=	39.436	in	Main bar is outside layer
Area of steel per unit width, As=	1.000	in2/ft	
distance compression fibre to NA, Cb=	1.508	in	$C_b = \frac{A_s f_y}{\beta_1 0.85 f_c' b}$
depth of equiv stress block, a=	1.282	in	$a = \beta_1 C_b$
Nominal Flexural Resistance, Mn=	220	k-ft/ft	$M_n = A_s f_y \left(d_e - \frac{a}{2}\right)$: AASHTO 5.6.3.3
Factored Flexural Resistance, φMn=	220	k-ft/ft	Demand<Capacity OK
Demand=	61	k-ft/ft	

SHEAR - Concrete only

shear load Demand, Vu=	26	kips/ft	
Effective depth, dv=	38.6	in	AASHTO 5.7.2.8
tensile strain in section, ε_s=	0.001551	in/in	AASHTO 5.7.3.4.2-4
crack spacing parameter (1), sx=	12.0	in	
crack spacing parameter (1), sxe=	12.0	in	assumed aggregate size
shear resistance parameter, β=	2.21891		AASHTO 5.7.3.4.2-2
Concrete modification factor, λ=	1		AASHTO 5.4.2.8
Nominal Shear Resistance, Vc=	65	kips/ft	AASHTO 5.7.3.3-3
Factored Shear Resistance, Vr=φVc	58	kips/ft	OK

Check Tension Capacity of Wall

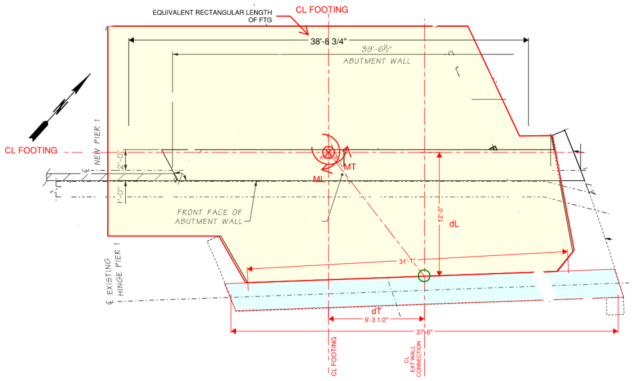
Qty. Main Wall vertical rebar=	80	(37 rebar each face + 3 each side) x 2 faces and sides
Rebar size=	10	#
Area of all rebars, As=	101.6	in2
Expected yield stress, f'ye=	68	ksi
Max. Tensile Capacity, Ti=	6909	kips
Max. Tensile Demand, Td=	2314	kips

Case 2 Compression from New footing

LOADING:

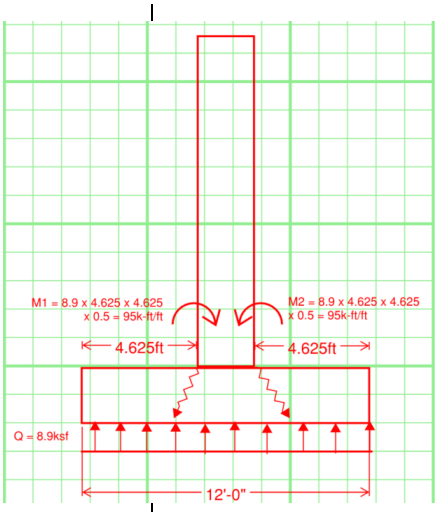
Max transverse and longitudinal moment from new footing divide by moment arms between centroid of new footing and edge of existing wall

CONSERVATIVELY IGNORING SOIL UNDER NEW FOOTING



Compression Force into existing wall

Case	MT	ML	dT	dL	P
	k-ft	k-ft	ft	ft	kips
EXT	36567	0	9.291667	13.39583	3935
	30675	1581	9.291667	13.39583	3419
	22400	15593	9.291667	13.39583	3575



Max. Axial Load from new footing, P=	3935	kips	
Max. self weight of existing wall, p=	503	kips	
Existing Footing Width, B=	12	ft	
Existing Footing Length, L=	41.67	ft	
Max pressure under existing footing, Q=	8.9	ksf	=(P+p)/(BxL)
Moment demand at wall/footing interface, M=	190	k-ft/ft	= M1 + M2
Shear demand at wall/footing interface, V=	27.2	k/ft	=V1; Q x (4.625ft-Dv/2)

FLEXURE

Cover top bar, ct=	2	in
Cover bottom bar, cb=	3	in
bar size=	9	#
spacing=	12	in
bar diam=	1.128	in
bar area=	1.000	in2
Effective depth, de=	38.436	in
Area of steel per unit width, As=	1.000	in2/ft
distance compression fibre to NA, Cb=	1.508	in
depth of equiv stress block, a=	1.282	in
Nominal Flexural Resistance, Mn=	214	k-ft/ft
Factored Flexural Resistance, φMn=	214	k-ft/ft
Demand=	190	k-ft/ft

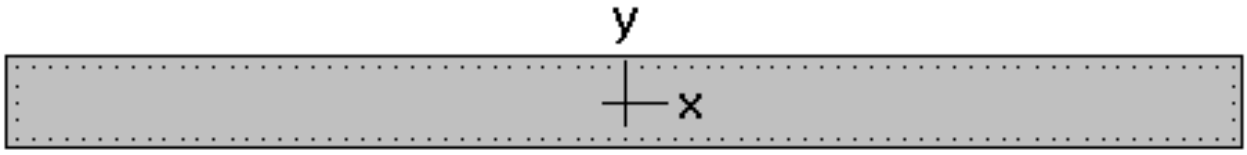
Demand<Capacity OK

SHEAR - Concrete only - Dv/2 away from wall/footing interface

shear load Demand, Vu=	27	kips/ft	
Effective depth, dv=	37.6	in	
tensile strain in section, ε_s=	0.003027	in/in	
crack spacing parameter (1), sx=	12.0	in	
crack spacing parameter (1), sxe=	12.0	in	
shear resistance parameter, β=	1.467685		
Concrete modification factor, λ=	1		
Nominal Shear Resistance, Vc=	42	kips/ft	
Factored Shear Resistance, Vr=φVc	38	kips/ft	OK

PIER 1 : WALL REINFORCEMENT DESIGN

WALL IS DESIGNED 36" THICK (WITH 1.5" SINE WAVE PATTERN ON THE FRONT FACE). HENCE, THE DESIGN THICKNESS OF WALL IS 34.5'"



REINFORCEMENT:
==========
156 #10 bars @ 1.276%
As = 198.12 in^2
Confinement: Tied
Clear Cover = 3.25 in
Min Clear Spacing = 4.70 in

COWI

PROJECT:	I-405 R2B Final Design - BR 17.7	**JOB NO.:**	A207833
SUBJECT:	Pier 1 Wall Reinforcement (EXT CASE)		
BY:	RSGR	**DATE:**	2021/11/29
CHK:	MWBM	**DATE:**	2021/11/29

Purpose:
Design the reinforcement for the wall of Pier 1 to EXT loads under FEE spectra. The wall resists these loads EQ loads elastically. Note that **this check is not required contractually** and is being carried out as a reasonable date point. Note that the demands from FEE case are 75% higher than STR case. The wall is designed to form a plastic hinge at the base for SEE level earthuake. Demand Mpo is 20094k-ft and Demand 1.2 x SEE (EDA) + 1.2 x LEP behind Pier 3 = 1.2 x 12599 + 1.2 x 6790 = 23267 k-ft. Wall top has a high shear due to passive pressure under SEE level EtoW EQ and is checked as well.

Asumptions:
1. Loads are extracted from the bridge FEM model and are combined as per appropriate load combinations.
2. Critical section occurs near the base of the wall for flexure.
3. Wall shear is taken at a distance Dv away from the base of the wall.
4. All shear ties are staggered as per WSDOT BDM Figure 7.5.10-1.

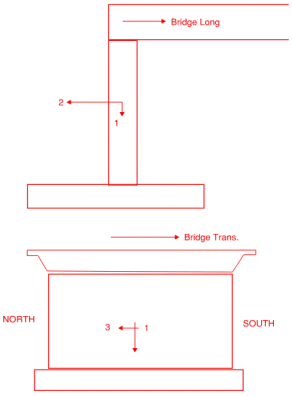
SUMMARY

	bar	Nos.	spacing	D/C
Wall main vertical reinforcement	10	156	6	87%
Wall main horizontal reinforcement	9	79	6	-
Wall Shear Ties Plastic Zone	5	360	6	18%
Wall Shear Ties outside Plastic Zone	5	540	12	92%

Notes and References

DIMENSIONS & PARAMETERS

	input		
f'c	4.0	ksi	concrete compressive strength
wc =	0.2	kcf	Unit weight of weight concrete
K1 =	1.0		Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4027.6	ksi	Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
f'y =	60.0	ksi	Yield strength of reinforcement
Es =	29000.0	ksi	Modulus of elasticity of reinforcement
η =	7.2		Modular ratio
β_1 =	0.9		Compression Zone Factor
φf=	1.0		EXT case flexural resistance factor
φv=	0.9		factor for shear resistance
Heel Length, HL=	12.0	ft	
Toe Length, TL=	9.7	ft	
Total footing Length=	38.7	ft	
Total Wall Length=	37.5	ft	
Stem Width at design section, D=	2.875	ft	
Footing Depth=Df=	4.0	ft	
Area of wall=	15525.0	in2	
Area of main tensile reinforcement=	99.1	in2	78 #10 rebar
Area of total reinforcement=	198.1	in2	Meets max. and min limits for compression member (provided = 1.28%)



LOADING ON THE WALL (Refer to Footing Sizing calcs)

		P	V2	V3	T	M2	M3	
		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
	DC, Supr	510	7	4	-15	1190	-139	
	DC, Supr	510	7	4	-15	-1126	84	
	DC, Wall	340	0	0	0	0	0	
	CR&SH	55	-13	-4	28	-410	250	
	PS	16	-14	-5	18	-64	279	
Max	LL							
Min	LL							
Min	LL	169	164	28	56	1573	2397	IM included; BR and CE included
Max	LL	-21	-148	-67	-64	-1338	-3216	IM included; BR and CE included
	LS_Behind_Pier1	0	-60	0	0	0	285	
	LEP_Behind_Pier1	0	-525	0	22	0	2150	
Max	WL	0	3	22	15	502	60	
Min	WL	0	-3	-22	-15	-502	-60	
Max	WS (SER)	0	18	9	9	196	295	
Min	WS (SER)	0	-18	-9	-9	-196	-295	
Max	WS (STR3)	16	32	19	18	654	520	
Min	WS (STR3)	-16	-32	-19	-18	-654	-520	
Max	WS (STR5)	1	24	11	11	258	385	
Min	WS (STR5)	-1	-24	-11	-11	-258	-385	
	SE	-13	-5	0	-1	-109	98	
	TG+	51	9	-7	11	183	-174	
	TG-	-15	-3	2	-3	-55	52	
	TU+	5	0	10	13	-132	7	
	TU-	-7	-125	-16	-11	211	755	

FEE (E to W)							
LEP Pressure Pier 3	58	47	42	-133	308	-6744	
LS at Pier 3	3	2	2	-6	14	-299	
1.0L+0.3T+0.3V (EtoW)	74	32	-179	161	-4165	-5335	-ve M3 Causes tension
1.0L+0.3T-0.3V (EtoW)	-74	32	-179	161	-4165	-5335	on the front face of
0.3L+1.0T+0.3V (EtoW)	69	13	-445	273	-10752	-2142	the wall (face exposed
0.3L+1.0T-0.3V (EtoW)	-69	13	-445	273	-10752	-2142	to i405)

FEE (W to E)							
LEP Pressure Pier 1	-12	-807	9	87	-74	3462	
LS at Pier 1	-1	-26	1	6	-7	148	
1.0L+0.3T+0.3V (WtoE)	120	-129	351	613	8457	2394	+ve M3 Causes tension
1.0L+0.3T-0.3V (WtoE)	-120	-129	351	613	8457	2394	on the back face of
0.3L+1.0T+0.3V (WtoW)	73	-100	435	745	10538	1860	the wall (face exposed
0.3L+1.0T-0.3V (WtoW)	-73	-100	435	745	10538	1860	to soil)

From Design of Front or back face rebar

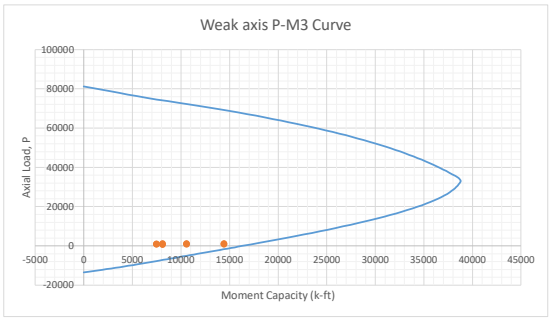
		DC	LL	CR SH PS	EH behind Pier 1	LS behind Pier 1	EH behind Pier 3	LS behind Pier 3	TU	TG	SE	EQ
	Max	1	0.5	1	0	0	1	1	0	0.0	0.0	1.2
EXT EtoW	Min	1	0.5	1	0	0	1	1	0	0.0	0.0	1.2
	Max	1	0.5	1	0	0	1	1	0	0.0	0.0	1.2
EXT EtoW	Min	1	0.5	1	0	0	1	1	0	0.0	0.0	1.2
	Max	1	0.5	1	1	1	1	0	0	0.0	0.0	1.2
EXT WtoE	Min	1	0.5	1	1	1	1	0	0	0.0	0.0	1.2
	Max	1	0.5	1	1	1	1	0	0	0.0	0.0	1.2
EXT WtoE	Min	1	0.5	1	1	1	1	0	0	0.0	0.0	1.2

EXT Cases

		Case	P	M2	M3	V2	V3	T	
EXT EtoW	Max	1	1154	-6944	14441	149	-209	55	100%L+30%T+30%Z
	Min	2	883	-6944	14441	149	-209	55	100%L+30%T-30%Z
EXT EtoW	Max	3	1148	-14849	10609	126	-528	248	30%L+100%T+30%Z
	Min	4	889	-14849	10609	126	-528	248	30%L+100%T-30%Z
EXT WtoE	Max	5	1137	9261	8148	-1056	440	882	100%L+30%T+30%Z
	Min	6	754	9261	8148	-1056	440	882	100%L+30%T-30%Z
EXT WtoE	Max	7	1081	11758	7506	-1021	541	1040	30%L+100%T+30%Z
	Min	8	810	11758	7506	-1021	541	1040	30%L+100%T-30%Z

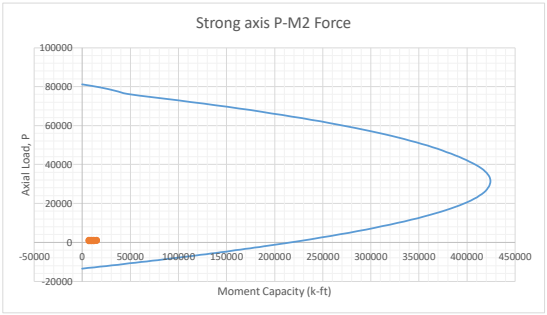
Axial - Flexure Design EXT Check

The P-M3 interaction curve is as follows with cases 1 to 8 plotted
f'ye = 68 ksi; f'c = 5.2 ksi, phi = 1.0
#10 rebar at 6" SPA. Throughout



Case	M3 Capacity	M3 demand	D/C
1	17603	14441	82%
2	17300	14441	83%
3	17596	10609	60%
4	17306	10609	61%
5	17583	8148	46%
6	17156	8148	47%
7	17521	7506	43%
8	17218	7506	44%

The P-M2 interaction curve is as follows with cases 1 to 8 plotted
f'ye = 68 ksi; f'c = 5.2 ksi, phi = 1.0
#10 rebar at 6" SPA. Throughout



Case	M2 Capacity	M2 demand	D/C
1	231560	6944	3%
2	228054	6944	3%
3	231483	14849	6%
4	228131	14849	7%
5	231331	9261	4%
6	226397	9261	4%
7	230612	11758	5%
8	227116	11758	5%

Net D/C combining ML and MT

Case	D/C
1	85%
2	87%
3	67%
4	68%
5	50%
6	52%
7	48%
8	49%

All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 15525 = 4657 kips

Case	Factored Axial Load	Limit
1	1154	4657.5
2	883	
3	1148	
4	889	
5	1137	
6	754	
7	1081	
8	810	

- If the factored axial load is less than $0.10\,\phi\,f'_c\,A_g$:

$$\frac{M_{ux}}{M_{rx}}+\frac{M_{uy}}{M_{ry}}\le 1.0 \qquad (5.6.4.5\text{-}3)$$

Shear Design Check (Weak Axis) - base of wall

V, Demand Vu= 28.2 kip/ft of wall =1056kips/37.5ft

Limit

dv= 27.7 in
bv= 12 in
f'c= 4 ksi
V, limit= 332 kips/ft of wall

$V_n = 0.25 f'_c b_v d_v + V_p$ (5.7.3.3-2)

Concrete Strength

tensile strain, es= 0.0014
β= 2.34
λ= 1
Vc, Concrete shear strength Vc= 49 kips/ft of wall

$V_c = 0.0316 \beta \lambda \sqrt{f'_c}\, b_v d_v$ (5.7.3.3-3)

Steel Tie Strength - #5 bar at 6in vertical and horizontal spacing

s= 6 in in vertical direction
Av= 0.31 in2 #5 tie
Θ= 34 degrees
Vs, Steel shear strength= 128 kips/ft of wall

$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \lambda_{duct}$

demand
Total factored shear strength, V= 159 kips/ft of wall > 28.2 OK

5.7.2.3—Regions Requiring Transverse Reinforcement

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

- $V_u > 0.5\phi(V_c + V_p)$ (5.7.2.3-1)

$d_v = \frac{M_n}{A_s f_y + A_{ps} f_{ps}}$

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of $0.9d_e$ or $0.72h$ (in.)

$\varepsilon_x = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps} f_{po}\right)}{E_s A_s + E_p A_{ps}}$ (5.7.3.4.2-4)

$|M_u|$ = absolute value of the factored moment at the section, not taken less than $|V_u - V_p| d_v$ (kip-in.)

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

V_u = factored shear force for the girder in Eq. 5.7.3.4.2-5 and for the web under consideration in Eq. 5.7.3.4.2-6 (kip)

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.7.2.5, the value of β may be determined by Eq. 5.7.3.4.2-1:

$\beta = \frac{4.8}{(1 + 750\varepsilon_x)}$ (5.7.3.4.2-1)

The value of θ in both cases may be as specified in Eq. 5.7.3.4.2-3:

$\theta = 29 + 3500\varepsilon_x$ (5.7.3.4.2-3)

Shear Design Check (Weak Axis) - Top of wall due to SEE EQ demands

V, Demand (SEE EQ) Vu= 82.4 kip/ft of wall =1.2 x 2575 kips/37.5ft

See Page 11 of Book i6

Limit

dv= 27.7 in
bv= 12 in
f'c= 4 ksi
V, limit= 332 kips/ft of wall

$V_n = 0.25 f'_c b_v d_v + V_p$ (5.7.3.3-2)

Concrete Strength

tensile strain, es= 0.0020
β= 1.93
λ= 1
Vc, Concrete shear strength Vc= 41 kips/ft of wall

$V_c = 0.0316 \beta \lambda \sqrt{f'_c}\, b_v d_v$ (5.7.3.3-3)

Steel Tie Strength - #5 bar at 12in vertical and horizontal spacing

s= 12 in in vertical direction
Av= 0.31 in2 #5 tie
Θ= 36 degrees
Vs, Steel shear strength= 59 kips/ft of wall

$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \lambda_{duct}$

demand
Total factored shear strength, V= 90 kips/ft of wall > 82.4 OK

Torsion Check

φt= 0.9
P= 921 kips unfactored compressive load DL + CR+ SH + PS
Ag= 15525.00 in2
fpc= 0.06 ksi
K= 1.11

$K = \sqrt{1 + \frac{f_{pc}}{0.126\lambda\sqrt{f'_c}}} \le 2.0$ (5.7.2.1-6)

Acp= 15525 in2
pc= 969 in
Tcr 5806 k-ft
Torsion threshold, T= 1306 k-ft
Max. Torsion Demand, Tu= 1040 k-ft

Since Tu < Threshold, Torsional effects need not be investigated

Torsional effects shall be investigated where:

$T_u > 0.25\phi T_{cr}$ (5.7.2.1-3)

- For solid shapes:

$T_{cr} = 0.126 K \lambda \sqrt{f'_c} \frac{A_{cp}^2}{p_c}$ (5.7.2.1-4)

A_{cp} = area enclosed by outside perimeter of concrete cross section (in.^{2})

p_c = length of outside perimeter of the concrete section (in.)

A_o = area enclosed by the shear flow path, including any area of holes therein (in.^{2})

b_e = effective width of the shear flow path taken as the minimum thickness of the exterior webs or flanges comprising the closed box section (in.). b_e shall be adjusted to account for the presence of ducts.

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 1 Wall Reinforcement (EXT CASE)
BY: RSGR
CHK: MWBM

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Minimum Shear Reinforcement (AASHTO LRFD BDS 5.11.4.1.4)

s=	18.0	in
hc=	31.5	in
f'c=	4	ksi
f'y=	60	ksi
Ag=	15525	in2
Ac=	12663	in2
Ash Provided=	11.5	in2
	37	#5 ties in 1ft high wall
Ash Limit 1=	2.563	in2 AASHTO LFRD BDS EQ 5.11.4.1.4-2
Ash Limit 2=	4.536	in2 AASHTO LFRD BDS EQ 5.11.4.1.4-3

For a rectangular column, the total gross sectional area, A_{sh}, of rectangular hoop reinforcement shall satisfy both of the following:

$$A_{sh} \geq 0.30\, sh_c \frac{f'_c}{f_{yh}}\left[\frac{A_g}{A_c} - 1\right] \qquad (5.11.4.1.4\text{-}2)$$

$$A_{sh} \geq 0.12 sh_c \frac{f'_c}{f_{yh}} \qquad (5.11.4.1.4\text{-}3)$$

Shear Design - Strong Axis - Elastic Check

λ=	1		
f'c=	4	ksi	
b=	450	in	
d=	36	in	
f'y=	60	ksi	
Vr=	8197	kips	for entire wall
Vn=	2041	kips	without consideration of horizontal reinforcement
Vu, demand=	541	kips	

The factored shear resistance, V_r, in the pier shall be taken as the lesser of the following:

$$V_r = 0.25\lambda\sqrt{f'_c}bd, \qquad (5.11.4.2\text{-}1)$$

$$V_r = \phi V_n \qquad (5.11.4.2\text{-}2)$$

in which:

$$V_n = \left[0.063\lambda\sqrt{f'_c} + \rho_h f_y\right]bd \qquad (5.11.4.2\text{-}3)$$

b = width of pier (in.)
d = depth of pier (in.)
ρ_h = ratio of area of horizontal shear reinforcement to area of gross concrete area of a vertical section

Shear Design Check at bottom of wall - Overstrength SEE EQ demands as per AASHTO SEISMIC 8.6

Overstrength Shear, Vpo=	1030	kips			
Ag=	15525	in2			
Ae=	12420	in2	$A_e = 0.8A_g$		
f'c=	4	ksi			
Pu=	900	kips	unfactored DL		
Av=	11.16	in2	in 1ft high section	36nos. #5 ties	
b=	34.5	in	width of colum		
s=	12	in	vertical direction		
ρw=	0.027				
fw=	3.23		>	Limit 0.35, Hence, use 0.35	
fw=	0.35				
μd=	3.0		For SDC C, the concrete shear capacity of a section within the plastic hinge region shall be determined using $\mu_D = 3$.		
α'=	3.0				
			limit		
concrete shear stress strength, vc=	0.20	ksi	0.22		
Concrete Shear strength, Vc=	2456	kips	demand		
Factored 0.9=	2211	kips	> 1030	OK	$V_c = v_c A_e$

For rectangular columns with ties:

$$\alpha' = \frac{f_w}{0.15} + 3.67 - \mu_D \qquad (8.6.2\text{-}8)$$

$$f_w = 2\rho_w f_{yh} \leq 0.35 \qquad (8.6.2\text{-}9)$$

$$\rho_w = \frac{A_v}{bs} \qquad (8.6.2\text{-}10)$$

If P_u is compressive:

$$v_c = 0.032\alpha'\left(1 + \frac{P_u}{2A_g}\right)\sqrt{f'_c} \leq \min\begin{cases} 0.11\sqrt{f'_c} \\ 0.047\alpha'\sqrt{f'_c} \end{cases} \qquad (8.6.2\text{-}3)$$

Pier Wall Design Checks AASHTO SEISMIC 8.6.5

- For SDCs C and D: $\rho_w \geq 0.004$ (8.6.5-4)

ρw, provided=	11.16	
ρw, required=	0.004	OK

Pier Wall Design Checks AASHTO SEISMIC 8.6.9 - Shear in Strong Direction

Not checked since designed to stay elastic

Pier Wall Design Checks AASHTO SEISMIC 8.6.10

ρv, provided=	0.013	158 #10 bars
ρh, provided=	0.010	#9 bars each face 6" (79 Total)
ρh, required=	0.0025	OK

8.6.10—Pier Wall Minimum Reinforcement

The horizontal reinforcement ratio, ρ_h, shall not be less than 0.0025. The vertical reinforcement ratio, ρ_v, shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 18 in.

The reinforcement required for shear shall be continuous and shall be distributed uniformly. Horizontal and vertical layers of reinforcement shall be provided on each face of a pier. Splices in horizontal pier reinforcement shall be staggered.

Joint Shear Check AASHTO SEISMIC 6.4.5

Mpo=	20094	k-ft	See Moment curvature analysis			
Dv=	27.6	in	Distance between Tension and compression arms			
Tc=	8752	kips				
f'c=	4.0	ksi				
Bc=	450	in				
Dftg=	48	in	- Stirrups or to be used in joint shear zone if: $P_t > 0.11\sqrt{f'_c}$ (ksi)			
Dc=	34.5	in				
A=	41085	in2				AASHTO SEISMIC eq 6.4.5-11
Pcol=	900	kips	unfactored DL at base of wall			
fv=	0.0219	ksi				AASHTO SEISMIC eq 6.4.5-9
vjv=	0.4052	ksi				AASHTO SEISMIC eq 6.4.5-5
pt=	0.3944	ksi	<	0.76	ksi OK	AASHTO SEISMIC eq 6.4.5-2
pc=	0.4163	ksi	<	1.00	ksi OK	AASHTO SEISMIC eq 6.4.5-1

Purpose:
Design the reinforcement for the wall of Pier 1 to STR/SER Loads
Asumptions:
1. Loads are extracted from the bridge FEM model and are combined as per appropriate load combinations.
2. Critical section occurs near the base of the wall for flexure.
3. Wall shear is taken at a distance Dv away from the base of the wall.

SUMMARY

	bar	Nos.	spacing	D/C
Wall main vertical reinforcement	10	156	6	60%
Wall main horizontal reinforcement	9	79	6	-
Wall Shear Ties Plastic Zone	5	360	6	29%
Wall Shear Ties outside Plastic Zone	5	540	12	-

Rashim Garg:
Governed by detailing requirements rather than loads

Notes and References

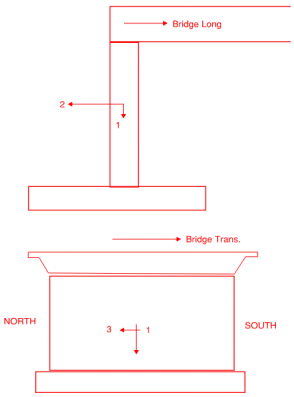
input

DIMENSIONS & PARAMETERS

f'c	4	ksi	concrete compressive strength
wc =	0.155	kcf	Unit weight of weight concrete
K1 =	1		Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4028	ksi	Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
fy =	60	ksi	Yield strength of reinforcement
Es =	29000	ksi	Modulus of elasticity of reinforcement
η =	7.20		Modular ratio
β_1 =	0.85		Compression Zone Factor
φf=	0.9		STR case flexural resistance factor
φv=	0.9		factor for shear resistance
Heel Length, HL=	12.00	ft	
Toe Length, TL=	9.67	ft	
Total footing Length=	38.72	ft	
Total Wall Length=	37.50	ft	
Stem Width at design section, D=	2.875	ft	
Footing Depth=Df=	4	ft	
Area of wall=	15525	in2	
Area of main tensile reinforcement=	99.06	in2	78 #10 rebar
Area of total reinforcement=	198.12	in2	Meets max. and min limits for compression member (provided = 1.28%)

5.11.4.1.1—Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.04 times the gross cross-section area, A_g.



LOADING ON THE WALL (Refer to Footing Sizing calcs)

		P	V2	V3	T	M2	M3	
		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
	DC, Supr	510	7	4	-15	1190	-139	
	DC, Supr	510	7	4	-15	-1126	84	
	DC, Wall	340	0	0	0	0	0	
	CR&SH	55	-13	-4	28	-410	250	
	PS	16	-14	-5	18	-64	279	
Min	LL	169	164	28	56	1573	2397	IM included; BR and CE included
Max	LL	-21	-148	-67	-64	-1338	-3216	IM included; BR and CE included
	LS_Behind_Pier1	0	-60	0	0	0	285	
	LEP_Behind_Pier1	0	-525	0	22	0	2150	
Max	WL	0	3	22	15	502	60	
Min	WL	0	-3	-22	-15	-502	-60	
Max	WS (SER)	0	18	9	9	196	295	
Min	WS (SER)	0	-18	-9	-9	-196	-295	
Max	WS (STR3)	16	32	19	18	654	520	
Min	WS (STR3)	-16	-32	-19	-18	-654	-520	
Max	WS (STR5)	1	24	11	11	258	385	
Min	WS (STR5)	-1	-24	-11	-11	-258	-385	
	SE	-13	-5	0	-1	-109	98	
	TG+	51	9	-7	11	183	-174	
	TG-	-15	-3	2	-3	-55	52	
	TU+	5	0	10	13	-132	7	
	TU-	-7	-125	-16	-11	211	755	

DESIGN LOADS

		DC	LL	CR SH PS	EH behind Pier 1	LS behind Pier 1	WL	WS	TU	TG	SE
STR I	Max	1.25	1.75	0.5	1.35	1.75	0	0.0	1	0.0	1.0
	Min	0.9	1.75	0.5	1.35	0	0	0.0	1	0.0	1.0
STR III	Max	1.25	0	0.5	1.35	0	0	1.0	1	0.0	1.0
	Min	0.9	0	0.5	1.35	0	0	1.0	1	0.0	1.0
STR V	Max	1.25	1.35	0.5	1.35	1.75	1	1.0	1	0.0	1.0
	Min	0.9	1.35	0.5	1.35	0	1	1.0	1	0.0	1.0
SER	Max	1	1	0.5	1	1	1	1.0	1	1.0	1.0
	Min	1	1	0.5	1	0	1	1.0	1	1.0	1.0

STR/SER Cases

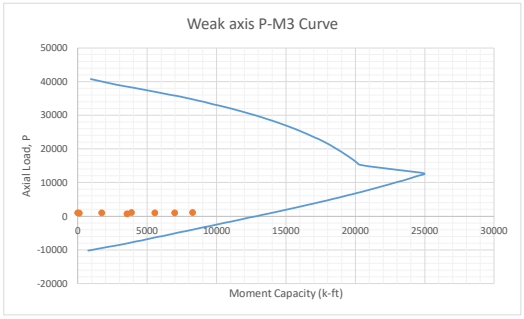
		Case	P	M2	M3	V2	V3	
	Max	1	1054	-4227	-1752	-1083	-107	
STR I	Min	2	1075	1604	8291	-559	32	
	Max	3	1106	-1232	3898	-687	30	
STR III	Min	4	763	-1802	3576	-878	-36	
	Max	5	1063	-2933	-21	-997	-46	
STR V	Min	6	1007	1219	7008	-645	32	
	Max	7	909	-2061	-146	-715	-33	
SER	Min	8	1018	563	5567	-515	27	Used to check cracking

Check Cracking using SER demands

M, SER=	66798	k-in		
Ye =	0.75	Class 2 exposure		
thickness of member, h=	34.50	in		
Thickness of concrete cover, dc=	3.76	in		
β=	1.17			
Tensile Reinforcement Ratio, ρ =	0.006	Tensile reinforcement one face only		
η =	7.20			
k=	0.261	$k = \sqrt{2n\rho + (n\rho)^2} - n\rho$		
j=1-k/3=	0.913			0.6 x f'y
fss=	25	ksi	<	36 ksi
max. spacing of reinforcement, s=	11	in		
provided, s=	6	in	OK	

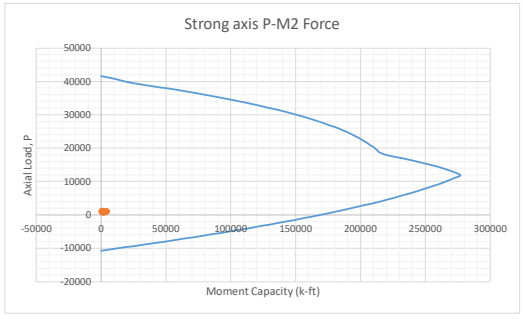
Axial - Flexure Design STR Check

The P-M3 interaction curve is as follows with cases 1 to 8 plotted
f'y = 60 ksi; f'c = 4ksi, phi = 0.9
#10 rebar at 6" SPA. Throughout



Case	M3 Capacity	M3 demand	D/C
1	14018	1752	13%
2	14042	8291	59%
3	14076	3898	28%
4	13693	3576	26%
5	14028	21	0%
6	13966	7008	50%
7	13856	146	1%
8	13978	5567	40%

The P-M2 interaction curve is as follows with cases 1 to 8 plotted
f'y = 60 ksi; f'c = 4ksi, phi = 0.9
#10 rebar at 6" SPA. Throughout



Case	M2 Capacity	M2 demand	D/C
1	181816	4227	2%
2	182074	1604	1%
3	182437	1232	1%
4	178324	1802	1%
5	181925	2933	2%
6	181267	1219	1%
7	180091	2061	1%
8	181397	563	0%

Net D/C combining ML and MT

Case	D/C
1	15%
2	60%
3	28%
4	27%
5	2%
6	51%
7	2%
8	40%

All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 15525 = 4657 kips

Case	Factored Axial Load	Limit
1	1054	4657.5
2	1075	
3	1106	
4	763	
5	1063	
6	1007	
7	909	
8	1018	

- If the factored axial load is less than $0.10\,\phi\,f'_c\,A_g$:

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}} \le 1.0 \qquad (5.6.4.5\text{-}3)$$

Shear Design Check - Weak Axis

V, Demand	Vu	28.9	kip/ft of wall	
V, Threshold Limit	Vt	21.7	= 0.5 x 0.9 x Vc; Since Vu > Vt; atleast minimum shear ties are needed	
dv		27.7	in	
bv		12	in	$V_n = 0.25 f'_c b_v d_v + V_p$ (5.7.3.3-2)
f'c		4	ksi	
V, limit=		332	kips/ft of wall	

Concrete Strength

tensile strain, es	0.0014	
β	2.30	
λ	1	
Vc, Nominal concrete shear strength	48	kips/ft of wall

$V_c = 0.0316 \beta \lambda \sqrt{f'_c}\, b_v d_v$ (5.7.3.3-3)

Steel Tie Strength - #5 bar at 12in vertical spacing

s	12	in	in vertical direction
Av	0.31	in2	in ft. wide wall
Θ	34	degrees	
Vs, Steel shear strength	63	kips/ft of wall	

$V_s = \frac{A_v f_y d_v (\cot\theta + \cot\alpha)\sin\alpha}{s} \lambda_{duct}$

Total factored shear strength, V=	101	kips/ft of wall	>	demand 28.9 OK

5.7.2.3—Regions Requiring Transverse Reinforcement

Except for slabs, footings, and culverts, transverse reinforcement shall be provided where:

- $V_u > 0.5\phi\left(V_c + V_p\right)$ (5.7.2.3-1)

$d_v = \frac{M_u}{A_s f_y + A_{ps} f_{ps}}$

d_v = effective shear depth taken as the distance, measured perpendicular to the neutral axis, between the resultants of the tensile and compressive forces due to flexure; it need not be taken to be less than the greater of $0.9d_e$ or $0.72h$ (in.)

$\varepsilon_s = \frac{\left(\frac{|M_u|}{d_v} + 0.5N_u + |V_u - V_p| - A_{ps}f_{po}\right)}{E_s A_s + E_p A_{ps}}$ (5.7.3.4.2-4)

$|M_u|$ = absolute value of the factored moment at the section, not taken less than $|V_u - V_p| d_v$ (kip-in.)

N_u = factored axial force, taken as positive if tensile and negative if compressive (kip)

V_u = factored shear force for the girder in Eq. 5.7.3.4.2-5 and for the web under consideration in Eq. 5.7.3.4.2-6 (kip)

For sections containing at least the minimum amount of transverse reinforcement specified in Article 5.7.2.5, the value of β may be determined by Eq. 5.7.3.4.2-1:

$\beta = \frac{4.8}{(1+750\varepsilon_s)}$ (5.7.3.4.2-1)

The value of θ in both cases may be as specified in Eq. 5.7.3.4.2-3:

$\theta = 29 + 3500\varepsilon_s$ (5.7.3.4.2-3)

Minimum Transverse Reinforcement

λ	1			
f'c	4	ksi		
bv	34.5	in		
s	12	in		
f'y	60	ksi		
Av, required	0.44	in2		
Av, provided	1.31	in2	OK	Includes 1-#5 tie and 1-#9 horizontal bar

Spacing Check			0.125*f'c		
shear stress, vu=	0.10	<	0.50	ksi	
s, max=	22	in			
provided, s=	12	in	OK	Max Tie spacing	

$A_v \geq 0.0316\, \lambda \sqrt{f'_c}\, \frac{b_v s}{f_y}$

where:

A_v = area of transverse reinforcement within distance s (in.^{2})
λ = concrete density modification factor as specified in Article 5.4.2.8
b_v = width of web adjusted for the presence of ducts as specified in Article 5.7.2.8 (in.)
s = spacing of transverse reinforcement (in.)
f_y = yield strength of transverse reinforcement (ksi) ≤100 ksi

5.7.2.6—Maximum Spacing of Transverse Reinforcement

The spacing of the transverse reinforcement shall not exceed the maximum permitted spacing, s_{max}, determined as:

- If $v_u < 0.125 f'_c$, then:
  $s_{max} = 0.8 d_v \leq 24.0$ in. (5.7.2.6-1)
- If $v_u \geq 0.125 f'_c$, then:
  $s_{max} = 0.4 d_v \leq 12.0$ in. (5.7.2.6-2)

where:

v_u = shear stress calculated in accordance with Article 5.7.2.8 (ksi)
d_v = effective shear depth as defined in Article 5.7.2.8 (in.)

Minimum Horizontal Reinforcement

Use #9rebar at 6" SPA in horizontal direction

Area of steel in 1ft high wall=	2	in2	2 #9 bars combined in E.F	
Area of concrete in 1ft high wall=	414	in2		
			min. required	
ρ=	0.0048	>	0.0025	OK

5.11.4.2—Requirements for Wall-Type Piers

The provisions herein specified shall apply to the design for the strong direction of a pier. The weak direction of a pier may be designed as a column conforming to the provisions of Article 5.11.4.1, with the response modification factor for columns used to determine the design forces. If the pier is not designed as a column in its weak direction, the limitations for factored shear resistance herein specified shall apply.

The minimum reinforcement ratio, both horizontally, ρ_h, and vertically, ρ_v, in any pier shall not be less than 0.0025. The vertical reinforcement ratio shall not be less than the horizontal reinforcement ratio.

Reinforcement spacing, either horizontally or vertically, shall not exceed 18.0 in. The reinforcement required for shear shall be continuous and shall be distributed uniformly.

The factored shear resistance, V_r, in the pier shall be taken as the lesser of the followine:

C5.11.4.2

The requirements of this article are based on limited data available on the behavior of piers in the inelastic range. Consequently, the R-factor of 2.0 for piers is based on the assumption of minimal inelastic behavior.

The requirement that $\rho_v \geq \rho_h$ is intended to avoid the possibility of inadequate web reinforcements in piers, which are short in comparison to their height. Splices should be staggered in an effort to avoid weak sections.

PIER 1 : FOOTING REINFORCEMENT DESIGN

FOOTING IS DESIGNED AS 48" THICK AND IS TIED INTO EXISTING PIER 1 WALL VIA #9 DOWELS

COWI

PROJECT:	I405 BR17.7	JOB NO.:	A207833
SUBJECT:	Pier 1 Wall and Footing Reinforcement		
BY:	RSGR	DATE:	2021/11/29
CHK:	MWBM	DATE:	2021/11/29

Purpose:
Design the reinforcement for Pier 1 footing based on load combinations in section 2 of this sheet which are taken from footing sizing sheets

Asumptions:
1. Footing load effects M and V are derived for per ft of the wall length and then design is carried out on a per-ft basis.
2. Critical section in footing occurs at the Heel in all STR, SER and EXT cases at wall/heel interface
3. Main reinforcement for heel is at the bottom of footing for EXT case (East to West EQ)
4. Main reinforcement for heel is at the top of footing for STR SER case (due to heel soil load)
5. Main reinforcement for toe at the bottom of footing for EXT case. (West to East EQ)
6. Main reinforcement for toe at the top of footing for EXT case (East to West EQ)
7. Critical section for shear in footing occurs at Dv from the face of the wall.

SUMMARY	bar	Nos.	bar diam.	bar area	spacing	D/C
Heel Reinforcement (top and bottom)	11	86	1.41	1.56	6	87%
Heel Transverse Reinforcement	9	29	1.128	1	6	-
Heel Shear Stirrups	5	310	0.625	0.31	12	89%
Toe Reinforcement (bottom)	11	86	1.41	1.56	6	54%
Toe Transverse Reinforcement	9	18	1.128	1	6	-
Toe Shear Stirrups	5	250	0.625	0.31	12	69%

Notes and References

input

1.0 FOOTING DIMENSION AND MATERIALS

f'c	4	ksi	f'ce	5.8	ksi
wc =	0.155	kcf			
K1 =	1				
Ec =	4028	ksi			
fy =	60	ksi	f'ye =	68	ksi
Es =	29000	ksi			
η =	7.20				
β_1 =	0.85				
φf=	1				
φv=	0.9				
Design Width, b=	1	ft			
Heel Length, HL=	12.00	ft			
Toe Length, TL=	9.67	ft			
Total footing Length=	38.72	ft			
Stem Width at design section, D=	3	ft			
Total footing width, B=	24.67	ft			
Footing Depth=Df=	4	ft			

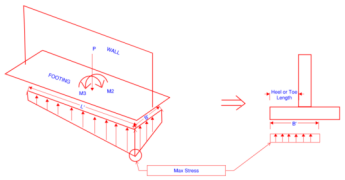
concrete compressive strength
Unit weight of weight concrete
Correction factor for source of aggregate, AASHTO 5.4.2.4
Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
Yield strength of reinforcement
Modulus of elasticity of reinforcement
Modular ratio
Compression Zone Factor
EXT case flexural resistance factor (0.9 for STR)
factor for shear resistance
design for unit width
Stem thickness at bottom

2.0 LOAD ON FOOTING

L= 24.67 ft
B= 38.72 ft

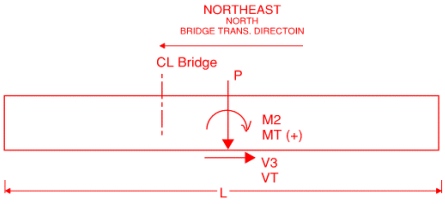
Use max bearing stress from biaxial moments and total axial load to calculate
Moment at face of the wall and shear at Dv from face
Moment on heel side at wall/footing interface= Max. bearing stress x heel length x heel length x 0.5
Shear on heel side at wall/footing interface= Max. bearing stress x (heel length - Dv)
Moment on heel side at wall/footing interface= Max. bearing stress x toe length x toe length x 0.5
Shear on heel side at wall/footing interface= Max. bearing stress x (toe length - Dv)

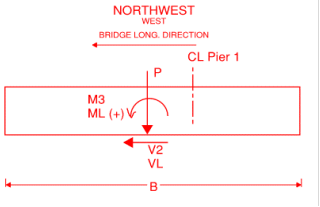
Footing Dv= 41.38 in



		P	ML	MT	e	angle	Effective B (ft)	Effective L (ft)	Bearing stress (ksf)	Design M	Design V	
		Kip	Kip-ft	Kip-ft						k-ft/ft	k-ft/ft	
	SER I	3166	8652	-5355	3.21	-1.017	19.2	35.3	4.7	336	40	
	STR I	4125	13038	-6262	3.51	-1.123	18.3	35.7	6.3	454	54	
	STR III	4138	9342	-4472	2.50	-1.124	20.2	36.6	5.6	404	48	Pressure on heel
	STR V	4125	12565	-6537	3.43	-1.091	18.6	35.6	6.2	450	53	
EtoW Max	EXT I	5025	0	-36568	7.28	0.000	24.7	24.2	8.4	607	72	
EtoW Min	EXT I	4310	0	-36568	8.48	0.000	24.7	21.8	8.0	578	69	Pressure on heel
WtoE Max	EXT I	3363	-15086	-22400	8.03	0.593	15.7	25.4	8.4	394	72	
WtoE Min	EXT I	3202	-15086	-22400	8.43	0.593	15.2	24.7	8.5	397	73	Pressure on Toe

MT: Moment about wall stong axis
ML: Moment about wall weak axis





FLEXURE DESIGN - HEEL - EXT CASE - Footing Bottom reinforcement

Cover, c=	3	in		AASHTO Table 5.10.1-1
bar size=	11	#		
spacing=	6	in		
bar diam=	1.410	in		
bar area=	1.560	in2		
Effective depth, de=	43.670	in		Main bar is outside layer
Area of steel per unit width, As=	3.120	in2/ft		$C_b = \frac{A_s f_y}{\beta_1 0.85 f_c' b}$
distance compression fibre to NA, Cb=	4.2	in		$a = \beta_1 C_b$
depth of equiv stress block, a=	3.6	in		
Nominal Flexural Resistance, Mn=	740	k-ft/ft		$M_n = A_s f_y \left(d_e - \frac{a}{2}\right)$
Factored Flexural Resistance, φMn=	740	k-ft/ft	EXT Case	Demand<Capacity OK

Minimum reinforcement

1.33 x Mu=	807.3	k-ft/ft	EXT Case	AASHTO 5.6.3.3

The cracking strength Mcr is calculated as follows:

fr=	0.48	ksi		AASHTO 5.4.2.6 $f_r = 0.24\sqrt{f'_c}$
Ig=	110592	in4	=4ft by 1ft section	
yt=	24	in	=Ig/Sc	
γ1=	1.60			AASHTO 5.6.3.3
γ3=	0.67			AASHTO 5.6.3.3 $M_{cr} = \gamma_3\left[\left(\gamma_1 f_r + \gamma_2 f_{cpe}\right)S_c - M_{dnc}\left(\frac{S_c}{S_{nc}} - 1\right)\right]$ (5.6.3.3-1)
Sc=	4608	in3	=(1*12)*(4ft*12)^2/6	
Mcr=	198	k-ft/ft		AASHTO eq 5.6.3.3-1
$\varphi_f M_n$ =	740	>	198	OK = φMn > min. (1.33 Mu or Mcr)

Control of cracking

Mser=	336	k-ft/ft		
Exposure Class=	2			
Ye =	0.75			Exposure Factor
Thickness of concrete cover, dc=	3.705	in		=cover +outer bar diam./2
Reinforcement Ratio, ρ =	0.005			
η =	7.20			Modular Ratio
k =	0.243			$k = \sqrt{2n\rho + (n\rho)^2} - n\rho$
j=1-k/3	0.919			=1-k/3
Tensile stress in steel, fs =	32.2	ksi		=Mser/(As x j x de)
βs=	1.12			$\beta_s = 1 + \frac{d_c}{0.7(T_{Bot} - d_c)}$
Maximum Spacing, smax=	7.2	in		Tbot=stem thickness
spacing provided,s=	6	in	OK	$s_{max} = \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$

SHEAR DESIGN - HEEL - EXT - Bottom reinforcement

factored shear load, Vu=	72	kips/ft	From EXT Case				
Effective depth, dv=	41.4	in					AASHTO 5.7.2.8
tensile strain in section, εs=	0.0027423	in/in					AASHTO 5.7.3.4.2-4
crack spacing parameter (1), sx=	12	in					
crack spacing parameter (1), sxe=	12	in					
shear resistance parameter,β=	1.570						AASHTO 5.7.3.4.2-2
Concrete modification factor, λ=	1						AASHTO 5.4.2.8
Nominal Shear Resistance, Vc=	49	kips/ft					AASHTO 5.7.3.3-3
Factored Shear Resistance, Vr=φVc	44	kips/ft	NG Add Transverse Steel				
bar size=	5	#					
spacing,s=	12	in	Prov spacing is OK		in bridge long. Direction		Staggered See WSDOT BDM Fig 7.5.10-1
Smax=	25	in					=0.6xdv
bar diam=	0.625	in					
Steel area, Av=	0.207	in2/ft	18" spacing in other transverse direction				
Θ=	39	degrees					AASHTO 5.7.3.4.2-4
Shear resistance from steel, Vs=	54	kips/ft		LIMIT			AASHTO 5.7.3.3-4
Factored Shear Resistance, φVn=φ(Vc+Vs)	93	kips/ft	OK	447	kips/ft	OK	Limit AASHTO 5.7.3.3-2

Transverse Reinforcement

Shrinkage and Temperatue AASHTO 5.10.6

bar size=	9	#	
spacing,s=	6	in	
bar diam=	1.128	in	
Steel area, As=	2.000	in2/ft	
b=	296	in	
Tbot=	48	in	

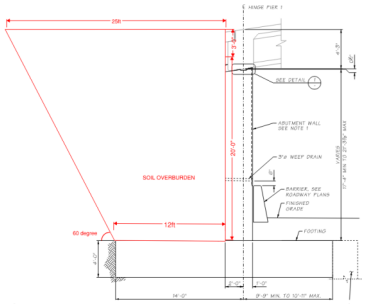
Check $A_s \geq \frac{1.30\, b\, T_{Bot}}{2(b + T_{Bot}) f_y}$ = 0.45 OK — AASHTO 5.10.6-1

FLEXURE DESIGN - HEEL - STR - Bottom reinforcement

Cover, c=	3	in		AASHTO Table 5.10.1-1
bar size=	11	#		
spacing=	6	in		
bar diam=	1.410	in		
bar area=	1.560	in2		
Effective depth, de=	43.670	in		$C_b = \frac{A_s f_y}{\beta_1 0.85 f_c' b}$
Area of steel per unit width, As=	3.120	in2/ft		$a = \beta_1 C_b$
distance compression fibre to NA, Cb=	5.398	in		
depth of equiv stress block, a=	4.588	in		$M_n = A_s f_y \left(d_e - \frac{a}{2}\right)$
Nominal Flexural Resistance, Mn=	645	k-ft/ft		
Factored Flexural Resistance, φMn=	581	k-ft/ft	STR Case	Demand<Capacity OK

FLEXURE DESIGN - HEEL - STR - Top reinforcement

The critical section for shear and moment is at the back face of the stem wall. The heel is designed to carry its self weight and the soil block above it assuming zero pressure under footing.



Calculate weight of soil block

Length of retained soil	38.72	ft
Height of retained soil	23.00	ft
Width of soil at base	12.00	ft
Width of soil at top	25.28	ft
Volume of soil	16600	ft3
Unit weight=	0.130	kcf
Total soil weight on heel=	2158	kips

	Unfactored Vertical Loads		Moment Arm from Wall/Footing Face		V k/ft	M k-ft/ft	
Dead Load, DC_{heel}=	288	kips	6.00	ft (half of heel length)	7	45	
Soil Load, Ev_{heel}=	2158	kips	6.00	ft (half of heel length)	56	334	divide by footing length

LOAD CASE	DC	Ev
Strength I Load Case:	1.25	1.35
Service I Load Case:	1	1

Load Comb	Vu	Mu
	kip/ft	k-ft/ft
Service I Load Case:	63	379
Strength I Load Case:	85	507

FLEXURE

Cover, c=	3	in		AASHTO Table 5.10.1-1
bar size,	11	#		
spacing,s=	6	in		spacing to match the wall main rebar spacing
bar diam=	1.410	in		
bar area=	1.560	in2		
Effective depth, de=	43.670	in		
Area of steel per unit width, As=	3.120	in2/ft		
distance compression fibre to NA, Cb=	5.398	in		$C_b = \frac{A_s f_y}{\beta_1 0.85 f_c' b}$
depth of equiv stress block, a=	4.588	in		$a = \beta_1 C_b$
Nominal Flexural Resistance, Mr=	645	k-ft/ft		$M_n = A_S f_y \left(d_e - \frac{a}{2}\right)$
Factored Flexural Resistance, φMr=	581	k-ft/ft	Demand<Capacity OK	

Control of cracking

Mser=	379	k-ft/ft		
Exposure Class=	2			
Ye =	0.75			Exposure Factor
Thickness of concrete cover, dc=	3.705	in		=cover + bar diam./2
Reinforcement Ratio, ρ =	0.005			
η =	7.20			Modular Ratio
k =	0.243			$k = \sqrt{2n\rho + (n\rho)^2} - n\rho$
j=1-k/3	0.919			=1-k/3
Tensile stress in steel, fs =	36.32	ksi		=Mser/(As x j x de)
βs=	1.119			$\beta_s = 1 + \frac{d_c}{0.7(T_{Bot} - d_c)}$
Maximum Spacing, smax=	6	in		$s_{max} = \frac{700\gamma_e}{\beta_s f_{ss}} - 2d_c$
spacing provided,s=	6	in	OK	

SHEAR

factored shear load, Vu=	85	kips/ft					
Effective depth, dv=	41.0	in					AASHTO 5.7.2.8
tensile strain in section, εs=	0.0025763	in/in					AASHTO 5.7.3.4.2-4
crack spacing parameter (1), sx=	12.0	in					
crack spacing parameter (1), sxe=	12	in					
shear resistance parameter,β=	1.6369935						AASHTO 5.7.3.4.2-2
Concrete modification factor, λ=	1						AASHTO 5.4.2.8
Nominal Shear Resistance, Vc=	51	kips/ft					AASHTO 5.7.3.3-3
Factored Shear Resistance, Vr=φVc	46	kips/ft	NG Add Transverse Steel				
Check minimum steel requirement 0.5Vr=	23	kips/ft	NG Add Min Transverse Steel		=0.5Vr < Vu		
bar size=	5	in					
spacing,s=	12	in					Staggered See WSDOT BDM Fig 7.5.10-1
Smax=	25	in	Prov spacing is OK				=0.6xdv
bar diam=	0.625	in					
Steel area, Av=	0.207	in2/ft	18" spacing in other transverse direction				
Av/s=	0.017	>	0.013	OK			AASHTO 5.7.2.5-1
Θ=	38						AASHTO 5.7.3.4.2-4
Shear resistance from steel, Vs=	54	kips/ft		LIMIT			AASHTO 5.7.3.3-4
Factored Shear Resistance, φVn=φ(Vc+Vs)	95	kips/ft	OK	442	kips/ft	OK	Limit AASHTO 5.7.3.3-2

COWI

PROJECT:	I405 BR17.7	JOB NO.:	A207833
SUBJECT:	Pier 1 Wall and Footing Reinforcement		
BY:	RSGR	DATE:	2021/11/29
CHK:	MWBM	DATE:	2021/11/29

FOOTING TOE DESIGN (reinforcement at bottom of footing)

The critical section for shear is dV from front face of wall stem and, for moment, is at the front face of wall stem. Section is designed to resist bearing stress acting on toe.

Cover, c=	3	in		
bar size=	11	#		
spacing=	6	in		
bar diam=	1.410	in		
bar area=	1.560	in2		
Effective depth, de=	43.670	in		
Area of steel per unit width, As=	3.120	in2/ft		
distance compression fibre to NA, Cb=	4.2	in		
depth of equiv stress block, a=	3.6	in		
Nominal Flexural Resistance, Mn=	740	k-ft/ft		
Factored Flexural Resistance, φMn=	740	k-ft/ft	EXT Case	Demand<Capacity OK

SHEAR DESIGN - TOE - EXT

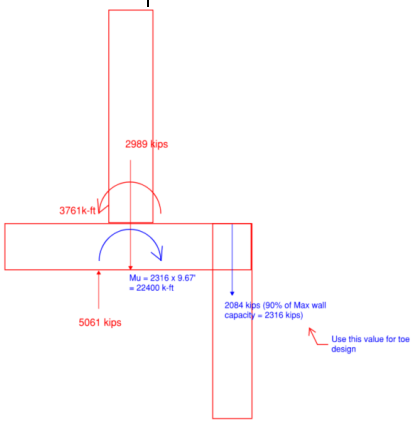
factored shear load, Vu=	73	kips/ft	From EXT Case				
Effective depth, dv=	41.4	in					AASHTO 5.7.2.8
tensile strain in section, εs=	0.00208	in/in					AASHTO 5.7.3.4.2-4
crack spacing parameter (1), sx=	12	in					
crack spacing parameter (1), sxe=	12	in					
shear resistance parameter,β=	1.878						AASHTO 5.7.3.4.2-2
Concrete modification factor, λ=	1						AASHTO 5.4.2.8
Nominal Shear Resistance, Vc=	59	kips/ft					AASHTO 5.7.3.3-3
Factored Shear Resistance, Vr=φVc	53	kips/ft	NG Add Transverse Steel				
bar size=	5	#					
spacing,s=	12	in	Prov spacing is OK				Staggered See WSDOT BDM Fig 7.5.10-1
Smax=	25	in					=0.6xdv
bar diam=	0.625	in					
Steel area, Av=	0.207	in2/ft	18" spacing in other transverse direction				
Θ=	36	degrees					AASHTO 5.7.3.4.2-4
Shear resistance from steel, Vs=	58	kips/ft		LIMIT			AASHTO 5.7.3.3-4
Factored Shear Resistance, φVn=φ(Vc+Vs)	105	kips/ft	OK	447	kips/ft	OK	Limit AASHTO 5.7.3.3-2

FOOTING TOE DESIGN (reinforcement at top of footing)

Based on Maximum Tie-Down force from existing wall

Mu=	22400	k-ft	
Demand, Mu=	597.3	k-ft/ft	=22400k-ft / 37.5ft

Cover, c=	3	in		
bar size,	11	#		
spacing,s=	6	in		
bar diam=	1.410	in		
bar area=	1.560	in2		
Effective depth, de=	43.670	in		
Area of steel per unit width, As=	3.120	in2/ft		
distance compression fibre to NA, Cb=	5.398	in		
depth of equiv stress block, a=	4.588	in		
Nominal Flexural Resistance, Mr=	645	k-ft/ft		
Factored Flexural Resistance, φMr=	645	k-ft/ft		Demand<Capacity OK



COWI

PROJECT:	I405 BR17.7	JOB NO.:	A207833
SUBJECT:	Pier 1 Wall and Footing Reinforcement		
BY:	RSGR	DATE:	2021/11/29
CHK:	MWBM	DATE:	2021/11/29

INTERFACE SHEAR RESISTANCE (Between wall and footing)

φv=	0.9			
bvi=	12	in		interface width considered to engage shear transfer
Lvi=	36	in	(stem depth)	interface length considered to engage shear transfer
Acv=	432	in^2		AASHTO 5.7.4.3-6
Pc=	22.67	kips/ft	Bridge DL = 510k + Wall DL 340 k = 850/37.5ft	Permanent net compressive force (kip/ft)
c=	0.075	ksi		
μ=	0.6			For concrete placed against a clean concrete surface, free of laitance but not intentionally roughened:
K1=	0.2			
K2=	0.8	ksi		
Avf=	5.080	in2/ft	4 - #10 rebars in each ft of wall	Area of wall (#10 @6" E.F.) steel within Acv

				Limit		
Vni=	229	kips/ft	<	345.6	kips/ft	OK

AASHTO 5.7.4.3 - 3 to 5

				Vui		
Vri=φVni=	206	kips/ft	>	37		OK

Vui = 1401kips /37.5 ft from EXT I case
where 1401 kips is maximum sliding demand
See footing sizing calcs

The nominal shear resistance of the interface plane shall be taken as:

$$V_{ni} = cA_{cv} + \mu\,(A_{vf} f_y + P_c) \qquad (5.7.4.3\text{-}3)$$

The nominal shear resistance, V_{ni}, used in the design shall not exceed either of the following:

$$V_{ni} \le K_1 f'_c\, A_{cv} \qquad (5.7.4.3\text{-}4)$$

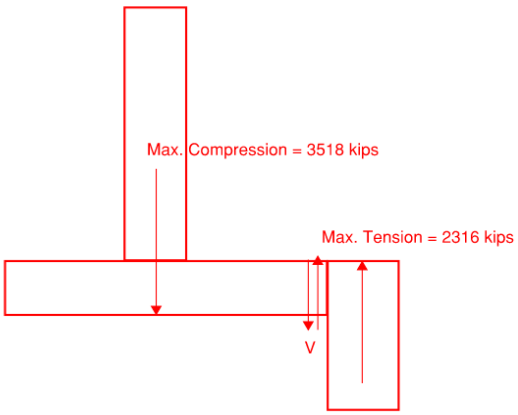
$$V_{ni} \le K_2\, A_{cv} \qquad (5.7.4.3\text{-}5)$$

COWI

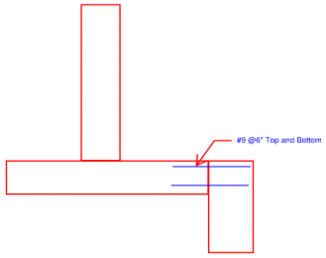
PROJECT:	i405 BR17.7	JOB NO.:	A207833
SUBJECT:	Design Connection between New and existing footing		
BY:	RSGR	DATE:	2021/11/29
CHK:	MWBM	DATE:	2021/11/29

1.0 Max Interface shear force between New and Existing footing

- Max. shear needed to be transferred at the interface between new pier 1 footing and existing Pier 1 wall should be adequate to transfer maximum compression load at new footing or maximum tension force needed to resist the overturning moment at new footing.
- Max. compression force at new footing = 3518 kips (see footing sizing checks for WtoE EQ)
- Max. Tension force at new footing = 2316 kips (see footing sizing checks for EtoW EQ)



Hence, Design Shear at connection interface, V=	3518 kips		
f'y of dowel rebar=	60 ksi		
Shear Resistance Factor, φ dowel=	0.65		ACI 318-14 17.3.3
Nos. of dowels in 29ft of connection, n=	116	nos.	58 nos. in top layer and 58nos. In bottom layer
spacing=	6	in	
Dowel Size #=	9	#	
Dowel Area, Ad=	1.128	in2	
Dowel diam., d=	1.000	in	
Nominal Dowel Resistance, Vn=	7851	kips	=f'y x n x Ad
Factored Resistance, Phi x Vn=	5103	kips	=0.65 x Vn



PIER 1 : MOMENT CURVATURE ANALYSIS

NOTES:

1.WALL IS DESIGNED TO GO PLASTIC ABOUT ITS WEAK AXIS DURING SEE LEVEL EQ

2. WALL IS CHECKED AND DESIGNED FOR OVERSTRENGTH SHEAR RESULTING FROM THE OVERSTRENGTH MOMENT OF THE WALL

3. THE SECTION IS MODELLED IN SECTION DESIGNER IN SAP-2000

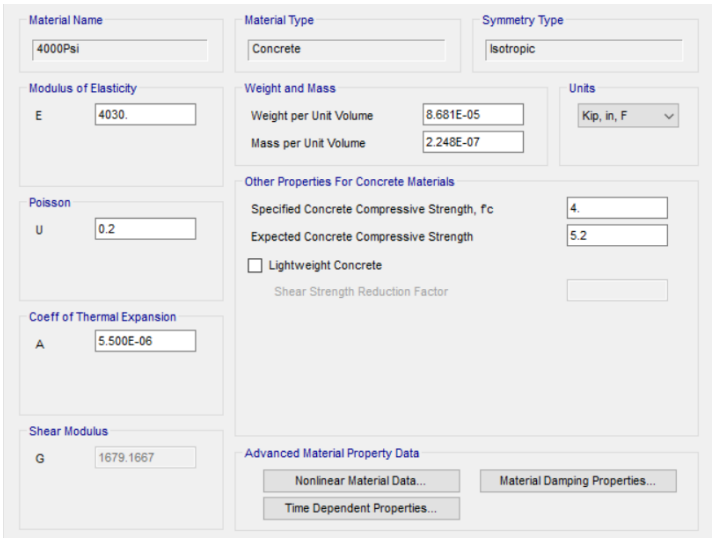
4. MATERIAL PROPERTIES ARE DEFIINED AS PER WSDOT BDM TABLE 4.3.2-1 AND AASHTO SEISMIC GUIDE 8.4.4

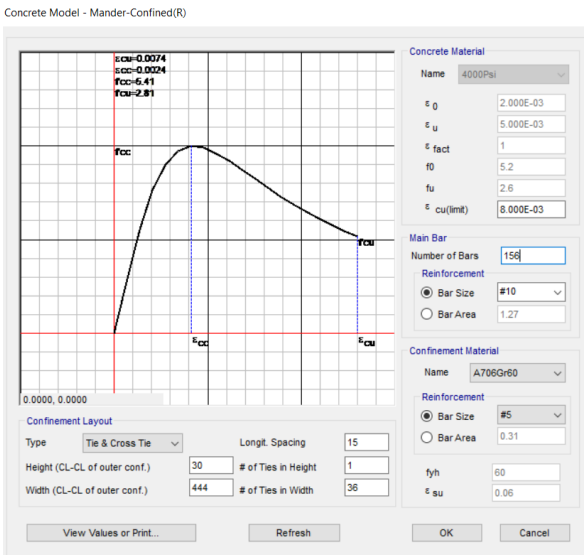
5. PLASTIC AND OVERSTRENGTH MOMENTS ARE THEN CALCULATED FROM THE MOMENT-CURVATURE ANALYSIS FROM SAP2000 RESULTS AND MOMENT-CURVATURE CURVE IDEALIZED TO ELASTIC PERFECTLY PLASTIC RESPONSE AS PER AASHTO SEISMIC GUIDE 8.5

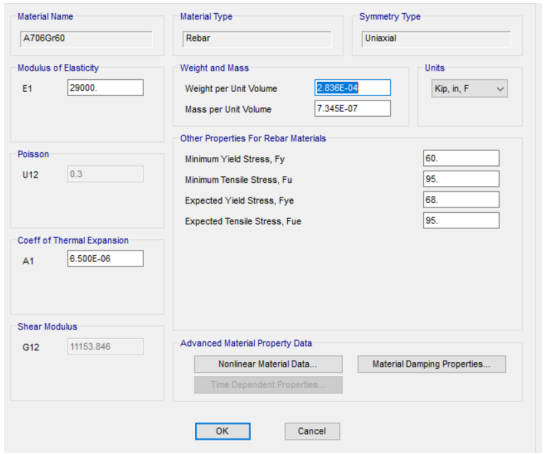
6. BOTH ELASTIC AND PLASTIC DISPLACEMENT CAPACITIES CAN ALSO BE ESTIMATED FROM MOMENT CURVATURE ANALYSIS.

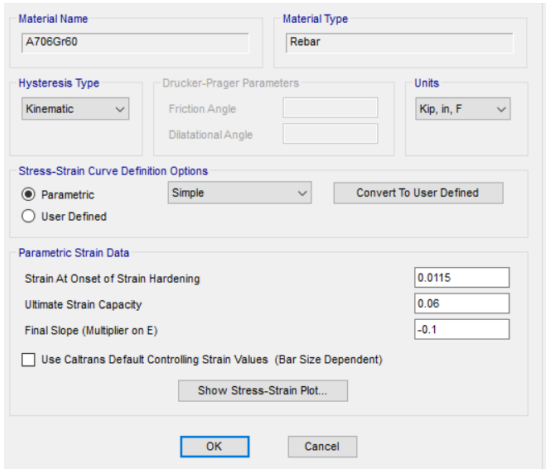
7. DUCTILITY CAPACITY CALCULATED FROM MOMENT CURVATURE ANALYSIS IS COMPARED WITH MAXIMUM DUCTILITY DEMAND FROM MAXIMUM PIER 1 MOVEMENT FROM ELASTIC DYNAMIC ANALYSIS OF THE BRIDGE.

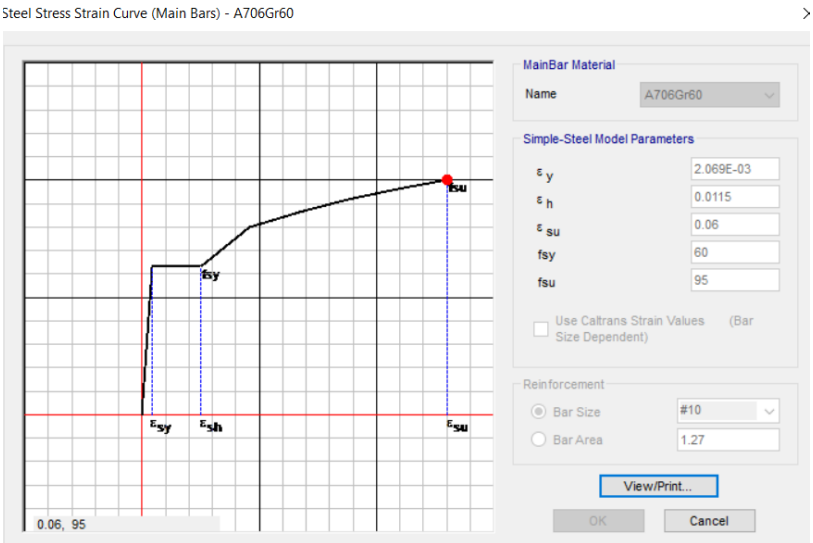
MATERIAL MODELLING











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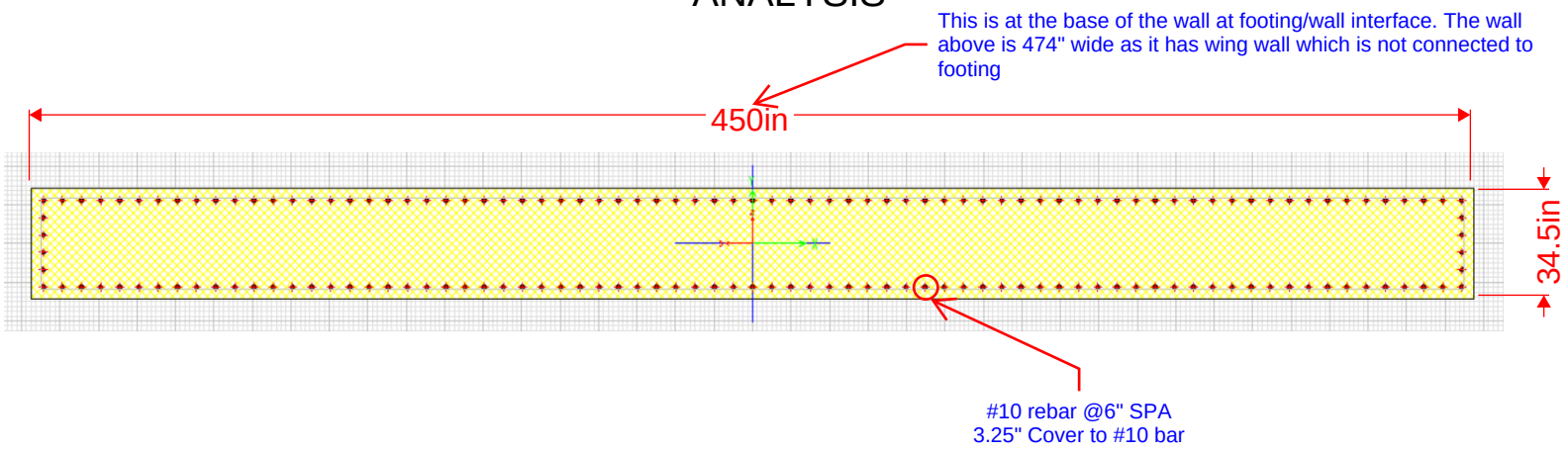
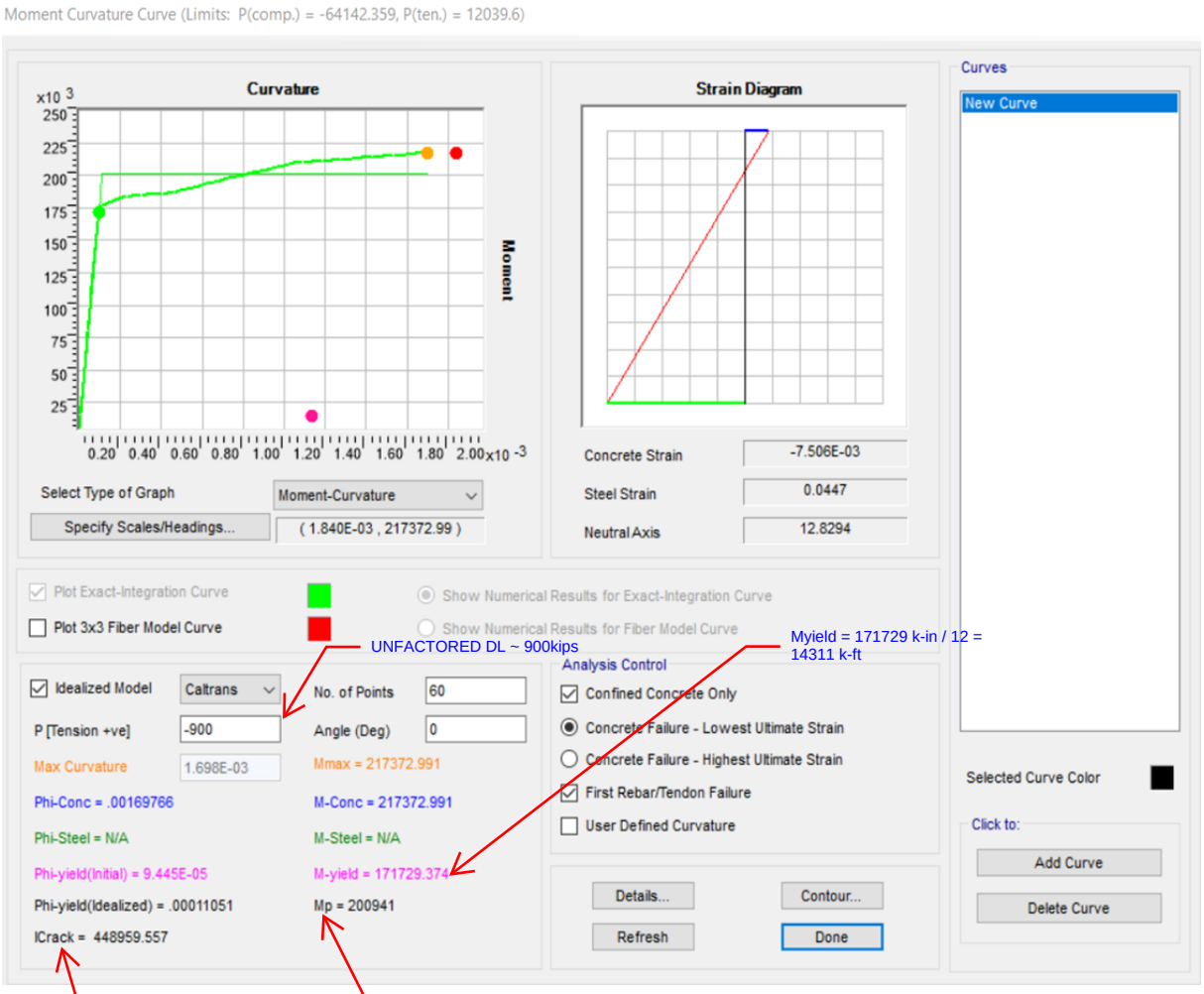
PROJECT WSDOT i405 - BR17.7 CONT

SUBJECT MOMENT - CURVATURE ANALYSIS PIER 1 PAGE

CHECKED BY MWBM DATE 29 NOV 2021 CALCULATIONS BY RSGR DATE 29 NOV 2021

WALL MODELLING AND MOMENT CURVATURE ANALYSIS





Mp = 200941 k-in / 12 = 16745 k-ft

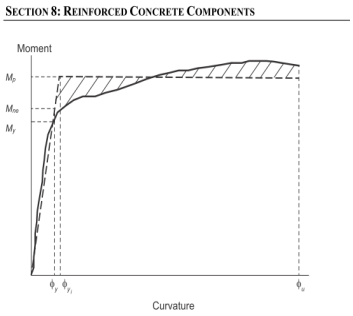
ICrack / I gross = 448959 / 1539886 = 0.30

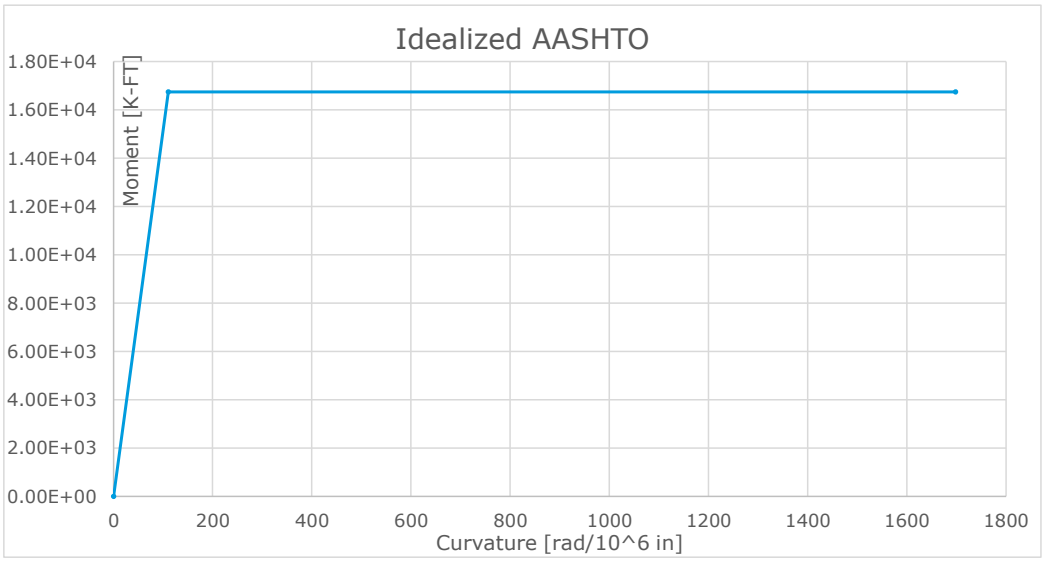
φ_first rebar yield=	9.45E+01	(rad/10^6 in)	From M-curvature analysis, see previous page
M_first rebar yield =	14311	k-ft	From M-curvature analysis, see previous page
EI_first rebar yield =	1.52E+02	k-ft/(rad/10^6in)	
φu =	1.70E+03	(rad/10^6 in)	From M-curvature analysis, see previous page

Moment -curvature

Mp =	16745	k-ft
φyi=	110.5162	(rad/10^6 in)
φu =	1.70E+03	(rad/10^6 in)
φpd=	1587.484	(rad/10^6 in)

Φ	M_idealized
rad/10^6	k-ft
0	0.00E+00
1.11E+02	1.67E+04
1.70E+03	1.67451E+04





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Project	I-405	**Project No**		A207833
Title	Displacement Capacity - Pier 1	**Date**	11/29/2021	
Author	RSGR	**Checker**	MWBM	

Longitudinal displacement capacity of column

Elastic displacement

L =	19.50	ft, Average height of Pier 1 Abutment Wall	
φyi=	0.0001	(rad/10^6 in), from AASHTO curve	
Δy =	2.02	in, φyi x L^2/3 elastic displacement	AASHTO Seismic Eq.C4.9-3

Plastic displacement

Longitudinal

Analytical plastic hinge

L =	19.5	ft	
	234.0	in	
fye =	68	ksi	AASHTO Seismic Table 8.4.2-1
dbl =	1.27	in	#10rebar
0.08L=	18.7	in	
0.15fye*dbl =	12.95	in	
Lp =	31.7	in	AASHTO Seismic Eq. 4.11.6-1
Lp(min)=	25.91	in	
Lp=	31.67	in	
φpd=	0.0016	(rad/10^6 in)	
θp =	0.050	rad, φpdxL	
L - Lp/2=	218.16	in,	
Δp=	10.97	in,	

Total displacement capacity

Δy =	2.02	in,
Δp=	10.97	in,
Δ.total =	12.99	in,
Results:	6%	Drift

Ductility Capacity=	6.44	=1+(Δp/Δy)
Total Displacement Capacity of Column	12.99	in

Overstrength Results:

M0	20094	k-ft	(1.2*Mp AASHTO Seismic Eq 8-5.1); Grade A706
H	19.50	ft	average height of Pier 1 wall
Vo	1030	Kips	(M0/H)

Ductility demand=

Max. Pier 1 movement, Δ=	2.55	in	From EtoW analysis of bridge		
θ col =	0.0117	=Δ"/(L - Lp/2)			
θ elastic =	0.0092	=Δy"/(L - Lp/2)			
θ plastic =	0.0024	=θ col - θ elastic			
Δp=	0.5329		Ductility Capacity		
Ductility demand=	1.26	=1+Δp/Δy	<	6.44	OK

PIER 1 : SHEAR KEY AT TOP WITH #9 HINGE DOWEL BAR @6" SPA.

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 1 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Purpose:
Design the hinge rebar (dowel) at Pier 1 to EXT Loads (DL; LL; EQ)
Design Code:
AASHTO LFRD 9th Edition Chapter 5.13 which refers to ACI 318 - 14 Chapter 17
AASHTO LFRD 9th Edition Chapter 5.7.4 for Interface shear
AASHTO SEISMIC DESIGN GUIDE
Asumptions:
1. Hinge rebar is a shear connector. It is designed to transfer shear force between superstructure and substructure inc. transfer of torsion from supersturcture as tension through the hinge rebars.
2. Total loads from superstructure/substructure are distributed per dowel to calculated max. load effect per dowel. i.e. Max Shear and Max. Tension in the critical dowel which is the dowel at the end
3. Required no. of dowels are first assumed and verified so that load demand per dowel is less than dowel capacity. The following failure modes are checked:
a) Steel failure in tension and shear
b) Concrete failure due to tension and shear

4. The transfer of shear force between superstructure and substructure will als ooccur via concrete interface shear in the shear key
Hence, concrete shear key is then detailed to have enough capacity for the interface shear transfer

input

1.0 DIMENSIONS & PARAMETERS

nominal f'c	4	ksi	expected f'ce	5.20	ksi		concrete compressive strength (nominal and expected)
wc =	0.155	kcf					Unit weight of weight concrete
K1 =	1						Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4028	ksi					Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
fy =	60	ksi	fye =	68.00	ksi		Yield strength of reinforcement (nominal and expected)
f,uta=	90	ksi	f,e uta=	95	ksi	AASHTO seismic Table 8.4.2.1	Ultimate tensile strength of reinforcement (nominal and expected)
Es =	29000	ksi					Modulus of elasticity of reinforcement
η =	7.20						Modular ratio
β_1 =	0.85						Compression Zone Factor
φv=	1						
φf=	1						
Dowel Qty., N=	72	Nos.					

2.0 Spacing of dowels

Length of Wall/Abut Diaphragm=	37.5	ft	
Edge distance from CL of anchor=	12	in	the smallest edge distance in long. Direction, this value is 24in in transverse direction for edge most bar
Spacing of anchors=	6.0	in	
Use=	6.0	in	

3.0 Horizontal Force demands (between superstructure and substructure)

Main Shear force in Bridge Longitudinal Direction **100%L + 30%T**

LEP EQ Demand, L1=	922	kips	**(Seismic EtoW) is the governing case**
EDA EQ Demand, L2=	1991	kips	From Seismic earth pressure behind Pier 3,
LS, EQ Demand, L3=	24	kips	From RSA **(Seismic EtoW)**, applied with a factor of 1.2
Live Load LL, L4=	164	kips	From Live Load Surcharge behind Pier 3 (0.5 factor used)
			Apply a factor of 0.5 in combination below

Associated Transverse shear forces (if any)

LEP Demand, T1=	36	kips	From Seismic earth pressure behind Pier 3
EDA Demand, T2=	565	kips	From RSA **(Seismic EtoW)**, applied with a factor of 1.2
LS Demand, T3=	0	kips	From Live Load Surcharge behind Pier 3 (0.5 factor used)
Live Load LL, T4=	70	kips	Apply a factor of 0.5 in combination below

Combined shear using SRSS

1=	923	kips
2=	2069	kips
3=	24	kips
4=	89	kips
Total=	3105	kips

Shear per dowel=	43.1	kips	Total shear load / nos. of dowels
Shear per dowel (transverse only)=	9.3	kips	
Shear per dowel (longitudinal only)=	41.8	kips	

Main Shear force in Bridge Transverse Direction **30%L + 100%T**

LEP Demand, T1=	36	kips
EDA Demand, T2=	1490	kips
LS Demand, T3=	0	kips
Live Load LL, T4=	70	kips

Associated Longitudinal shear forces (if any) 30%L + 100%T

LEP EQ Demand, L1=	277	kips
EDA EQ Demand, L2=	732	kips
LS, EQ Demand, L3=	24	kips
Live Load LL, L4=	164	kips

Combined shear using SRSS

1=	279	
2=	1660	
3=	24	
4=	89	
Total=	2053	kips

Shear per dowel=	28.5	kips	
Shear per dowel (transverse only)=	15.5	kips	
Shear per dowel (longitudinal only)=	21.7	kips	

4.0 Vertical Demands - Compression

DL=	510	kips	
LL=	169	kips	Apply a factor of 0.5 in combination
EQ=	-437	kips	(100%L + 30%T - 30% V)
Min. Compression under EQ =	157.7	kips	
Compression per dowel=	2.19	kips	

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PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 1 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

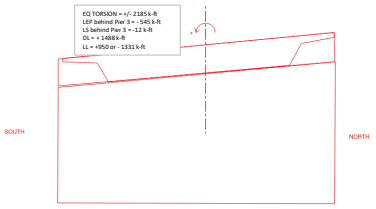
JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

5.0 Vertical Demands - Tension due to superstructure torsion

EQ (This can be + or -) =	2185	k-ft
Seismic Earth pressure behind Pier 3=	-454	k-ft
Live load surcharge behind Pier 3=	-12	k-ft
DL=	1488	k-ft
Max Torsion LL=	950	k-ft
Min Torsion, LL=	-1331	k-ft

From RSA **(Seismic EtoW)**, applied with a factor of 1.2

Apply a factor of 0.5 in combination
Apply a factor of 0.5 in combination



Max Torsion, Tmax	3694	k-ft
Min Torsion, Tmin	-1829	k-ft

Estimate the tension forces caused by torsion

Assumption: Torsion causes tension force in half of the dowel

Half nos. of dowels=	36	
Spacing=	6.0	in

Dowels experience tension force relative to their distance S from structure CL

Tension force in dowel = Tmax x Si / ∑(S^2); Tmax = 3603 kip/ft

	Si (ft)	S^2	Tension (kips)	Compression	Net Force (-ve is tension)	
S1	0.25	0.0625	-0.238	2.19	1.953	
S2	0.75	0.5625	-0.713	2.19	1.478	
S3	1.25	1.5625	-1.188	2.19	1.002	
S4	1.75	3.0625	-1.663	2.19	0.527	
S5	2.25	5.0625	-2.138	2.19	0.052	
S6	2.75	7.5625	-2.613	2.19	-0.423	
S7	3.25	10.5625	-3.089	2.19	-0.898	
S8	3.75	14.0625	-3.564	2.19	-1.373	
S9	4.25	18.0625	-4.039	2.19	-1.849	
S10	4.75	22.5625	-4.514	2.19	-2.324	
S11	5.25	27.5625	-4.989	2.19	-2.799	
S12	5.75	33.0625	-5.464	2.19	-3.274	
S13	6.25	39.0625	-5.940	2.19	-3.749	
S14	6.75	45.5625	-6.415	2.19	-4.225	
S15	7.25	52.5625	-6.890	2.19	-4.700	
S16	7.75	60.0625	-7.365	2.19	-5.175	
S17	8.25	68.0625	-7.840	2.19	-5.650	
S18	8.75	76.5625	-8.315	2.19	-6.125	
S19	9.25	85.5625	-8.791	2.19	-6.600	
S20	9.75	95.0625	-9.266	2.19	-7.076	
S21	10.25	105.0625	-9.741	2.19	-7.551	
S22	10.75	115.5625	-10.216	2.19	-8.026	
S23	11.25	126.5625	-10.691	2.19	-8.501	
S24	11.75	138.0625	-11.166	2.19	-8.976	
S25	12.25	150.0625	-11.642	2.19	-9.451	
S26	12.75	162.5625	-12.117	2.19	-9.927	
S27	13.25	175.5625	-12.592	2.19	-10.402	
S28	13.75	189.0625	-13.067	2.19	-10.877	
S29	14.25	203.0625	-13.542	2.19	-11.352	
S30	14.75	217.5625	-14.017	2.19	-11.827	
S31	15.25	232.5625	-14.493	2.19	-12.302	
S32	15.75	248.0625	-14.968	2.19	-12.778	
S33	16.25	264.0625	-15.443	2.19	-13.253	
S34	16.75	280.5625	-15.918	2.19	-13.728	
S35	17.25	297.5625	-16.393	2.19	-14.203	
S36	17.75	315.0625	-16.868	2.19	-14.678	Max. tension in edge dowel

Total, ∑(S^2)= 3887.25 ft

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PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 1 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

6.0 Verify Minimum Nos. of Dowel - Single Anchor shear capacity - Vsa

Dowel Size #=	9	#				
Dowel Area, Ad=	1	in2				
Dowel diam., d=	1.128	in				
Expected Dowel Utlimate Tensile Strength, f-ue=	95	ksi				
Shear Resistance Factor, φ dowel=	1					
Nominal Resistance, Vn=	57	kips		Max. Demand	D/C	=0.6x Ad x futa ACI 318 eq 17.5.1.2b
Factored Resistance, Phi x Vn=	57	kips	>	43	76%	

7.0 Verify Minimum Nos. of Dowel - Single Anchor Tensile capacity = Nsa

Dowel Size #=	9	#				
Dowel Area, Ad=	1	in2				
Dowel diam., d=	1.128	in				
Expected Dowel Utlimate Tensile Strength, f-ue=	95	ksi				
Tensile Resistance Factor, φ dowel=	1.00					
Nominal Resistance, Tn=	95	kips		Max. Demand	D/C	= Ad x futa ACI 318 eq 17.4.1.2
Factored Resistance, Phi x Vn=	95	kips	>	15	15%	

8.0 Verify Layout of Anchors in Concrete - Single Anchor

8.1 Concrete breakout due to tension - Ncb

C,a1=	12	in	Min. edge distance in longitudinal direction	
H,ef used.=	8.000	in	Use this for tension breakout calcs	ACI 318 cl17.4.2.3
H,ef prov.=	36	in	embedment length into concrete	
A, Nc=	576	in2	Effective concrete area around the edge anchor	ACI 318 cl17.4.2.1c
A, Nco=	576	in2	Projected area of single anchor = 9 x hef x hef	ACI 318 Fig R17.4.2.1
Nb	39	kips	Basic breakout strength of single anchor in tension	ACI 318 CL17.4.2.2a
kc	24		cast in anchor	
gamma a	1		aggregate factor	
sqrt f'c	72.111			
hef	8.000	in		
ψ, ed,N=	1		ca, min = 12in $\geq$ 1.5 h,ef = 12in	
ψ, c,N=	1.25		cast-in-anchors	
ψ, cp,N=	1		cast-in-anchors	
Factor=	1.00			
Factored Concrete breakout, Ncb=	49	kips	>	Max. Demand 15 / D/C 30%

8.2 Concrete breakout due to Shear at edge dowel Vcb

Shear in bridge long. direction

Ca1=	24	in	min. distance to edge parallel to edge	
Ca2=	12	in	min. distance to edge perpendicular to edge	
Avco=	648	in2	use smaller Ca	ACI 318 eq 17.5.2.1c
le=	9.024	in	le=he=load bearing length of dowel	ACI318 17.5.2.2
da=	1.128	in		diameter of dowel
sqrt f'c=	72.11			
Vb=	33.78	kips	use smaller Ca	ACI 318 eq 17.5.2.2a
Limit Vb=	26.98	kips	use smaller Ca	ACI 318 eq 17.5.2.2b
Use limit				
hef=	18	in		effective embedment for shear breakout = 1.5 x Min (Ca1, Ca2)
ha=	40	in		Bearing height
Avc=	756	in2	single anchor at edge	
ψ, ed,V=	1			
ψ, c,V=	1.2		cracking at ext loads	
ψ, h,V=	1		ha=1.5ca	

Shear Resistance Factor, φ = 1

Max. Trans. Demand 43 / D/C 114%

Factored Concrete breakout, Vcb= 37.8 kips

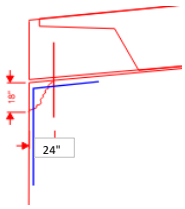
ACI 318 CL17.5.2.1a

Add supplementary rebar at edge

Bar #	6		atleast 2 nos. #6 bars in wall top
Qty.	2		
Area of steel, As=	0.88	in2	
f'ye	68.000	ksi	expected steel yield strength
Nominal Steel Strength, Vs=	59.84	kips	=f'ye x As
Factor, φ=	1		
Factored steel strength, f x Vs=	60	kips	

Max. Trans. Demand 43 / D/C 44%

Total Factored Strength, Concrete + Steel, = 97.6 kips



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PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 1 Shear Key at Top (EXT)
BY: RSGR
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DATE: 2021/11/29
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8.3 Concrete pryout due to Shear at edge dowel Vcp

Vcp=kcp x Ncb — ACI 318 eq. 17.5.3.1a

kcp=	2	
Ncb=	49.0	kips
Factor, φ=	1	
Factored Vcp=	97.9	kips
Factored Demand, Vu=	43.1	kips
D/C=	44%	

8.4 Shear and Tension Interaction

Max Shear D/C, Vdc=	76%		
Max Tension D/C, Tdc=	30%		
	0.763	OK	$Vdc^{5/3} + Tdc^{5/3} < 1.0$

ACI 318 R17.6

9.0 Verify Layout of Anchors in Concrete - Group Anchor

9.1 Concrete breakout due to tension - Ncbg

C,a1=	12	in	Min. edge distance in longitudinal direction
H,ef used.=	8.000	in	Use this for tension breakout calcs — ACI 318 cl17.4.2.3
H,ef prov.=	36	in	embedment length into concrete
A, Nc=	10800	in2	Effective concrete area around the edge anchor — ACI 318 cl17.4.2.1c
A, Nco=	576	in2	Projected area of single anchor = 9 x hef x hef — ACI 318 Fig R17.4.2.1
Nb	39	kips	Basic breakout strength of single anchor in tension — ACI 318 CL17.4.2.2a
kc	24		cast in anchor
gamma a	1		aggregate factor
sqrt f'c	72.111		
hef	8.000	in	
ψ, ed,N=	1		ca, min = 12in ≥ 1.5 h,ef = 12in
ψ, ec,N=	1		cast-in-anchors
ψ, cp,N=	1		cast-in-anchors
ψ, c,N=	1.25		
Factor=	1.00		

			Max. Tension Demand	D/C
Factored Concrete breakout, Ncb=	734	kips	> 150	20%

9.2 Concrete breakout due to Shear at edge dowel Vcbg - Shear in bridge long. Direction

Shear in bridge long. direction

Ca2=	24	in	min. distance to edge parallel to edge
Ca1=	12	in	min. distance to edge perpendicular to edge
s=	6.0	in	
Avco=	648	in2	ACI 318 eq 17.5.2.1c
le=	9.024	in	le=he=load bearing length of dowel
da=	1.128	in	diameter of dowel
sqrt f'c=	72.11		
Vb=	33.78	kips	ACI 318 eq 17.5.2.2a
Limit Vb=	26.98	kips	ACI 318 eq 17.5.2.2b
Use limit			
he=	18	in	note that the actual embedment is 40" — effective embedment for shear breakout = 1.5 x Ca1
Avc=	8316	in2	group of anchors
ψ, ec,V=	1		
ψ, ed,V=	1		Ca2 24" > 1.5Ca1 18"
ψ, c,V=	1.2		cracking at ext loads
ψ, h,V=	1		ha=1.5ca

			Max. long. Demand	D/C
Shear Resistance Factor, φ =	1			
Factored Concrete breakout, Vcb=	415.5	kips	3105	747%

ACI 318 CL17.5.2.1b

Check for main wall reinforcement at wall face to resist concrete breakout

Bar #	10		#10 bars at 6" throughout wall length ~ 75 nos.
Qty.	75		
Area of steel, As=	95.25	in2	
f'ye	68.000	ksi	expected steel yield strength
Nominal Steel Strength, Vs=	6477	kips	=f'ye x As
Factor, φ=	1		
Factored steel strength, f x Vs=	6477	kips	

			Max. Trans. Demand	D/C
Total Factored Strength, Concrete + Steel, =	6892.5	kips	3105	45%

Hence, main wall reinforcement at the face of the wall will prevent concrete breakout in bridge long. Direction

10.0 Interface shear resistance (AASHTO LRFD BDS 2017, 5.7.4)

This happens in superstructure where f'c = 5 ksi

- For normal weight concrete placed monolithically:

c = 0.40 ksi
μ = 1.4
K_1 = 0.25
K_2 = 1.5 ksi

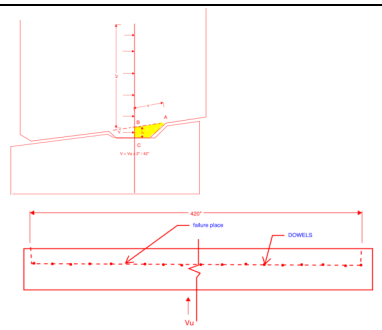
cohesion factor	c=	0.4	ksi	
Friction factor	u=	1.4		
Fraction of concrete strength	K1=	0.25		
Limiting interface shear resistance	K2=	1.5	ksi	
	b	5	in	(3" on top and 2" in height)
	L	450	in	Length of shear key failure plane
Area of concrete in interface	Acv=	2250	in2	b x L
Nominal Shear Resistance of interface	Vni=	900	kips	c x Acv
	fye=	0	ksi	Rebar Steel strength (nominal)
	Avf=	0	in2	Steel reinforcement area in shear key
	Pc=	0	kips	No compression assumed
Factor=		1		

				Max. Demand		D/C
Factored resistance=	900	kips	>	688	=3105 x 2"/le"	76%

Max Limits for resistance

				Max. Demand	D/C
AASHTO LRFD BDS 5.7.4.3-4	V1	2813	kips	> 688	24%
AASHTO LRFD BDS 5.7.4.3-5	V2	3375	kips		

Hence, concrete shear key shall will not fail and the failure will be governed by checks in section 6 to 9



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PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I7 CALC BOOK COMPLETION DATE: **2021 Dec 05**

Title: Existing Pier 2 - Design Checks

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I7-0	-	I7-0	Calc Register (this sheet)	N/A	N/A	
2	I7-1	-	I7-3	Pier 2 Wall - Design Forces	TOTG	RSGR	
3	I7-4	-	I7-6	Pier 2 Wall - Design Check (Bi-Axial)	TOTG	RSGR	
4	I7-7	-	I7-17	Pier 2 Wall - Design Check - About wall weak axis	TOTG	RSGR	
5	I7-18	-	I7-27	Pier 2 Wall - Design Check - About wall strong axis	TOTG	RSGR	
6	I7-28	-	I7-30	Pier 2 Footing - Design Forces	TOTG	RSGR	
7	I7-31	-	I7-36	Pier 2 Footing - Geotechnical, Flexure and Shear Capacity Check - Bending about wall weak axis	TOTG	RSGR	
8	I7-37	-	I7-42	Pier 2 Footing - Geotechnical, Flexure and Shear Capacity Check - Bending about wall strong axis	TOTG	RSGR	
9	I7-43	-	I7-44	Pier 2 Footing - Geotechnical Capacity Check (Bi-Axial)	TOTG	RSGR	
10	I7-45	-	I7-49	Pier 2 - Top Shear Key Design (EXT CASE)	RSGR	LDLI	
11							
12							
13							
14							
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Pier 2 Wall - Design Forces

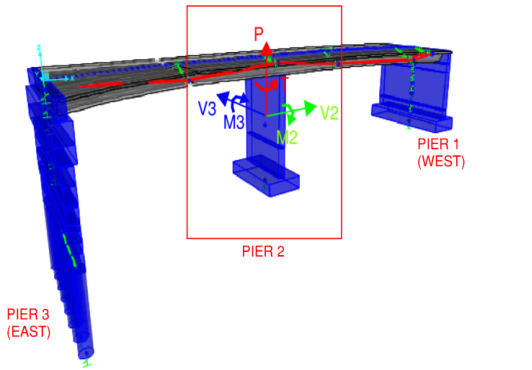
PROJECT: BR 17.7 - PIER 2 (Median Pier)
SUBJECT: Wall Design: Factored Limit States Combinations
BY: TOTG
CHK: RSGR

Purpose:
Tabulate factored loads from unfactor loads determined by the Extreme, Strength and Service limit states.

Method:
1. Limit states per AASHTO LFRD 9th Edition

Assumptions:
1. Strong bending uses EQ limit state factor of 1.0 and weak bending uses EQ limit state factor of 1.2.

Direction of Force Effects and Sign Convention



Demands - At bottom of wall

Load Cases		P (kips)	V2 (kips)	V3 (kips)	T (kip-ft)	M2 (kip-ft)	M3 (kip-ft)	
DC		-1290	3	9	-3	-162	85	
LL	Min	0	20	-44	18	481	655	
	Max	-310	-22	14	-10	-697	-708	
PS		31	3	-8	-1	20	108	
WS - STR III	Min	50	9	-12	4	288	139	
	Max	-50	-9	12	-4	-288	-139	
WS - STR V	Min	1	6	-4	2	123	103	
	Max	-1	-6	4	-2	-123	-103	
WS - SER I	Min	1	5	-3	1	94	79	
	Max	-1	-5	3	-1	-94	-79	
WL	Min	0	0	-8	4	217	10	
	Max	0	0	8	-4	-217	-10	
TG	Min	101	-2	-14	2	378	-52	
	Max	-30	0	4	-1	-113	16	
TU	Min	4	-11	-13	0	334	-370	
	Max	-5	5	22	-1	-571	151	
EH - At Rest		80	4	-30	-7	934	132	
SE - Pier 1		-26	1	0	0	-25	30	
CR + SH		-70	3	6	-13	-233	86	
LEP_Pier3		-109	42	44	0	-1369	1370	
LS_Pier3		-3	1	1	0	-35	36	
100%L+30%T+30%V	Max	-533	154	180	-23	-4778	2900	EQ East
100%L-30%T-30%V	Min	533	154	-180	23	4778	2900	to West
30%L+100%T+30%V	Max	-566	53	425	-65	-10931	968	
30%L-100%T-30%V	Min	566	53	-425	65	10931	968	
LEP_Pier1		-23	-2	10	0	-323	-80	
LS_Pier1		-1	0	1	0	-23	-6	
100%L+30%T+30%V	Max	-978	129	224	-4	-4922	1319	EQ West
100%L-30%T-30%V	Min	978	129	-224	4	4922	1319	to East
30%L+100%T+30%V	Max	-703	73	322	-5	-6822	754	
30%L-100%T-30%V	Min	703	73	-322	5	6822	754	

Load Combos:

Load Factors (AASHTO Table 3.4.1-1)

	DC		PS		EH		CR + SH		LL	WS	TU	TG	SE	EQ	
	Max	Min	Max	Min	Max	Min	Max	Min						Strong-M2	Weak-M3
Strength I	1.25	0.9	1.25	0.9	1.25	0.9	1.25	0.9	1.75	0	0.5	0	1	0	0
Strength III	1.25	0.9	1.25	0.9	1.25	0.9	1.25	0.9	0	1	0.5	0	1	0	0
Strength V	1.25	0.9	1.25	0.9	1.25	0.9	1.25	0.9	1.35	1	0.5	0	1	0	0
Extreme I	1	1	1	1	1	1	1	1	0.5	0	0.5	0	1	1	1.2
Service I	1	1	1	1	1	1	1	1	1	1	0.5	0.5	1	0	0

Factored Loads:

	P (kips)		V2 (kips)		V3 (kips)		T (kip-ft)		M2 (kip-ft)		M3 (kip-ft)		
	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	
Strength I	-2132	-1148	55	-31	-112	15	2	-41	1683	-1027	1766	-1024	
Strength III	-1640	-1098	21	-2	-47	2	-26	-26	1129	-95	758	76	
Strength V	-2058	-1098	56	-32	-107	21	-48	6	1778	-841	1643	-880	
EXT I EtoW	-2262	-824	250	221	239	-111	20	-49	3399	-7198	5580	4638	100%L±30%T±30%V
EXT I EtoW	-2225	-707	99	70	502	-387	70	-100	10523	-12381	2290	1348	30%L±100%T±30%V
EXT I WtoE	-2709	-203	175	146	259	-189	-3	-27	4594	-6290	2212	1270	100%L±30%T±30%V
EXT I WtoE	-2363	-516	109	80	368	-294	-2	-29	6729	-7956	1592	650	30%L±100%T±30%V
Service I	-1653	-1173	46	-24	-84	7	-1	-39	1659	-793	1319	-617	

Pier 2 Wall - Biaxial Flexure Check

PROJECT: BR 17.7 - PIER 2 (Median Pier)
SUBJECT: Wall Design: Bi-Axial
BY: TOTG
CHK: RSGR

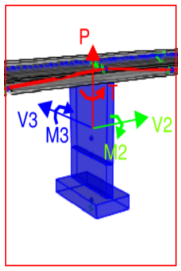
Purpose:
Design check of the as-built wall for strength, service and extreme limit states in Bi-Axial loading.

Methodology:
1. Combine capacity to load ratios and verified per AASHTO LRFD 9th edition for Bi-axial flexure.

Asumptions:
1. Critical section of wall occurs at the base of the wall.

Dimensions of Wall:

Longitudinal Thickness=	2.5	ft	Direction of 2
Transverse Thickness=	24	ft	Direction of 3



PIER 2

Bi-Axial Factored Loads:

	P (kips)		V2 (kips)		V3 (kips)		T (kip-ft)		Strong Bending M2 (kip-ft)		Weak Bending M3 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min
Strength I	-2132	-1148	55	-31	-112	15	2	-41	1683	-1027	1766	-1024
Strength III	-1640	-1098	21	-2	-47	2	-26	-26	1129	-95	758	76
Strength V	-2058	-1098	56	-32	-107	21	-48	6	1778	-841	1643	-880
100%L+30%T EXT I EtoW	-2262	-824	250	221	239	-111	20	-49	3399	-7198	5580	4638
30%L+100%T EXT I EtoW	-2225	-707	99	70	502	-387	70	-100	10523	-12381	2290	1348
100%L+30%T EXT I WtoE	-2709	-203	175	146	259	-189	-3	-27	4594	-6290	2212	1270
30%L+100%T EXT I WtoE	-2363	-516	109	80	368	-294	-2	-29	6729	-7956	1592	650
Service I	-1653	-1173	46	-24	-84	7	-1	-39	1659	-793	1319	-617
									EQ factor = 1.0		EQ factor = 1.2	

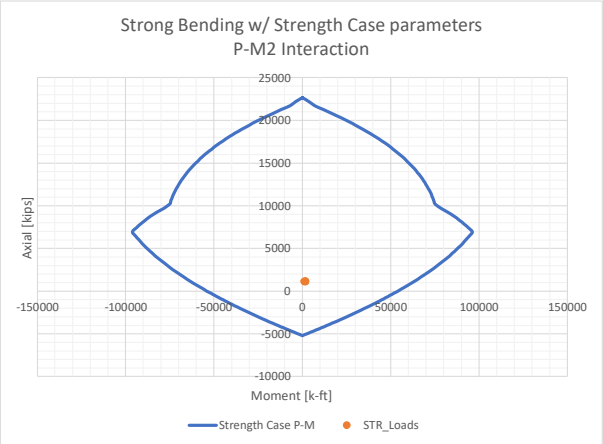
Strength Limit State

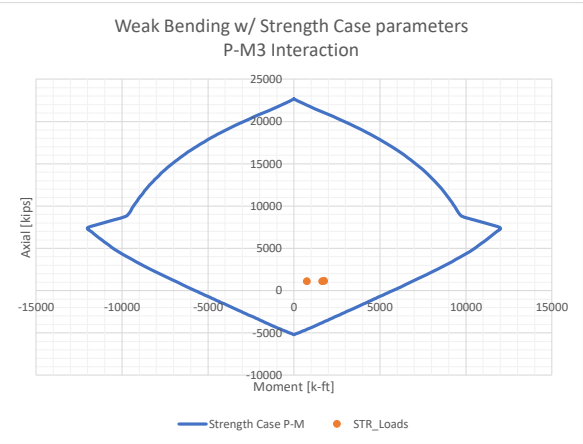
Biaxial Flexure

References:

AASHTO LFRD 9TH EDITION

User Input=			
resistance factor, φ(flexure)=	0.9		
design concrete compressive strength, f'c=	4	ksi	
gross area, Ag=	8640	in2	
0.1*φ*f'c*Ag=	3110	kips	5.6.4.5
Max STR factored axial load, Nu=	2132	kips	Max Axial Load is less, use 5.6.4.5-3





	Min P	Abs Max M2	Abs Max 3		Capacity M2	Capacity M3	D/C
	kips	k-ft	k-ft				
Strength I	1148	1683	1766	Case 1	63369	6959	28%
Strength III	1098	1129	758	Case 2	62983	6907	13%
Strength V	1098	1778	1643	Case 3	62983	6907	27%

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}$$ 5.6.4.5-3

Extreme Limit State

Biaxial Flexure

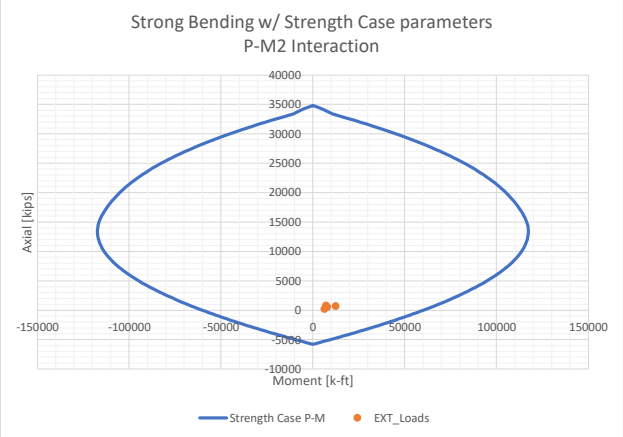
References:

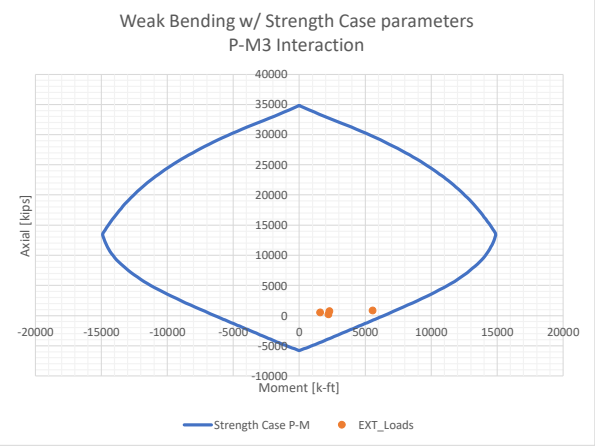
AASHTO LFRD 9TH EDITION

User Input=		
resistance factor, φ(flexure)=	1	
design concrete compressive strength, f'c=	5.2	ksi
gross area, Ag=	8640	in2
0.1*φ*f'c*Ag=	4493	kips
factored axial load, Nu=	2709	kips

5.6.4.5

Max Axial Load is less, use 5.6.4.5-3





	Min P	Abs Max M2	Abs Max 3		Capacity M1	Capacity M3	D/C
	kips	k-ft	k-ft				
100%L+30%T EXT I EtoW	824	7198	5580	Case 4	66858	7262	88%
30%L+100%T EXT I EtoW	707	12381	2290	Case 5	65938	7140	51%
100%L+30%T EXT I WtoE	203	6290	2212	Case 6	61750	6608	44%
30%L+100%T EXT I WtoE	516	7956	1592	Case 7	64353	6939	35%

$$\frac{M_{ux}}{M_{rx}} + \frac{M_{uy}}{M_{ry}}$$ 5.6.4.5-3

Pier 2 Wall - Design Check about wall weak axis

PROJECT:	BR 17.7 - PIER 2 (Median Pier)
SUBJECT:	Wall Design: About Weak Axis
BY:	TOTG
CHK:	RSGR

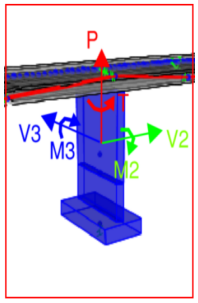
Purpose:
Design check of the as-built wall for strength, service and extreme limit states under longitudinal loading.

Methodology:
1. Strength and service and extreme case limit states are checked per AASHTO LRFD 9th Edition and per AASHTO LRFD Seismic.

Asumptions:
1. Critical section of wall occurs at the base of the wall.

Dimensions of Wall:

Longitudinal Thickness=	2.5	ft	Direction of 2
Transverse Thickness=	24	ft	Direction of 3



PIER 2

Strength Limit State

Demand:

	P (kips) Max	P (kips) Min	V2 (kips) Max	V2 (kips) Min	T (kip-ft) Max	T (kip-ft) Min	M3 (kip-ft) Max	M3 (kip-ft) Min
Strength I	-2132	-1148	55	-31	2	-41	1766	-1024
Strength III	-1640	-1098	21	-2	-26	-26	758	76
Strength V	-2058	-1098	56	-32	-48	6	1643	-880
Service I	-1653	-1173	46	-24	-1	-39	1319	-617

Flexural Design: Strength, about wall weak axis

References:

User Input=					**AASHTO LFRD 9TH EDITION**
direction 2 reinforcement bar size=	6				Note: horizontal bars
bar dia=	0.75				
resistance factor, φ(flexure)=	0.9				5.5.4.2
steel yield, fy=	60	ksi			5.6.2.1
design concrete compressive strength, f'c=	4	ksi			
Cover=	1	in			As built
bar size=	10	#			As Built
spacing=	8	in			As Built
bar dia=	1.27	in			
bar area=	1.27	in2			
Thickness of wall, h=	30	in			
Width of Wall, b=	288	in			
Number of reinforcement bars=	37	nos.			
Area of tensile steel, As=	47.0	in2			
distance from extreme comp. fiber to o.c. of tensile steel, ds=	27.1	in			
Area of compression steel, A's=	47.0	in2			Note: Per face, symmetric
distance from extreme comp. fiber to o.c. of tensile steel, d's=	2.89	in			
stress block factor, α1=	0.85				5.6.2.2
stress block factor, β1=	0.85				5.6.2.2
distance from extreme fiber to NA, c=	0.000	in			5.6.3.1.2-4
depth of equivalent stress block, a=	0.000	in			
Nominal flexural resistance, Mn=	5693	kip-ft			5.6.3.2.2-1
Factored flexural resistance, φMn=	5124	kip-ft	Capacity	Demand<Capacity OK	5.6.3.2.1-1
Factored Moment Load, Mu=	1766	kip-ft	Demand		5.6.3.2.1-1

Minimum Reinforcement

correction factor for source of aggregate, K1=	1				5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.145	kcf			Table 3.5.1-1
concrete density modification factor,λ=	1				C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.480	ksi			5.4.2.6
flexural cracking vairability facgtor, γ1=	1.6				
ratio of specified yield to ultimate strength, γ3=	0.67				
section modulus, Sc=	43200	in3			
Cracking Moment, Mcr=	1852	kip-ft			5.6.3.3-1
1.33 times factored moment required, 1.33 x Mu=	2348	kip-ft			5.6.3.3-1
Lesser of (Mu, Mcr)=	1852	kip-ft	Demand	Demand<Capacity OK	5.6.3.3-1

Torsion Investigation: Strength, about wall weak axis

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(torsion)=	0.9				5.5.4.2
design concrete compressive strength, f'c=	4	ksi			
Thickness of wall, h=	30	in			
Width of Wall, b=	288	in			
area enclosed by outside perimeter of cross section, Acp=	8008	in2			5.7.2.1-4
length of outside perimet of the cross section, pc=	628	in			5.7.2.1-4
Factored axial force, Nu=	1098	kips			
gross area, Ag=	8640	in2			
Factored Axial stress, Nu/Ag=	0.127	ksi			5.7.2.1-6
concrete density modification factor, λ=	1				
K=	0.70				5.7.2.1-6
Torsional cracking moment, Tcr=	1510	kip-ft			5.7.2.1-4
0.25*φ*Tcr=	340	kip-ft	Capacity	Torsional investigation is not required	
Factored Torsion Load, Tu=	48	kip-ft	Demand		5.7.2.1-3

Transverse Reinforcement Requirements: Strength, about wall weak axis

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Transverse Reinforcement					
Thickness of wall, h=	30	in			
adjusted width, bv=	288	in			5.7.2.8
bar size=	6	#			Note: Hoops
in-plane spacing=	284.25	in			
Spacing of transverse reinforcement, s=	7.5	in			
bar dia=	0.75	in			
bar area=	0.44	in2			
Number of transverse bars=	2	nos.			
Area of transverse bars per space, Av(bars)/s=	0.117	in2			
tie size=	4	#			Note: Ties
in-plane spacing=	48	in			
Spacing of transverse reinforcement, s=	22.5	in			
bar dia=	0.5	in			
bar area=	0.2	in2			
Number of ties=	7	nos.			Note:As built is 6-7 staggered ties
Area of transverse ties per space, Av(ties)/s=	0.0622	in2			
concrete density modification factor, λ=	1				5.4.2.8
yield strength of transverse reinforcement, fyt=	60	ksi			
design concrete compressive strength, f'c=	4	ksi			
Minimum area of tie steel per space, Av(min)/s=	0.30	in2			5.7.2.5-1
Area of transverse steel, Av/s=	0.180	in2		Minimum transverse reinforcement is not met	
Avg spacing, s=	16.7	in			
Maximum Spacing of Transverse Reinforcement					
resistance factor, φ(shear)=	0.9				5.5.4.2
steel yield, fy=	60	ksi			
design concrete compressive strength, f'c=	4	ksi			
Thickness of wall, h=	30	in			
adjusted width, bv=	288	in			5.7.2.8
effective depth, de=	27.1	in			5.7.2.8-2
Area of tensile steel, As=	47.0	in2			
Nominal flexural resistance, Mn=	5693	k-ft			
effective shear depth, dv=	24.4	in			C5.7.2.8-1
factored shear load, Vu=	55	kips			
Factored shear stress, vu=	0.0087	ksi			5.7.2.8
0.125*f'c=	0.5	ksi			
Maximum spacing, smax=	19.5	in		Maximum transverse reinforcement spacing is met	5.7.2.6-1 & 5.7.2.6-2

Shear Design: Strength, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	288	in			5.7.2.8
effective shear depth, dv=	24.4	in			C5.7.2.8-1
Max nominal shear capacity, Vn2=	7028	kips			5.7.3.3-2
crack spacing parameter, sx=	24.4	in			5.7.3.4.2-3
aggregate size, ag=	0.75	in			As Built: aggregate size assumption
crack spacing parameter influenced by aggregate size, sxe=	24.4	in			5.7.3.4.2-7
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	47.0	in2			
Factored Moment Load, Mn=	1766	k-ft			
Factored Axial Load, Nu=	1098	kips			
Factored Shear Load, Vu=	56	kips	Demand		
longitudinal tensile strain, εs=	0.001081				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	32.8	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	2.13	for sections less than minimum shear reinforcement			5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	947	kips			5.7.3.3-3
Area of transverse steel, Av=	3.0	in2			
angle of inclination of transverse reinforcement to longitudinal axis, α=	90	deg.			
shear strength reduction factor accounting for the presence of PT ducts, λduct=	1				
spacing of transverse reinforcement, s=	16.7	in			
Nominal shear resistance of the steel, Vs=	408	kips			C5.7.3.3-1
Nominal shear resistance, Vn=	1355	kips			5.7.3.3-1
factored shear resistance, φVn=	1220	kips	Capacity	Demand<Capacity OK	5.7.2.1-1

Longitudinal Reinforcement Requirements: Strength, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Area of tensile steel, As=	47.0	in2			
steel yield, fy=	60	ksi			
As*fy=	2819	kips			
effective shear depth, dv=	24.4	in			
Factored Moment Load, Mu=	1766	k-ft			5.5.4.2
resistance factor, φ(flexure)=	0.9				
Factored Axial Load, Nu=	1098	kips			
resistance factor, φ(compression)=	0.9				5.5.4.2
Factored Shear Load, Vu=	56	kips			
resistance factor, φ(shear)=	0.9				5.5.4.2
Nominal shear resistance of the steel, Vs=	408	kips			
As*fy>=	1354	kpis	Longitudinal reinforcement is satisfied		5.7.3.5-1

Interface Shear Transfer Requirements: Strength, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Area of Shear Reinforcement					
Area of tensile steel, As=	47.0	in2			
Area of compression steel, A's=	47.0	in2			
area of interface shear reinforcement, Avf=	94.0	in2			
Design yield stress, fy=	60.0	ksi			
Thickness of wall, h=	30	in			
adjusted width, bv=	288	in			
area of concrete engaged in interface, Acv=	8640	in2			5.7.4.3-6
Avf>=	7.2	in2	Interface area of reinforcement is satisfied		5.7.4.2-1

Interface Shear Transfer Design: Strength, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
cohesion factor, c=	0.24	ksi			5.7.4.4
frition factor, μ=	1				5.7.4.4
friction of concrete strength available, K1=	0.25				5.7.4.4
limiting interface shear resistance, K2=	1.5	ksi			5.7.4.4
Permanent net compressive force, Pc=	0	kips			Note: Conservative
nominal shear resistance, Vni=	7712	kips			5.7.4.3-3
factored shear resistance, φVni=	6941	kips			5.7.4.3-1
Factored shear load, Vu=	56	kips	Demand<Capacity OK		5.7.4.3-2

Service Limit State

Demand:

	P (kips)		V2 (kips)		T (kip-ft)		M3 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min
Service	-1653	-1173	46	-24	-1	-39	1319	-617

Control of cracking: Service Transverse

References:

AASHTO LFRD 9TH EDITION

User Input=				
yield stress of steel, fy=	60	ksi		
Cover=	1	in		
longitudinal bar dia=	1.27	in		
direction 3 bar dia=	0.75	in		
transverse reinforcement bar dia=	0.5	in		
distance of extreme tension fiber to center of flexural reinforcement, dc=	2.89	in		
Thickness of wall, h=	30	in		
ratio of flexural strain, βs=	1.15			5.6.7-2
tensile stress in nonprestressed reinforcement, fss=	36	ksi		
exporsure factor, γe=	1			Note: Class 1 exposure
smax<=	11.11			5.6.7-1
spacing of nonprestressed reinforcement in layer closest to the tension face, s=	8		Spacing is satisfied	

Extreme Limit State

Demand:

	P (kips)		V2 (kips)		T (kip-ft)		M3 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min
100%L+30%T EXT I EtoW	-2262	-824	250	221	20	-49	5580	4638
30%L+100%T EXT I EtoW	-2225	-707	99	70	70	-100	2290	1348
100%L+30%T EXT I WtoE	-2709	-203	175	146	-3	-27	2212	1270
30%L+100%T EXT I WtoE	-2363	-516	109	80	-2	-29	1592	650

Flexural Design: Extreme, Longitudinal Bending

References:

AASHTO LFRD 9TH EDITION

User Input=					
direction 2 reinforcement bar size=	6	#			
bar dia=	0.75	in			Note: Transverse bar
resistance factor, φ(flexure)=	1				5.5.4.2
steel yield, fy=	68	ksi			5.6.2.1
design concrete compressive strength, f'c=	5.2	ksi			
Cover=	1	in			As built
bar size=	10	#			As Built
spacing=	8	in			As Built
bar dia=	1.27	in			
bar area=	1.27	in2			
Thickness of wall, h=	30	in			
Width of Wall, b=	288	in			
Number of reinforcement bars=	37	nos.			Note: Conservative did not account for skin reinforceme
Area of tensile steel, As=	47.0	in2			
distance from extreme comp. fiber to o.c. of tensile steel, ds=	27.1	in			
Cover to longitudinal=	2.9	in			
Skin Reinforcement area per layer=	2.5	in2			Note: Skin reinforcement is neccesary for strength
Number of spaces=	3.0	#			
O.C. Spacing=	8.1	in			
number of layer excluding outer most (already accounted for)=	1.0	#			
Tensile skin reinforcement area, As(skin)=	2.5	in			
Tensile centroid distance from extreme comp. fiber, ds(tensile centroid of skin)=	19.0	in			
Comp. skin reinforcement area, A's(skin)=	2.5	in2			
Comp. centroid distance from extreme comp. fiber, d's(comp. centroid of skin)=	11.0	in			
Area of compression steel, A's=	47.0	in2			Note: Per face, symmetric
distance from extreme comp. fiber to o.c. of tensile steel, d's=	2.89	in			
stress block factor, α1=	0.85				5.6.2.2
stress block factor, β1=	3.94				5.6.2.2
distance from extreme fiber to NA, c=	0.000	in			5.6.3.1.2-4
depth of equivalent stress block, a=	0.000	in			
Nominal flexural resistance, Mn=	6568	kip-ft			5.6.3.2.2-1
Factored flexural resistance, φMn=	6568	kip-ft	Capacity	Demand<Capacity OK	5.6.3.2.1-1
Factored Moment Load, Mu=	5580	kip-ft	Demand		5.6.3.2.1-1

Minimum Reinforcement

correction factor for source of aggregate, K1=	1				5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.1452	kcf			Table 3.5.1-1
concrete density modification factor,λ=	1				C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.547	ksi			5.4.2.6
flexural cracking vairability facgtor, γ1=	1.6				
ratio of specified yield to ultimate strength, γ3=	0.67				
section modulus, Sc=	43200	in3			
Cracking Moment, Mcr=	2112	kip-ft			5.6.3.3-1
1.33 times factored moment required, 1.33 x Mu=	7422	kip-ft			5.6.3.3-1
Lesser of (Mu, Mcr)=	2112	kip-ft	Demand	Demand<Capacity OK	5.6.3.3-1

Torsion Investigation: Extreme, Longitudinal Bending

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(torsion)=	1				5.5.4.2
design concrete compressive strength, f'c=	5.2	ksi			
Thickness of wall, h=	30	in			
Width of Wall, b=	288	in			
area enclosed by outside perimeter of cross section, Acp=	8008	in2			5.7.2.1-4
length of outside perimet of the cross section, pc=	628	in			5.7.2.1-4
Factored axial force, Nu=	707	kips			
gross area, Ag=	8640	in2			
Factored Axial stress, Nu/Ag=	0.082	ksi			5.7.2.1-6
concrete density modification factor, λ=	1				
K=	0.85				5.7.2.1-6
Torsional cracking moment, Tcr=	2068	kip-ft			5.7.2.1-4
0.25*φ*Tcr=	516.95	kip-ft	Capacity	Torsional investigation is not required	
Factored Torsion Load, Tu=	100.38	kip-ft	Demand		5.7.2.1-3

Transverse Reinforcement Requirements: Extreme, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Transverse Reinforcement					
Thickness of wall, h=	30	in			
adjusted width, bv=	288	in			5.7.2.8
bar size=	6	#			Note: Hoops
in-plane spacing=	284.25	in			
Spacing of transverse reinforcement, s=	7.5	in			
bar dia=	0.75	in			
bar area=	0.44	in2			
tie size=	4	#			Note: Ties
in-plane spacing=	48	in			
Spacing of transverse reinforcement, s=	22.5	in			
bar dia=	0.5	in			
bar area=	0.2	in2			
Number of transverse bars=	2	nos			
Area of transverse bars per space, Av(bars)/s=	0.117	in2			
Number of ties=	6.5	#			Note:As built is 6-7 staggered ties
Area of transverse ties per space, Av(ties)/s=	0.0578	in2			
concrete density modification factor, λ=	1				5.4.2.8
yield strength of transverse reinforcement, fyt=	60	ksi			
design concrete compressive strength, f'c=	5.2	ksi			
Minimum area of tie steel per space, Av(min)/s=	0.35	in2			5.7.2.5-1
Area of transverse steel, Av/s=	0.175	in2		Minimum transverse reinforcement is not met	
Avg spacing, s=	16.4	in			
Maximum Spacing of Transverse Reinforcement					
resistance factor, φ(shear)=	0.9				5.5.4.2
steel yield, fy=	60	ksi			
design concrete compressive strength, f'c=	4	ksi			
Thickness of wall, h=	30	in			
adjusted width, bv=	288	in			5.7.2.8
effective depth, de=	27.1	in			5.7.2.8-2
Area of tensile steel, As=	47.0	in2			
Nominal flexural resistance, Mn=	6568	k-ft			
effective shear depth, dv=	28.0	in			C5.7.2.8-1
factored shear load, Vu=	221	kips			
Factored shear stress, vu=	0.0305	ksi			5.7.2.8
0.125*f'c=	0.5	ksi			
Maximum spacing, smax=	22.4	in		Maximum transverse reinforcement spacing is met	5.7.2.6-1 & 5.7.2.6-2

Shear Design: Extreme, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	288	in			5.7.2.8
effective shear depth, dv=	28.0	in			C5.7.2.8-1
Max nominal shear capacity, Vn2=	8051	kips			5.7.3.3-2
crack spacing parameter, sx=	28.0	in			5.7.3.4.2-3
aggregate size, ag=	0.75	in			As Built: aggregate size assumption
crack spacing parameter influenced by aggregate size, sxe=	28.0	in			5.7.3.4.2-7
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	47.0	in2			
Factored Moment Load, Mn=	5580	k-ft			
Factored Axial Load, Nu=	824	kips			
Factored Shear Load, Vu=	250	kips	Demand		
longitudinal tensile strain, εs=	0.002244				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	36.9	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	1.36	for sections less than minimum shear reinforcement			5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	693	kips			5.7.3.3-3
Area of transverse steel, Av=	0.175	in2			
angle of inclination of transverse reinforcement to longitudinal axis, α=	90	deg.			
shear strength reduction factor accounting for the presence of PT ducts, λduct=	1				
spacing of transverse reinforcement, s=	16.4	in			
Nominal shear resistance of the steel, Vs=	24	kips			C5.7.3.3-1
Nominal shear resistance, Vn=	717	kips			5.7.3.3-1
factored shear resistance, φVn=	646	kips	Capacity	Demand<Capacity OK	5.7.2.1-1

Interface Shear Transfer Requirements: Extreme, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Area of Shear Reinforcement					
Area of tensile steel, As=	47.0	in2			
Area of compression steel, A's=	47.0	in2			
area of interface shear reinforcement, Avf=	94.0	in2			
Design yield stress, fy=	60.0	ksi			
Thickness of wall, h=	30	in			
adjusted width, bv=	288	in			
area of concrete engaged in interface, Acv=	8640	in2			5.7.4.3-6
Avf>=	7.2	in2	Interface area of reinforcement is satisfied		5.7.4.2-1

Interface Shear Transfer Design: Extreme, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
cohesion factor, c=	0.24	ksi			5.7.4.4
frition factor, μ=	1				5.7.4.4
friction of concrete strength available, K1=	0.25				5.7.4.4
limiting interface shear resistance, K2=	1.5	ksi			5.7.4.4
Permanent net compressive force, Pc=	0	kips			Note: Conservative
nominal shear resistance, Vni=	7712	kips			5.7.4.3-3
factored shear resistance, φVni=	6941	kips			5.7.4.3-1
Factored shear load, Vu=	250	kips	Demand<Capacity OK		5.7.4.3-2

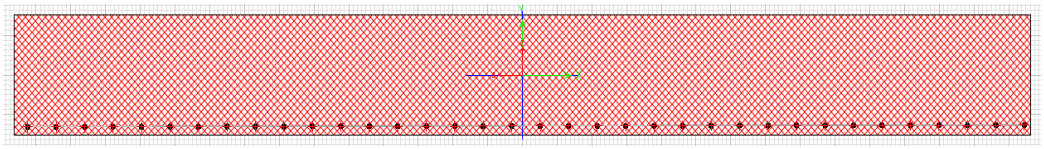
Extreme Limit State for Seismic

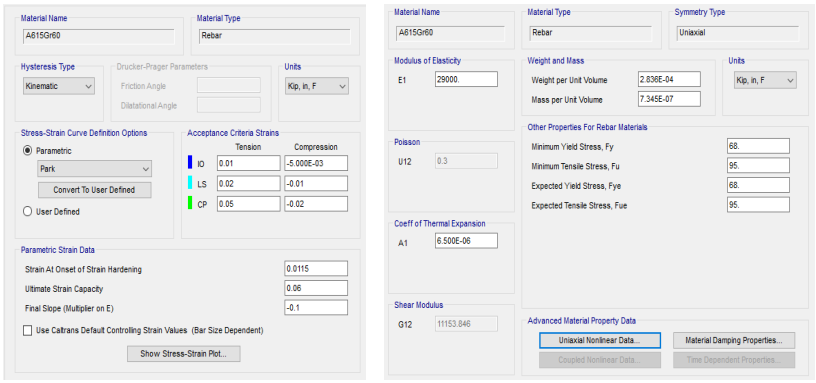
Max Longitudinal Displacement Demand:

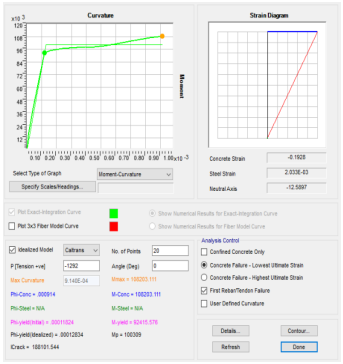
2.65 in

Ductility Demand:

1+2.65/6.36= 1.42







Cross Section Properties [Longitudinal]

References:

AASHTO LFRD Seismic Guide

User Input=

compressive strength of concrete, f'c= 4 ksi Section modelled with unconfined concrete

expected concrete compressive strength, f'ce= 5.2 ksi 8.4.4-1

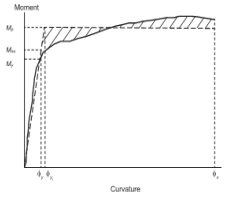
Curvature of initial yield, φy= 0.000118 rad/in from SAP Section Designer

Moment of initial yield, My= 7701 kip-ft from SAP Section Designer

Idealized Ultimate curvature, φu= 0.000914 rad/in from SAP Section Designer

EI of first yeild= 65132764 kip-ft/(rad/in)

Moment - Curvature [Longitudinal]



Idealized plastic moment capacity, Mp= 8359 k-ft from SAP Section Desig Figure 8.5-1

Idealized curvature of initial yield, φyi= 0.000128 rad/in Figure 8.5-1

Idealized Ultimate curvature, φu= 0.000914 rad/in Figure 8.5-1

Idealized curvatureof Plastic Ductility, φpd= 0.000786 C4.9-2

Elastic Displacement [Longitudinal]

Full height of Wall, L= 32.1 ft

386

Idealized curvature of initial yield, φyi= 0.000128 rad/in

elastic displacement, Δy= 6.36 C4.9-3

Plastic Displacement [Longitudinal]

Full height of Wall, L=	385.7	in		
expected yield stress, fye=	68	ksi		Table 8.4.2-1
longitudinal bar size=	10	#		
bar dia, dbl=	1.27	in		
0.08L=	30.9	in		
0.15*fye*dbl=	12.95	in		
plastic hinge length, Lp=	43.81	in		4.11.6-1
Lp(min)=	25.91	in		4.11.6-1
plastic hinge length, Lp=	43.8	in		4.11.6-1
Idealized curvatureof Plastic Ductility, φpd=	0.000786	rad/in		
plastic kinge angle, θp=	0.0344	rad		
L-Lp=	342	in		
plastic displacement, Δp=	11.8	in		

Total Displacement Capacity [Longitudinal]

elastic displacement, Δy=	6.36	in
plastic displacement, Δp=	11.8	in
total displacement, Δ=	18.1	in
Ductility Capacity=	2.85	

Ductility Requirement

ductility demand, μD=	1.42	OK

Plastic Hinge Forces [Longitudinal]

Overstrength Moment, Mpo=	11703	k-ft		
Overstrength Shear Demand, Vu=	364	kips		
Idealized plastic moment capacity, Mp=	8359	k-ft		
overstrength magnifier, λmo=	1.4	A615 Grade 60		
plastic moment capacity of hinge member, Mpo=	11703	k-ft		8.5-1 & 4.11.2-1
Full height of Wall, L=	32.14	ft		
plastic hinge length, Lp=	3.7	ft		
plastic shear capacity of hinge member, Vpo=	364	kips		Fig. 4.11.2-1

Shear Design: Extreme, Longitudinal shear

References:

AASHTO LFRD Seismic Guide

User Input=					
resistance factor, φ(shear)=	0.9				
steel yield, fy=	68	ksi			
design concrete compressive strength, f'c=	4	ksi			
Thickness of wall, h=	30	in			
Width of Wall, b=	288	in			
gross area, Ag=	8640	in2			
bar size=	10	#			Note: Longitudinal Bars
spacing=	8	in			
bar dia=	1.27	in			
bar area=	1.27	in2			
cover=	1	in			
bar size=	6	#			Note: Horizontal Bars
in-plane spacing=	284	in			
Spacing of transverse reinforcement, s=	7.5	in			
bar dia=	0.75	in			
bar area=	0.44	in2			
Number of transverse bars=	2	#			
Area of transverse bars per space, Av(bars)/s=	0.117				
tie size=	4	#			Note: Ties
in-plane spacing=	48	in			
Spacing of transverse reinforcement, s=	22.5	in			
bar dia=	0.5	in			
bar area=	0.2	in2			
Number of ties=	7	#			Note:As built is 6-7 staggered ties
Area of transverse ties per space, Av(ties)/s=	0.062				
Avg spacing, s=	16.7	in			
total area of shear reinforcement, Av=	3.00	in2			
tie reinforcement ratio, ρw=	0.000623				8.6.2-10
fw=	0.0848	ksi	<=	0.35	8.6.2-9
Maximum Local displacement ductility ratio, μ_D=	3				
concrete shear stress adjustment factor, α'=	1.24				8.6.2-8
Factored Axial Load, Pu=	-203	kips			
Nominal Shear Resistance, Vc=	0.0781				8.6.2-3
0.11*sqrt(f'c)=	0.220				
0.047*α'*sqrt(f'c)=	0.116				
Nominal concrete shear strength, vc=	0.08	ksi			
Vc = vc x Ae=	540	kips			
effective depth of flexural tensile reinforcement, d=	27.12	in			
nominal shear reinforcement strength, Vs=	331	kips			8.6.3-2
Vs(max)=	331	kips			
factored shear load, Vu=	364	kips			
nominal shear capacity, Vn=	871	kips			8.6.1-2
factored nominal shear capacity, φVn=	784	kips		Demand<Capacity OK	8.6.1-1

Maximum Shear reinforcement

Effective area, Ae=	6912	in2			
Maximum shear strength allowed, 0.25*sqrt(f'c)*Ag=	3456	kips	OK		8.6.4-1

Minimum Shear Reinforcement

minimum tie reinforcement=	0.004	For SDC C	NG, not enough confinement	8.6.5-4

Ductile Member Design Requirements: Extreme, Longitudinal shear

Dead Load=	1292	kips	DC		
Axial Load from seismic=	0	kips	Pu		
Height of Wall, Hh=	32.1	ft			
Depth of Wall, Ds=	2.5	ft			
fixity factor, Λ=	1		fixed-free		
minimum lateral flexural capacity, Mne=	4314	k-ft		OK, minimum plastic moment is met	8.7.1-1
plastic moment capacity of hinge member, Mpo=	11703	k-ft			

Longitudinal and Lateral Reinforcement Requirements

Maximum Longitudinal Reinforcement

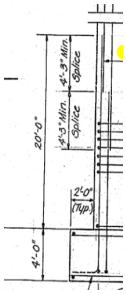
Thickness of wall, h=	30	in			
Width of Wall, b=	288	in			
Area of tensile steel, As=	47.0	in2			Note: Includes Steel Skin
Tensile skin reinforcement area, As(skin)=	2.5	in2			
Area of compression steel, A's=	47.0	in2			
Comp. skin reinforcement area, A's(skin)=	2.5	in2			
area of longitudinal, Al=	99.1	in2			
gross area, Ag=	8640	in2			
maximum area of longitudinal, Al(max)=	346	in2	OK		8.8.1-1

Minimum Longitudinal Reinfocement

minimum area of longitudinal, Al(min)=	21.6	in2	For SDC C Wall	OK	8.8.2-3

Splicing of Longitudinal

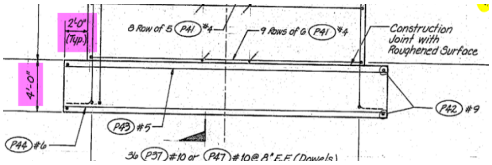
plastic hinge length, Lp=	3.7	ft		OK	8.8.3
Splicing Location=	11.5	ft			

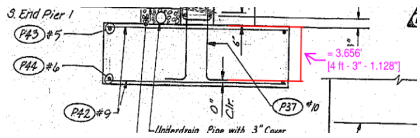


Splicing is outside of plastic hinge region

Minimum Development Length of Reinforcing: **for SDC C**

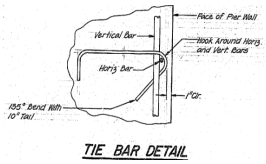
diameter of longitudinal, dbl=	1.3	in			
expected yield stress of longitudinal, fye=	68	ksi			
nominal compressive strength of concrete, f'c=	5.2	ksi			
minimum required anchored length of longitudinal , lac(min)=	29.9	in			8.8.4-1
	2.49	ft			
As-built length of anchors=	3.66	ft	Conservative	OK	





Maximum Bar Diameter: **For SDC C**

full height of wall, L=	385.7	in		
depth of wall, Dc=	30.0	in		
nominal compressive strength of concrete, f'c=	4	ksi		
expected yield stress, fye=	68	ksi		
maximum longitudinal dia, dbl(max)=	8.61	in	OK	8.8.6-1



TIE BAR DETAIL

Lateral Reinforcement Inside Plastic Hinge Region: For SDC C

NG, not enough confinement	see 8.6.5-4 above	8.8.7

Lateral Column Reinforcement outside the Plastic Hinge Region

Wall Reinforcement is Uniform	8.8.8
Lateral Reinforcement is not extended into footing	

Requirements for Lateral Reinforcement

Reinforcement Type=	Ties Hoops			
Avg spacing, s=	16.7	in		
Maximum Plastic Hinge Spacing=	6	in	Not Met	8.8.9

Pier 2 Wall - Design Check about wall strong axis

PROJECT:	BR 17.7 - PIER 2 (Median Pier)
SUBJECT:	Wall Design: About Strong Axis
BY:	TOTG
CHK:	RSGR

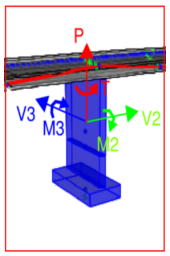
Purpose:
Design check of the as-built wall for strength, service and extreme limit states under transverse loading.

Methodology:
1. Strength and service and extreme case limit states are checked per AASHTO LRFD 9th Edition and per AASHTO LRFD Seismic.

Asumptions:
1. Critical section of wall occurs at the base of the wall.

Dimensions of Wall:

Longitudinal Thickness=	24	ft	Direction of 2
Transverse Thickness=	2.5	ft	Direction of 3



PIER 2

Strength Limit State

Demand:

	P (kips)		**V3 (kips)**		T (kip-ft)		**M2 (kip-ft)**	
	Max	Min	Max	Min	Max	Min	Max	Min
Strength I	-2132	-1148	-112	15	2	-41	1683	-1027
Strength III	-1640	-1098	-47	2	-26	-26	1129	-95
Strength V	-2058	-1098	-107	21	-48	6	1778	-841
Service I	-1653	-1173	-84	7	-1	-39	1659	-793

References:

User Input=			**AASHTO LFRD 9TH EDITION**
direction 3 reinforcement bar size=	6		Note: Hoops
bar dia=	0.75		
resistance factor, φ(flexure)=	0.9		5.5.4.2
nominal flexural resistance, fy=	60	ksi	5.6.2.1
design concrete compressive strength, f'c=	4	ksi	
Cover=	1	in	
bar size=	10	#	
spacing=	8	in	
bar dia=	1.27	in	
bar area=	1.27	in2	
Thickness of wall, h=	288	in	
Width of Wall, b=	30	in	
correction factor for source of aggregate, K1=	1		5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.145	kcf	Table 3.5.1-1
concrete density modification factor,λ=	1		C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.480	ksi	5.4.2.6

Torsion Investigation: Strength, Transverse Bending

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(torsion)=	0.9				5.5.4.2
design concrete compressive strength, f'c=	4	ksi			
Thickness of wall, h=	288	in			
Width of Wall, b=	30	in			
area enclosed by outside perimeter of cross section, Acp=	8008	in2			5.7.2.1-4
length of outside perimet of the cross section, pc=	628	in			5.7.2.1-4
Factored axial force, Nu=	1148	kips			
gross area, Ag=	8640	in2			
Factored Axial stress, Nu/Ag=	0.133	ksi			5.7.2.1-6
concrete density modification factor, λ=	1				
K=	0.69				5.7.2.1-6
Torsional cracking moment, Tcr=	1474	kip-ft			5.7.2.1-4
0.25*φ*Tcr=	332	kip-ft	Capacity	Torsional investigation is not required	
Factored Torsion Load, Tu=	41	kip-ft	Demand		5.7.2.1-3

Transverse Reinforcement Requirements: Strength, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Transverse Reinforcement					
Thickness of wall, h=	288	in			
adjusted width, bv=	30	in			5.7.2.8
bar size=	6	#			Note: Hoops
in-plane spacing=	26.25	in			
Spacing of transverse reinforcement, s=	7.5	in			
bar dia=	0.75	in			
bar area=	0.44	in2			
Number of transverse bars=	2	#			
Area of transverse bars per space, Av(bars)/s=	0.117	in2/in			Note: Ties
tie size=	4	#			
in-plane spacing=	48	in			
Spacing of transverse reinforcement, s=	22.5	in			
bar dia=	0.5	in			
bar area=	0.2	in2			
Number of ties=	1	#			
Area of transverse ties per space, Av(ties)/s=	0.0089	in2/in			
concrete density modification factor, λ=	1				5.4.2.8
yield strength of transverse reinforcement, fyt=	60	ksi			
design concrete compressive strength, f'c=	4	ksi			
Minimum area of tie steel per space, Av(min)/s=	0.03	in2			5.7.2.5-1
Area of transverse steel, Av/s=	0.126	in2		Minimum Transverse Reinforcement is met	
Avg spacing, s=	10.3				
resistance factor, φ(shear)=	0.9				5.5.4.2
nominal flexural resistance, fy=	60	ksi			
design concrete compressive strength, f'c=	4	ksi			
Thickness of wall, h=	288	in			
adjusted width, bv=	30	in			5.7.2.8

Shear Design: Strength, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	30	in			5.7.2.8
effective shear depth, dv=	211.7	in			C5.7.2.8-1
Max nominal shear capacity, Vn2=	6350	kips			5.7.3.3-2
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	15.2	in2	12 #10 bars		
Factored Moment Load, Mn=	1778	k-ft			
Factored Axial Load, Nu=	1148	kips			
Factored Shear Load, Vu=	112	kips	Demand		
longitudinal tensile strain, εs=	0.001781				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	35.2	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	2.06		for sections at least minimum shear reinforcement		5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	825	kips			5.7.3.3-3
Area of transverse steel, Av=	1.297	in2			
angle of inclination of transverse reinforcement to longitudinal axis, α=	90	deg.			
shear strength reduction factor accounting for the presence of PT ducts, λduct=	1				
spacing of transverse reinforcement, s=	10.3	in			
Nominal shear resistance of the steel, Vs=	2270	kips			C5.7.3.3-1
Nominal shear resistance, Vn=	3095	kips			5.7.3.3-1
factored shear resistance, φVn=	2785	kips	Capacity	Demand<Capacity OK	5.7.2.1-1

Interface Shear Transfer Requirements: Strength, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Area of Shear Reinforcement					
Area of tensile steel, As=	47.0	in2	37 #10		
Area of compression steel, A's=	47.0	in2	37 #10		
area of interface shear reinforcement, Avf=	94.0	in2			
Design yield stress, fy=	60.0	ksi			
Thickness of wall, h=	288	in			
adjusted width, bv=	30	in			
area of concrete engaged in interface, Acv=	8640	in2			5.7.4.3-6
Avf>=	7.2	in2		Interface area of reinforcement is satisfied	5.7.4.2-1

Interface Shear Transfer Design: Strength, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
cohesion factor, c=	0.24	ksi			5.7.4.4
frition factor, μ=	1				5.7.4.4
friction of concrete strength available, K1=	0.25				5.7.4.4
limiting interface shear resistance, K2=	1.5	ksi			5.7.4.4
Permanent net compressive force, Pc=	0	kips			Note: Conservative
nominal shear resistance, Vni=	7712	kips			5.7.4.3-3
factored shear resistance, φVni=	6941	kips			5.7.4.3-1
Factored shear load, Vu=	112	kips		Demand<Capacity OK	5.7.4.3-2

Service Limit State

Demand:

	P (kips)		V3 (kips)		T (kip-ft)		M2 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min
Service	-1653	-1173	-84	7	-1	-39	1659	-793

Control of cracking: Service Transverse

References:

AASHTO LFRD 9TH EDITION

User Input=			
yield stress of steel, fy=	60	ksi	
Cover=	1	in	
longitudinal bar dia=	1.27	in	
direction 3 bar dia=	0.75	in	
transverse reinforcement bar dia=	0.5	in	
distance of extreme tension fiber to center of flexural reinforcement, dc=	2.885	in	
Thickness of wall, h=	288	in	
ratio of flexural strain, βs=	1.01		5.6.7-2
tensile stress in nonprestressed reinforcement, fss=	36	ksi	
exporsure factor, γe=	1		Note: Class 1 exposure
smax<=	13.40		5.6.7-1
spacing of nonprestressed reinforcement in layer closest to the tension face, s=	8		Spacing is satisfied

Extreme Limit State

Demand:

	P (kips)		V3 (kips)		T (kip-ft)		M2 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min
100%L+30%T EXT I EtoW	-2262	-824	239	-111	20	-49	3399	-7198
30%L+100%T EXT I EtoW	-2225	-707	502	-387	70	-100	10523	-12381
100%L+30%T EXT I WtoE	-2709	-203	259	-189	-3	-27	4594	-6290
30%L+100%T EXT I WtoE	-2363	-516	368	-294	-2	-29	6729	-7956

References:

AASHTO LFRD 9TH EDITION

User Input=			
direction 2 reinforcement bar size=	6		
bar dia=	0.75		Note: Transverse bar
resistance factor, φ(flexure)=	1		5.5.4.2
steel yield, fy=	68	ksi	5.6.2.1
design concrete compressive strength, f'c=	5.2	ksi	
Cover=	1	in	As built
bar size=	10	#	As Built
spacing=	8	in	As Built
bar dia=	1.27	in	
bar area=	1.27	in2	
Thickness of wall, h=	288	in	
Width of Wall, b=	30	in	

Transverse Reinforcement Requirements: Extreme, Transverse Shear

References:

User Input=

AASHTO LFRD 9TH EDITION

Minumum Transverse Reinforcement

Thickness of wall, h=	288	in		
adjusted width, bv=	30	in		5.7.2.8
bar size=	6	#		Note: Hoops
in-plane spacing=	26.25	in		
Spacing of transverse reinforcement, s=	7.5	in		
bar dia=	0.75	in		
bar area=	0.44	in2		
tie size=	4	#		Note: Ties
in-plane spacing=	48	in		
Spacing of transverse reinforcement, s=	22.5	in		
bar dia=	0.5	in		
bar area=	0.2	in2		
Number of transverse bars=	2	#		
Area of transverse bars per space, Av(bars)/s=	0.117			
Number of ties=	6.5	#		Note:As built is 6-7 staggered ties
Area of transverse ties per space, Av(ties)/s=	0.0578			
concrete density modification factor, λ=	1			5.4.2.8
yield strength of transverse reinforcement, fyt=	60	ksi		
design concrete compressive strength, f'c=	5.2	ksi		
Minimum area of tie steel per space, Av(min)/s=	0.04	in2		5.7.2.5-1
Area of transverse steel, Av/s=	0.175	in2	Minimum Transverse Reinforcement is met	
Avg spacing, s=	16.4			

Shear Design: Extreme, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	30	in			5.7.2.8
effective shear depth, dv=	212	in			C5.7.2.8-1
Max nominal shear capacity, Vn2=	6350	kips			5.7.3.3-2
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	15.2	in2			
Factored Moment Load, Mn=	12381	k-ft			
Factored Axial Load, Nu=	707	kips			
Factored Shear Load, Vu=	502	kips	Demand		
longitudinal tensile strain, εs=	0.003523				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	41.3	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	1.32		for sections at least minimum shear reinforcement		5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	529	kips			5.7.3.3-3
Area of transverse steel, Av=	0.175	in2			
angle of inclination of transverse reinforcement to longitudinal axis, α=	90	deg.			
shear strength reduction factor accounting for the presence of PT ducts, λduct=	1				
spacing of transverse reinforcement, s=	16.4	in			
Nominal shear resistance of the steel, Vs=	154	kips			C5.7.3.3-1
Nominal shear resistance, Vn=	683	kips			5.7.3.3-1
factored shear resistance, φVn=	614	kips	Capacity	Demand<Capacity OK	5.7.2.1-1

Longitudinal Reinforcement Requirements: Extreme, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Area of tensile steel, As=	15.2	in2			
steel yield, fy=	68	ksi			
As*fy=	1036	kips			
effective shear depth, dv=	211.7	in			
Factored Moment Load, Mu=	12381	k-ft			5.5.4.2
resistance factor, φ(flexure)=	1.0				
Factored Axial Load, Nu=	707	kips			
resistance factor, φ(compression)=	0.9				5.5.4.2
Factored Shear Load, Vu=	502	kips			
resistance factor, φ(shear)=	0.9				5.5.4.2
Nominal shear resistance of the steel, Vs=	154	kips			
As*fy>=	1641	kpis	Longitudinal reinforcement is not satisfied		5.7.3.5-1

Interface Shear Transfer Requirements: Extreme, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
Minumum Area of Shear Reinforcement					
Area of tensile steel, As=	47.0	in2			
Area of compression steel, A's=	47.0	in2			
area of interface shear reinforcement, Avf=	94.0	in2			
Design yield stress, fy=	60.0	ksi			
Thickness of wall, h=	288	in			
adjusted width, bv=	30	in			
area of concrete engaged in interface, Acv=	8640	in2			5.7.4.3-6
Avf>=	7.2	in2	Interface area of reinforcement is satisfied		5.7.4.2-1

Interface Shear Transfer Design: Extreme, Transverse Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
cohesion factor, c=	0.24	ksi			5.7.4.4
frition factor, μ=	1				5.7.4.4
friction of concrete strength available, K1=	0.25				5.7.4.4
limiting interface shear resistance, K2=	1.5	ksi			5.7.4.4
Permanent net compressive force, Pc=	0	kips			Note: Conservative
nominal shear resistance, Vni=	7712	kips			5.7.4.3-3
factored shear resistance, φVni=	6941	kips			5.7.4.3-1
Factored shear load, Vu=	502	kips	Demand<Capacity OK		5.7.4.3-2

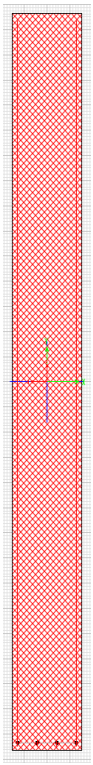
Extreme Limit State for Seismic

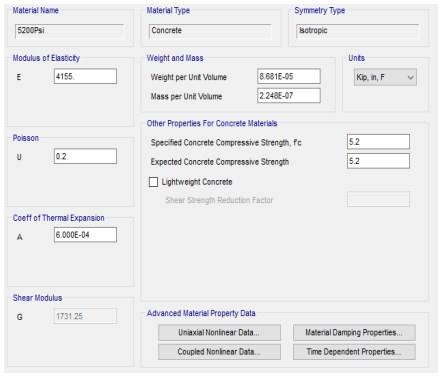
Max Transverse Displacement Demand:

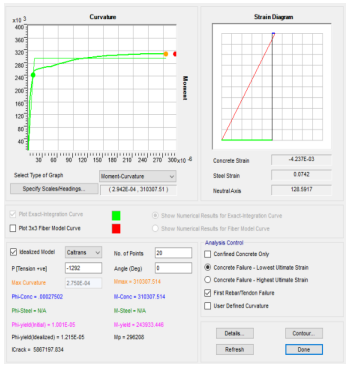
0.6 in

Ductility Demand:

1+0.6/0.6= 2.00







Cross Section Properties [Transverse]

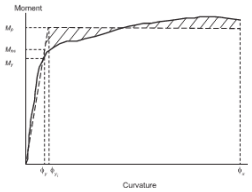
References:

User Input=

AASHTO LFRD Seismic

compressive strength of concrete, f'c=	4	ksi		
expected concrete compressive strength, f'ce=	5.2	ksi		8.4.4-1
Curvature of initial yield, φy=	0.00001001	rad/in	from SAP Section Designer	
Moment of initial yield, My=	20328	kip-ft	from SAP Section Designer	
Idealized Ultimate curvature, φu=	0.000275	rad/in	from SAP Section Designr	
EI of first yeild=	2030747969	kip-ft/(rad/in)		

Moment - Curvature [Transverse]



Idealized plastic moment capacity, Mp=	24684	k-ft	from SAP Section Designr	Figure 8.5-1
Idealized curvature of initial yield, φyi=	0.000012	rad/in		Figure 8.5-1
Idealized Ultimate curvature, φu=	0.000275	rad/in		Figure 8.5-1
Idealized curvatureof Plastic Ductility, φpd=	0.000263			C4.9-2

Elastic Displacement [Transverse]

Full height of Wall, L=	32.1	ft	
	386		
Idealized curvature of initial yield, φyi=	0.000012	rad/in	
elastic displacement, Δy=	0.60		C4.9-3

Plastic Displacement [Transverse]

Full height of Wall, L=	385.7	in	
expected yield stress, fye=	68	ksi	Table 8.4.2-1
longitudinal bar size=	10	#	
bar dia, dbl=	1.27	in	
0.08L=	30.9	in	
0.15*fye*dbl=	12.95	in	
plastic hinge length, Lp=	43.81	in	4.11.6-1
Lp(min)=	25.91	in	4.11.6-1
plastic hinge length, Lp=	43.8	in	4.11.6-1
Idealized curvatureof Plastic Ductility, φpd=	0.000263	rad/in	
plastic kinge angle, θp=	0.0115	rad	
L-Lp=	342	in	
plastic displacement, Δp=	3.9	in	

Total Displacement Capacity [Transverse]

elastic displacement, Δy=	0.60	in
plastic displacement, Δp=	3.9	in
total displacement, Δ=	4.5	in
ductility capacity, μC=	7.5	

Ductility Requirement [Transverse]

ductility demand, μD=	2.00	OK

Plastic Hinge Forces

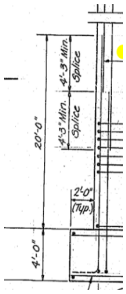
Plastic Moment Demand, Mu=	24684	k-ft	
Plastic Shear Demand, Vu=	768	kips	
Idealized plastic moment capacity, Mp=	24684	k-ft	
overstrength magnifier, λmo=	1.4	A615 Grade 60	
plastic moment capacity of hinge member, Mpo=	34558	k-ft	8.5-1 & 4.11.2-1
Full height of Wall, L=	32.14	ft	
plastic hinge length, Lp=	3.7	ft	
plastic shear capacity of hinge member, Vpo=	1075	kips	Fig. 4.11.2-1

Pier Wall Shear Capacity in Strong Direction

Thickness of wall, h=	288	in	
Width of Wall, b=	30	in	
Avg spacing, s=	10.3	in	
cross-sectional area of shear reinforcement, Av=	10.37	in2	
transverse reinforcement ratio, ρh=	0.0336		
design concrete compressive strength, f'c=	5.2	ksi	
Effective area, Ae=	6912	in2	
0.25*sqrt(f'c)*Ae=	3940	kips	
yield stress of tie reinforcement, fyh=	68	ksi	
unreduced elastic demand, Vn=	22319	kips	
φVn=	20087	kips	Demand<Capacity OK

Splicing of Longitudinal

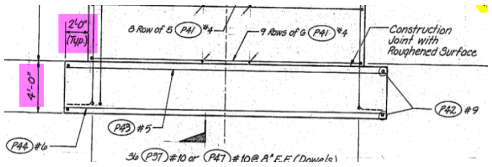
plastic hinge length, Lp=	3.7	ft	OK	8.8.3
Splicing Location=	11.5	ft		

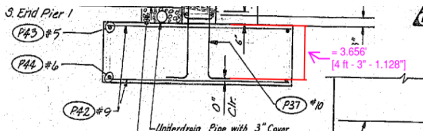


Splicing is outside of plastic hinge region

Minimum Development Length of Reinforcing: **for SDC C**

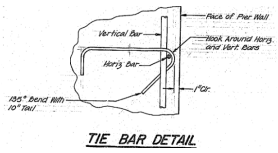
diameter of longitudinal, dbl=	1.27	in		
expected yield stress of longitudinal, fye=	68	ksi		
nominal compressive strength of concrete, f'c=	5.2	ksi		
minimum required anchored length of longitudinal , lac(min)=	29.9	in		8.8.4-1
	2.49	ft		
As-built length of anchors=	3.66	ft	Conservative	OK





Maximum Bar Diameter: **For SDC C**

full height of wall, L=	385.7	in		
depth of wall, Dc=	288.0	in		
nominal compressive strength of concrete, f'c=	5.2	ksi		
expected yield stress, fye=	68	ksi		
maximum longitudinal dia, dbl(max)=	6.40	in	OK	8.8.6-1



TIE BAR DETAIL

Lateral Reinforcement Inside Plastic Hinge Region: For SDC C

OK see 8.6.5-4 above 8.8.7

Lateral Column Reinforcement outside the Plastic Hinge Region

Wall Reinforcement is Uniform 8.8.8

Lateral Reinforcement is not extended into footing

Requirements for Lateral Reinforcement

Reinforcement Type=	Ties Hoops			
Avg spacing, s=	10.3	in		
Maximum Plastic Hinge Spacing=	57.6	in	OK	8.8.9

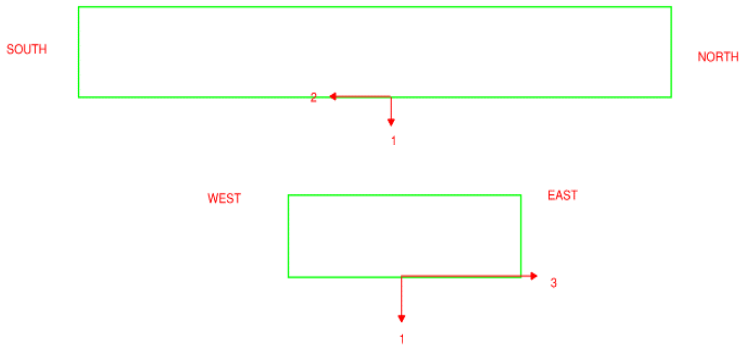
Pier 2 Footing - Design loads

PROJECT: BR 17.7 - PIER 2 (Median Pier)
SUBJECT: Footing Design: Factored Limit States Combinations
BY: TOTG
CHK: RSGR

Purpose:
Tabulate factored loads from unfactor loads determined by the Extreme, Strength and Service limit states.

Method:
1. Limit states per AASHTO LFRD 8th Edition

*Note: Directional notation is different as noted for Wall



Demands - At bottom of footing

Load Cases		P (kips)	V2 (kips)	V3 (kips)	T (kip-ft)	M3 (kip-ft)	M2 (kip-ft)	
DC		-1535	3	9	-3	-126	97	
LL	Min	0	20	-44	18	305	735	
	Max	-310	-22	14	-10	-641	-796	
PS		31	3	-8	-1	-12	120	
WS - STR III	Min	50	9	-12	4	240	175	
	Max	-50	-9	12	-4	-240	-175	
WS - STR V	Min	1	6	-4	2	107	127	
	Max	-1	-6	4	-2	-107	-127	
WS - SER I	Min	1	5	-3	1	82	99	
	Max	-1	-5	3	-1	-82	-99	
WL	Min	0	0	-8	4	185	10	
	Max	0	0	8	-4	-185	-10	
TG	Min	101	-2	-14	2	322	-60	
	Max	-30	0	4	-1	-97	16	
TU	Min	4	-11	-13	0	282	-414	
	Max	-5	5	22	-1	-483	171	
EH - At Rest		80	4	-30	-7	814	148	
SE - Pier 1		-26	1	0	0	-25	34	
CR + SH		-70	3	6	-13	-209	98	
LEP_Pier3		-109	42	44	0	-1194	1540	
LS_Pier3		-3	1	1	0	-31	40	
100%L+30%T+30%V	Max	-533	154	180	-23	-4057	3516	EQ East to West
100%L-30%T-30%V	Min	533	154	-180	23	4058	3516	
30%L+100%T+30%V	Max	-566	53	425	-65	-9231	1180	
30%L-100%T-30%V	Min	566	53	-425	65	9231	1180	
LEP_Pier1		-23	-2	10	0	-281	-90	
LS_Pier1		-1	0	1	0	-20	-5	
100%L+30%T+30%V	Max	-978	129	224	-4	-4024	1834	EQ West to East
100%L-30%T-30%V	Min	978	129	-224	4	4024	1834	
30%L+100%T+30%V	Max	-703	73	322	-5	-5534	1044	
30%L-100%T-30%V	Min	703	73	-322	5	5534	1044	

Load Combos:

Load Factors (AASHTO Table 3.4.1-1)

	DC		PS		EH		CR + SH		LL	WS	TU	TG	SE	EQ	
	Max	Min	Max	Min	Max	Min	Max	Min						Strong-M3	Weak-M2
Strength I	1.25	0.9	1.25	0.9	1.25	0.9	1.25	0.9	1.75	0	0.5	0	1	0	0
Strength III	1.25	0.9	1.25	0.9	1.25	0.9	1.25	0.9	0	1	0.5	0	1	0	0
Strength V	1.25	0.9	1.25	0.9	1.25	0.9	1.25	0.9	1.35	1	0.5	0	1	0	0
Extreme I	1	1	1	1	1	1	1	1	0.5	0	0.5	0	1	1	1.2
Service I	1	1	1	1	1	1	1	1	1	1	0.5	0.5	1	0	0

Factored Loads (absolute values):

	P (kips)		V3		V2		T (kip-ft)		M3		M2	
	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min
Strength I	2439	1369	55	31	112	15	2	41	1234	968	1985	1149
Strength III	1947	1319	21	2	47	2	26	26	940	86	873	69
Strength V	2365	1319	56	32	107	21	48	6	1352	788	1866	1006
EXT I EtoW	2508	1069	250	221	239	111	20	49	2770	6201	6581	5523
EXT I EtoW	2470	952	99	70	502	387	70	100	8790	10528	2686	1628
EXT I WtoE	2955	449	175	146	259	189	3	27	3654	5249	2911	1853
EXT I WtoE	2609	762	109	80	368	294	2	29	5368	6555	2027	969
Service I	1898	1418	46	24	84	7	1	39	1289	729	1501	711

Uni-Axial Factored Envelope:

	P (kips)		V3		V2		T (kip-ft)		M3		M2	
	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min
Strength	2439	1319	56	32	112	21	48	41	1352	968	1985	1149
EXT I EtoW	2508	1069	250	221	502	387	70	100	8790	10528	6581	5523
EXT I WtoE	2955	762	175	146	368	294	3	29	5368	6555	2911	1853
Service	1898	1418	46	24	84	7	1	39	1289	729	1501	711

Bi-Axial Envelopes:

sum of square roots

	P (kips)		V2 & V3 (kips)		M2 & M3 (kips)	
	Max	Min	Max	Min	Max	Min
Strength	2439	1319	125	3	2337	110
EXT I EtoW	2508	1069	561	446	10980	11889
EXT I WtoE	2955	762	408	328	6106	6812
Service	1898	1418	95	25	1978	1018

Pier 2 Footing - Design Check
(Bending about wall weak axis)

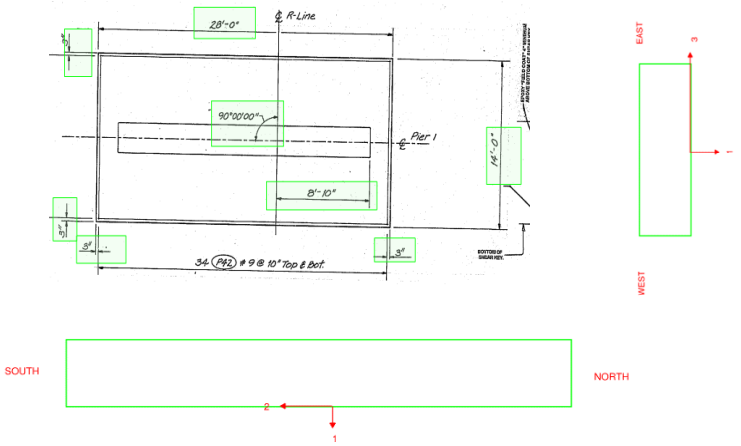
PROJECT:	BR 17.7 - PIER 2 (Median Pier)
SUBJECT:	Footing Design: Bending About weak axis
BY:	TOTG
CHK:	RSGR

Purpose:
Design check of the as-built footing for strength, service and extreme limit states for wall bending

Methodology:
1. Geotechnical design of the footing is check per WSHDOT BDM and AASHTO LRFD Seismic.
2. Concrete Design is check for strength, service and extreme case limit states are per AASHTO LRFD 9th Edition

Asumptions:

Dimensions of Footing:



Bending in direction of Centerline of R-line

longitudinal footing thickness=	28	ft	Direction of 2
longitudinal wall thickness=	24	ft	Direction of 2
transverse footing thickness=	14	ft	Direction of 3
transverse wall thickness=	2.5	ft	Direction of 3
footing thickness=	4	ft	

Factored Demands [Longitudinal]

Demand:

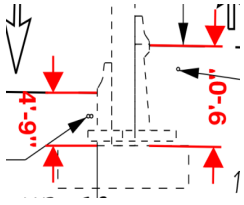
	P (kips)		**V3 (kips)**		T (kip-ft)		**M2 (kip-ft)**	
	Max	Min	Max	Min	Max	Min	Max	Min
Strength	2439	1319	56	32	48	41	1985	1149
EXT I EtoW	2508	1069	250	221	70	100	6581	5523
EXT I WtoE	2955	762	175	146	3	29	2911	1853
Service	1898	1418	46	24	1	39	1501	711

Undactored Vertcal DC Load superstructure + substructure= 1535 kips

Additional EV from soil over footing

User Input=		
unit weight of soil, γEV=	130	pcf
length of footing, B=	14	ft
soil top surface of footing width, Bev=	11.5	ft
parellel to wall dimension, L=	28	ft
Centerline of footing, C=	0	ft
Toe earth depth, Hev(toe)=	57	in
equivalent point load of earth above toe, Pev(toe)=	99.4	kips
distance from middle of wall, Xev(toe)=	-7	ft
Heel earth depth, Hev(heel)=	108	in
equivalent point load of soil above heel, Pev(heel)=	188.4	kips
distance from middle of wall, Xev(heel)=	7	ft

Center of Wall & Footing



References:

WSHDOT BDM
AASHTO LRFD
AASHTO Seismic

Fig 11.6.3.2-1 (LRFD)
Fig. 7.7.4-2 (BDM)

Combine Demand for increase envelope:

	P (kips)		**V3 (kips)**		T (kip-ft)		**M3 (kip-ft)**	
	Max	Min	Max	Min	Max	Min	Max	Min
Strength	2727	1607	56	32	48	41	1985	1149
EXT I EtoW	2796	1357	250	221	70	100	6581	5523
EXT I WtoE	3243	1050	175	146	3	29	2911	1853
Service	2186	1706	46	24	1	39	1501	711

*Additional EV from longitudinal soil load on other direction of footing surface

Total unfactored Vertical Force including soil above footing= 1823 kips

Bearing Stress [Longitudinal]

Bearing Stress [Strength] - worst case

Factored Strength Bearing Resistance, qmax=	47.9	ksf			Geotech Report
bearing resistance factor, φb=	0.45				Table 10.5.5.2.2-1 (LRFD)
resultant force, R=	2727	kips			
resultant moment, M=	1985	k-ft			
eccentricity, e=	0.73		e<=Lr/6	OK for strength limits	10.6.3.3 (LRFD)
maximum soil stress, σmax=	255.5	k-ft		Trapezoidal	7.7.4 B (BDM)
minimum soil stress, σmin=	134.0	k-ft			
bearing pressure, q=	9.13	ksf			
C/D=	**5**			Demand<Capacity OK	

Bearing Stress [Extreme] - worst case

Factored Extreme Bearing Resistance, qmax=	95.9	ksf			
bearing resistance factor, φb=	0.45				Geotech Report Table 10.5.5.2.2-1 (LRFD)
resultant force, R=	1357	kips			
resultant moment, M=	6581	k-ft			
eccentricity, e=	4.85	ft	e>Lr/6		
maximum soil stress, σ=	420.7	k-ft		Triangular	7.7.4 B (BDM)
minimum soil stress, σmin=	0.0	k-ft			
bearing pressure, q=	15.03	ksf			
C/D=	**6**			Demand<Capacity OK	

Sliding [Longitudinal]

Failure by Sliding [Strength]

Max STR factored shear demand, Vu=	32	kips			
coefficient of friction, C=	0.8		for precast concrete		10.6.3.4-2 (LRFD)
angle of friction, θf=	38	deg.			
Total unfactored Vertical Force including soil above footing=	1823	kips			
nominal sliding resistance, Rt=	1140	kips			
bearing resistance factor, φt=	0.8	0.8 factor for STR			Table 10.5.5.2.2-1 (AASHTO)
nominal sliding resistance, φt*Rn=	912	kips			
C/D=	**29**			Demand<Capacity OK	

Failure by Sliding [Extreme]

Max EXT shear demand from wall, Vu=	250	kips			
factored shear demand, Vu=	250	kips			
coefficient of friction, C=	0.8		for precast concrete		10.6.3.4-2 (LRFD)
angle of friction, θf=	38	deg.			
total vertical force, V=	1823	kips			
nominal sliding resistance, Rt=	1140	kips			10.6.3.4-2 (LRFD)
bearing resistance factor, φt=	0.9	0.9 factor for EXT			Table 10.5.5.2.2-1 (LRFD)
nominal sliding resistance, φt*Rn=	1026	kips			6.3.5-1 (Seismic)
C/D=	**4**			Demand<Capacity OK	

Overturning Stability [Longitudinal]

Overturning Stability [Strength]

Lp=	14	ft		
MAX STR eccentricity, e=	0.73	ft		
Lp/6=	2.33	ft	Overturning Stability is OK	10.6.3.3 (LRFD)

Overturning Stability [Extreme]

middle 2/3 of Lp=	4.67	for γeq= 0		11.6.5 (LRFD)
middle 8/10 of Lp=	5.60	for γeq= 0.5		11.6.5 (LRFD)
for γeq= 0.5: resultant force, R=	1357			Note: Conservative does not include EV
for γeq= 0.5: resultant moment demand, Mu=	6581			
for γeq= 0.5: eccentricity, e=	4.85		OK<5.60	11.6.5 (LRFD)
factored axial force, Pu=	2796	kips		
length of footing, L=	28	ft		
width of footing, B=	14	ft		
Factored Extreme Bearing Resistance, qmax=	95.9	ksf		
resistance factor for overturning, φ=	1			
a=	1.04	ft		6.3.4-2 (Seismic)
Max Resistance Capacity Moment, Mr=φ*Pu*(B-a)/2=	18114	k-ft	Capacity Mr>Demand Mu of wall	6.3.4-1 (Seismic)

Footing Reinforcement Design: Strength [Longitudinal]

Critical Section Forces: Strength, Longitudinal Bending

resultant force, R=	2727	kips			
resultant moment, M=	1985	k-ft			
eccentricity, e=	0.73				
width of wall=	2.5	ft			
width of footing=	14.0	ft			
parallel dimension, L=	28.0	ft			
height of footing, h=	4.0	ft			
maximum soil stress, σmax=	255.5	k-ft			
minimum soil stress, σmin=	134.0	k-ft			

Flexural Critical Cross Section

location of critical bending plane from far edge=	8.3	ft			
location of critical bending plane from close edge=	5.8	ft			
soil stress at bending plane, σf=	205.6	k-ft	Trapezoidal		10.6.5 (LRFD)
Factored moment demand at critical section, Mu=	3949	k-ft	Centroid*Area		

Shear Critical Cross Section

dv/2=	20.0	in			
location of critical bending plane from far edge=	9.9	ft			
location of critical bending plane from close edge=	4.1	ft			
soil stress at bending plane, σf=	220.1	k-ft	Trapezoidal		10.6.5 (LRFD)
Factored shear demand at critical section, Vu=	971	kips	Area		
Factored moment demand at critical section, Mu=	2032	k-ft	Centroid*Area		

Flexural Design: Strength, Longitudinal Bending

References:

AASHTO LFRD 9TH EDITION

User Input=					
direction 2 reinforcement bar size=	5	#			Note: horizontal top bars
bar dia=	0.625	in			
bar area=	0.31	in2			
direction 2 reinforcement bar size=	6	#			Note: horizontal bottom bars
bar dia=	0.75	in			
bar area=	0.44	in2			
resistance factor, φ(flexure)=	0.9				5.5.4.2
steel yield, fy=	60	ksi			5.6.2.1
design concrete compressive strength, f'c=	4	ksi			
Cover=	3	in			As built
bar size=	9	#			As Built
spacing=	10	in			As Built
bar dia=	1.128	in			
bar area=	1	in2			
Thickness of footing, h=	48	in			
Width of footing, b=	336.0	in			
Number of reinforcement bars=	34	#			Note:
Area of tensile steel, As=	34.0	in2			
distance from extreme comp. fiber to o.c. of tensile steel, ds=	44.4	in			
Area of compression steel, A's=	34.0	in2			Note: Per face, symmetric
distance from extreme comp. fiber to o.c. of tensile steel, d's=	3.56	in			
stress block factor, α1=	0.85				5.6.2.2
stress block factor, β1=	0.85				5.6.2.2
distance from extreme fiber to NA, c=	0.000	in			5.6.3.1.2-4
depth of equivalent stress block, a=	0.000	in			
Nominal flexural resistance, Mn=	6948	kip-ft			5.6.3.2.2-1
Factored flexural resistance, φMn=	6253	kip-ft	Capacity	Demand<Capacity OK	5.6.3.2.1-1
Factored Moment Load, Mu=	3949	kip-ft	Demand		5.6.3.2.1-1

Minimum Reinforcement

correction factor for source of aggregate, K1=	1				5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.145	kcf			Table 3.5.1-1
concrete density modification factor,λ=	1				C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.480	ksi			5.4.2.6
flexural cracking vairability facgtor, γ1=	1.6				
ratio of specified yield to ultimate strength, γ3=	0.67				
section modulus, Sc=	129024	in3			
Cracking Moment, Mcr=	5533	kip-ft			5.6.3.3-1
1.33 times factored moment required, 1.33 x Mu=	5252	kip-ft			5.6.3.3-1
Lesser of (Mu, Mcr)=	5252	kip-ft	Demand	Demand<Capacity OK	5.6.3.3-1

Shear Design: Strength, Longitudinal Shear

References:

User Input=				**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9			
design concrete compressive strength, f'c=	4	ksi		
yield strength of transverse reinforcement, fyt=	60	ksi		
adjusted width, bv=	336.0	in		5.7.2.8
effective shear depth, dv=	39.99	in		C5.7.2.8-1
Modulus of elasticity, Es=	29000	ksi		
Area of tensile steel, As=	34.0	in2		
Factored Moment Load, Mn=	2032	k-ft		
Factored Axial Load, Nu=	2727	kips		
Factored Shear Load, Vu=	971	kips	Demand	
longitudinal tensile strain, εs=	0.002986			5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	39.5	deg.		5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	1.48		for sections at least minimum shear reinforcement	5.7.3.4.2-2
concrete density modification factor, λ=	1			5.4.2.8
Nominal shear resistance of the concrete, Vc=	1258	kips		5.7.3.3-3
factored concrete shear resistance, φVn=	1132	kips	Demand<Concrete Capacity OK	

Footing Design: Service [Longitudinal]

Control of cracking: Service

References:

User Input=				**AASHTO LFRD 9TH EDITION**
yield stress of steel, fy=	60	ksi		
Cover=	3	in		
longitudinal bar dia=	1.128	in		
direction 2 bar dia=	0.75	in		
transverse reinforcement bar dia=	0.44	in		
distance of extreme tension fiber to center of flexural reinforcement, dc=	3.564	in		
Thickness of footing, h=	48	in		
ratio of flexural strain, βs=	1.11			5.6.7-2
tensile stress in nonprestressed reinforcement, fss=	36	ksi		
exporsure factor, γe=	1			Note: Class 1 exposure
smax<=	10.32			5.6.7-1
spacing of nonprestressed reinforcement in layer closest to the tension face, s=	10		Spacing is satisfied	

Footing Design: Extreme [Longitudinal]

Critical Section Forces: Extreme, Longitudinal Bending

resultant force, R=	1357	kips		
resultant moment, M=	6581	k-ft		
eccentricity, e=	4.85			
width of wall=	2.5	ft		
width of footing=	14.0	ft		
long dimension, L=	28.0	ft		
height of footing, h=	4.0	ft		
maximum soil stress, σmax=	420.7	k-ft		
minimum soil stress, σmin=	0.0	k-ft		

Flexural Critical Cross Section

location of critical bending plane from far edge=	8.3	ft		
location of critical bending plane from close edge=	5.8	ft		
width of shear stress distribution=	6.5	ft		
soil stress at bending plane, σf=	45.7	k-ft	Triangular	10.6.5 (LRFD)
Factored moment demand at critical section, Mu=	4889	k-ft	Centroid*Area	

Shear Critical Cross Section

dv/2=	20.1	in		
location of critical bending plane from far edge=	9.9	ft		
location of critical bending plane from close edge=	4.1	ft		
width of shear stress distribution=	6.5	ft		
soil stress at bending plane, σf=	154.8	k-ft	Triangular	10.6.5 (LRFD)
Factored shear demand at critical section, Vu=	1173	kips	Area	
Factored moment demand at critical section, Mu=	2760	k-ft	Centroid*Area	

Flexural Design: Extreme, Longitudinal Bending

References:

User Input=					**AASHTO LFRD 9TH EDITION**
direction 2 reinforcement bar size=	5	#			Note: horizontal top bars
bar dia=	0.625	in			
bar area=	0.31	in2			
direction 2 reinforcement bar size=	6	#			Note: horizontal bottom bars
bar dia=	0.75	in			
bar area=	0.44	in2			
resistance factor, φ(flexure)=	1				5.5.4.2
steel yield, fy=	68	ksi			5.6.2.1
design concrete compressive strength, f'c=	5.2	ksi			
Cover=	3	in			As built
bar size=	9	#			As Built
spacing=	10	in			As Built
bar dia=	1.128	in			
bar area=	1	in2			
Thickness of footing, h=	48	in			
Width of footing, b=	336.0	in			
Number of reinforcement bars=	34	#			Note:
Area of tensile steel, As=	34.0	in2			
distance from extreme comp. fiber to o.c. of tensile steel, ds=	43.1	in			
Area of compression steel, A's=	34.0	in2			Note: Per face, symmetric
distance from extreme comp. fiber to o.c. of tensile steel, d's=	4.94	in			
stress block factor, α1=	0.85				5.6.2.2
stress block factor, β1=	3.94				5.6.2.2
distance from extreme fiber to NA, c=	0.000	in			5.6.3.1.2-4
depth of equivalent stress block, a=	0.000	in			
Nominal flexural resistance, Mn=	7345	kip-ft			5.6.3.2.2-1
Factored flexural resistance, φMn=	7345	kip-ft	Capacity	Demand<Capacity OK	5.6.3.2.1-1
Factored Moment Load, Mu=	4889	kip-ft	Demand		5.6.3.2.1-1

Minimum Reinforcement

correction factor for source of aggregate, K1=	1				5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.1452	kcf			Table 3.5.1-1
concrete density modification factor,λ=	1				C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.547	ksi			5.4.2.6
flexural cracking vairability facgtor, γ1=	1.6				
ratio of specified yield to ultimate strength, γ3=	0.67				
section modulus, Sc=	129024	in3			
Cracking Moment, Mcr=	6308	kip-ft			5.6.3.3-1
1.33 times factored moment required, 1.33 x Mu=	6502	kip-ft			5.6.3.3-1
Lesser of (Mu, Mcr)=	6308	kip-ft	Demand	Demand<Capacity OK	5.6.3.3-1

Shear Design: Extreme, Longitudinal Shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	336.0	in			5.7.2.8
effective shear depth, dv=	40.16	in			C5.7.2.8-1
crack spacing parameter, sx=	40.2	in			5.7.3.4.2-3
aggregate size, ag=	0.75	in			As Built: aggregate size assumption
crack spacing parameter influenced by aggregate size, sxe=	40.2	in			5.7.3.4.2-7
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	34.0	in2			
Factored Moment Load, Mn=	2760	k-ft			
Factored Axial Load, Nu=	1357	kips			
Factored Shear Load, Vu=	1173	kips	Demand		
longitudinal tensile strain, εs=	0.002714				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	38.5	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	1.58	for sections at least minimum shear reinforcement			5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	1349	kips	Capacity	Demand<Concrete Capacity OK	5.7.3.3-3

Pier 2 Footing - Design Check
(Bending about wall strong axis)

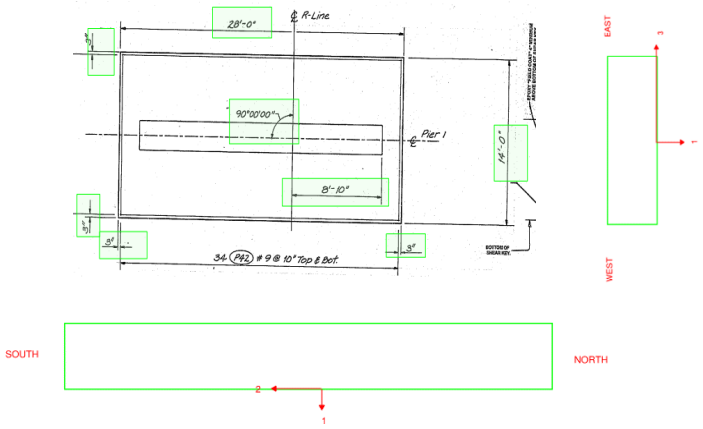
PROJECT: BR 17.7 - PIER 2 (Median Pier)
SUBJECT: Footing Design: About strong axis
BY: TOTG
CHK: RSGR

Purpose:
Design check of the as-built footing for strength, service and extreme limit states about strong axis

Methodology:
1. Geotechnical design of the footing is check per WSHDOT BDM and AASHTO LRFD Seismic.
2. Concrete Design is check for strength, service and extreme case limit states are per AASHTO LRFD 9th Edition

Asumptions:

Dimensions of Footing:



Bending in direction of Centerline of Pier

longitudinal footing thickness=	14	ft	Direction of 3
longitudinal wall thickness=	2.5	ft	Direction of 3
transverse footing thickness=	28	ft	Direction of 2
transverse wall thickness=	24	ft	Direction of 2
footing elevation thickness=	4	ft	

Factored Demands [Transverse]

Demand:

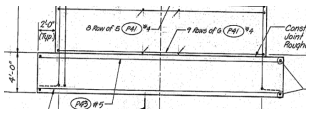
	P (kips)		V2 (kips)		T (kip-ft)		M3 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min
Strength	2439	1319	112	21	48	41	1352	968
EXT I EtoW	2508	1069	502	387	70	100	8790	10528
EXT I WtoE	2955	762	368	294	3	29	5368	6555
Service	1898	1418	84	7	1	39	1289	729

Vertcal DC Load= 1535 kips

Additional EV

User Input=

unit weight of soil, γEV= 130 pcf
length of footing, B= 28 ft
soil top surface of footing width, Bev= 4 ft
parellel to wall dimension, L= 14 ft



References:

WSHDOT BDM
AASHTO LRFD
AASHTO Seismic

Centerline of footing, C= 0 ft Center of Wall & Footing
Toe earth depth, Hev(toe)= 57 in Average depth
equivalent point load of earth above toe, Pev(toe)= 99.4 kips
distance from middle of wall, Xev(toe)= -7 ft
Heel earth depth, Hev(heel)= 108 in Average depth
equivalent point load of soil above heel, Pev(heel)= 188.4 kips
distance from middle of wall, Xev(heel)= 7 ft

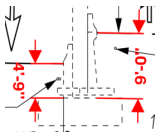


Fig 11.6.3.2-1 (LRFD)
Fig. 7.7.4-2 (BDM)

Combined Demand for increase envelope:

	P (kips)		V2 (kips)		T (kip-ft)		M2 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min
Strength	2727	1607	112	21	48	41	1352	968
EXT I EtoW	2796	1357	502	387	70	100	8790	10528
EXT I WtoE	3243	1050	368	294	3	29	5368	6555
Service	2186	1706	84	7	1	39	1289	729

Total Vertical Force= 1823 kips

Bearing Stress [Transverse]

Bearing Stress [Strength] - worst case

Factored Strength Bearing Resistance, qmax=	47.9	ksf			
bearing resistance factor, φb=	0.45				Geotech Report Table 10.5.5.2.2-1 (LRFD)
resultant force, R=	2727	kips			
resultant moment, M=	1352	k-ft			
eccentricity, e=	0.50		e<=Lr/6	OK for strength limits	10.6.3.3 (LRFD)
maximum soil stress, σmax=	107.7	k-ft		Trapezoidal	7.7.4 B (BDM)
minimum soil stress, σmin=	87.0	k-ft			
bearing pressure, q=	7.70	ksf			
C/D=	**6**			Demand<Capacity OK	

Bearing Stress [Extreme] - worst case

Factored Extreme Bearing Resistance, qmax=	95.9	ksf			
bearing resistance factor, φb=	0.45			k-ft	Geotech Report Table 10.5.5.2.2-1 (LRFD)
resultant force, R=	2796	kips			
resultant moment, M=	10528	k-ft			
eccentricity, e=	3.77	ft	e<=Lr/6		
maximum soil stress, σ=	180.4	k-ft		Uniform	7.7.4 B (BDM)
minimum soil stress, σmin=	19.3	k-ft			
bearing pressure, q=	12.89	ksf			
C/D=	**7**			Demand<Capacity OK	

Sliding [Transverse]

Failure by Sliding [Strength]

Max STR factored shear demand, Vu=	112	kips			
coefficient of friction, C=	0.8		for precast concrete		10.6.3.4-2 (LRFD)
angle of friction, θf=	38	deg.			
total vertical force, V=	1823	kips			
nominal sliding resistance, Rt=	1140	kips			
bearing resistance factor, φt=	0.8	0.8 factor for STR			Table 10.5.5.2.2-1 (AASHTO)
nominal sliding resistance, φt*Rn=	912	kips			6.3.5-1 (AASHTO Seismic)
C/D=	**8**			Demand<Capacity OK	

Failure by Sliding [Extreme]

Max EXT shear demand from wall, Vu=	502	kips			As-built design of wall
factored shear demand, Vu=	502	kips			
coefficient of friction, C=	0.8		for precast concrete		10.6.3.4-2 (LRFD)
angle of friction, θf=	38	deg.			
total vertical force, V=	1823	kips			
nominal sliding resistance, Rt=	1140	kips			10.6.3.4-2 (LRFD)
bearing resistance factor, φt=	0.9	0.9 factor for EXT			Table 10.5.5.2.2-1 (LRFD)
nominal sliding resistance, φt*Rn=	1026	kips			6.3.5-1 (Seismic)
C/D=	**2.0**			Demand<Capacity OK	

Overturning Stability [Transverse]

Overturning Stability [Strength]

Lp=	28	ft		
strength eccentricity, e=	0.50	ft		
Lp/6=	4.67	ft	Overturning Stability is OK	10.6.3.3 (LRFD)

Overturning Stability [Extreme]

middle 2/3 of Lp=	9.33	for γeq= 0	11.6.5 (LRFD)
middle 8/10 of Lp=	11.20	for γeq= 0.5	11.6.5 (LRFD)
for γeq= 0.5: resultant force, R=	2796		Note: Conservative does not include EV
for γeq= 0.5: resultant moment, M=	10528		
for γeq= 0.5: eccentricity, e=	3.77	Overturning Stability is OK	11.6.5 (LRFD)

Footing Design: Strength [Transverse]

Critical Section Forces: Strength, Transverse Bending

resultant force, R=	2727	kips		
resultant moment, M=	1352	k-ft		
eccentricity, e=	0.50			
width of wall=	24.0	ft		
width of footing=	28.0	ft		
parallel dimension, L=	14.0	ft		
height of footing, h=	4.0	ft		
maximum soil stress, σmax=	107.7	k-ft		
minimum soil stress, σmin=	87.0	k-ft		

Flexural Critical Cross Section

location of critical bending plane from far edge=	26.0	ft		
location of critical bending plane from close edge=	2.0	ft		
soil stress at bending plane, σf=	106.3	k-ft	Trapezoidal	10.6.5 (LRFD)
Factored moment demand at critical section, Mu=	214	k-ft	Centroid*Area	

Shear Critical Cross Section

dv/2=	19.6	in		
location of critical bending plane from far edge=	27.6	ft		
location of critical bending plane from close edge=	0.4	ft		
soil stress at bending plane, σf=	107.5	k-ft	Trapezoidal	10.6.5 (LRFD)
Factored shear demand at critical section, Vu=	40	kips	Area	
Factored moment demand at critical section, Mu=	7	k-ft	Centroid*Area	

Flexural Design: Strength, Transverse Bending

References:

AASHTO LFRD 9TH EDITION

User Input=					
direction 2 reinforcement bar size=	9	#			
bar dia=	1.128	in			
bar area=	1	in2			
direction 2 reinforcement bar size=	9	#			
bar dia=	1.128	in			
bar area=	1	in2			
resistance factor, φ(flexure)=	0.9				5.5.4.2
steel yield, fy=	60	ksi			5.6.2.1
design concrete compressive strength, f'c=	4	ksi			
Cover=	3	in			As built
bar size=	5	#			Note: horizontal top bars
spacing=	12	in			As Built
bar dia=	0.625	in			
bar area=	0.31	in2			
Number of reinforcement bars=	15	#			
bar size=	6	#			Note: horizontal bottom bars
spacing=	10	in			As Built
bar dia=	0.75	in			
bar area=	0.44	in2			
Number of reinforcement bars=	15	#			
Thickness of footing, h=	48	in			
Width of footing, b=	168.0	in			
Area of tensile steel, As=	6.6	in2			
distance from extreme comp. fiber to o.c. of tensile steel, ds=	43.5	in			
Area of compression steel, A's=	4.7	in2			Note: Per face, symmetric
distance from extreme comp. fiber to o.c. of tensile steel, d's=	4.44	in			
stress block factor, α1=	0.85				5.6.2.2
stress block factor, β1=	0.85				5.6.2.2
distance from extreme fiber to NA, c=	0.241	in			5.6.3.1.2-4
depth of equivalent stress block, a=	0.205	in			
Nominal flexural resistance, Mn=	1331	kip-ft			5.6.3.2.2-1
Factored flexural resistance, φMn=	1198	kip-ft	Capacity	Demand<Capacity OK	5.6.3.2.1-1
Factored Moment Load, Mu=	214	kip-ft	Demand		5.6.3.2.1-1

Minimum Reinforcement

correction factor for source of aggregate, K1=	1				5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.145	kcf			Table 3.5.1-1
concrete density modification factor,λ=	1				C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.480	ksi			5.4.2.6
flexural cracking vairability facgtor, γ1=	1.6				
ratio of specified yield to ultimate strength, γ3=	0.67				
section modulus, Sc=	64512	in3			
Cracking Moment, Mcr=	2766	kip-ft			5.6.3.3-1
1.33 times factored moment required, 1.33 x Mu=	285	kip-ft			5.6.3.3-1
Lesser of (Mu, Mcr)=	285	kip-ft	Demand	Demand<Capacity OK	5.6.3.3-1

Shear Design: Strength, Transverse shear

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	168.0	in			5.7.2.8
effective shear depth, dv=	39.15	in			C5.7.2.8-1
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	6.6	in2			
Factored Moment Load, Mn=	7	k-ft			
Factored Axial Load, Nu=	2727	kips			
Factored Shear Load, Vu=	40	kips	Demand		
longitudinal tensile strain, εs=	0.007343				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	54.7	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	0.74		for sections at least minimum shear reinforcement		5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	307	kips			5.7.3.3-3
Area of transverse steel, Av=	0.0	in2			Note: No steel strength considered
angle of inclination of transverse reinforcement to longitudinal axis, α=	90	deg.			
shear strength reduction factor accounting for the presence of PT ducts, λduct=	1				
spacing of transverse reinforcement, s=	8.3	in			
Nominal shear resistance of the steel, Vs=	0	kips			C5.7.3.3-1
Nominal shear resistance, Vn=	307	kips			5.7.3.3-1
factored shear resistance, φVn=	276	kips	Capacity	Demand<Capacity OK	5.7.2.1-1
Max nominal shear capacity, Vn2=	6577	kips			5.7.3.3-2

Footing Design: Service [Transverse]

Control of cracking: SER

References:

User Input=					**AASHTO LFRD 9TH EDITION**
yield stress of steel, fy=	60	ksi			
Cover=	3	in			
longitudinal bar dia=	0.625	in			
direction 2 bar dia=	1.128	in			
transverse reinforcement bar dia=	1	in			
distance of extreme tension fiber to center of flexural reinforcement, dc=	3.3125	in			
Thickness of footing, h=	48	in			
ratio of flexural strain, βs=	1.11				5.6.7-2
tensile stress in nonprestressed reinforcement, fss=	36	ksi			
exporsure factor, γe=	1				Note: Class 1 exposure
smax<=	11				5.6.7-1
spacing of nonprestressed reinforcement in layer closest to the tension face, s=	12			Spacing is marginally OK	

Footing Design: Extreme [Transverse]

Critical Section Forces: Extreme, Transverse Bending

resultant force, R=	2796	kips			
resultant moment, M=	10528	k-ft			
eccentricity, e=	3.77	ft			
width of wall=	24.0	ft			
width of footing=	28.0	ft			
long dimension, L=	14.0	ft			
height of footing, h=	4.0	ft			
maximum soil stress, σmax=	180.4	k-ft			
minimum soil stress, σmin=	19.3	k-ft			

Flexural Critical Cross Section

location of critical bending plane from far edge=	26.0	ft			
location of critical bending plane from close edge=	2.0	ft			
width of shear stress distribution=	31.0	ft			
soil stress at bending plane, σf=	168.8	k-ft	Uniform		10.6.5 (LRFD)
Factored moment demand at critical section, Mu=	353	k-ft	Centroid*Area		

Shear Critical Cross Section

dv/2=	20.0	in			
location of critical bending plane from far edge=	27.7	ft			
location of critical bending plane from close edge=	0.3	ft			
width of shear stress distribution=	31.0	ft			
soil stress at bending plane, σf=	178.5	k-ft	Uniform		10.6.5 (LRFD)
Factored shear demand at critical section, Vu=	60	kips	Area		
Factored moment demand at critical section, Mu=	10	k-ft	Centroid*Area		

Flexural Design: Extreme, Transverse Bending

References:

User Input=					**AASHTO LFRD 9TH EDITION**
direction 2 reinforcement bar size=	9	#			Note: horizontal top bars
bar dia=	1.128	in			
bar area=	1	in2			
direction 2 reinforcement bar size=	9	#			Note: horizontal bottom bars
bar dia=	1.128	in			
bar area=	1	in2			
resistance factor, φ(flexure)=	1				5.5.4.2
steel yield, fy=	68	ksi			5.6.2.1
design concrete compressive strength, f'c=	5.2	ksi			
Cover=	3	in			As built
bar size=	5	#			As Built
spacing=	12	in			As Built
bar dia=	0.625	in			
bar area=	0.31	in2			
Number of reinforcement bars=	15	#			
bar size=	6	#			As Built
spacing=	10	in			As Built
bar dia=	0.75	in			
bar area=	0.44	in2			
Number of reinforcement bars=	15	#			Note:
Thickness of footing, h=	48	in			
Width of footing, b=	168.0	in			
Area of tensile steel, As=	6.6	in2			
distance from extreme comp. fiber to o.c. of tensile steel, ds=	43.5	in			
Area of compression steel, A's=	4.7	in2			Note: Per face, symmetric
distance from extreme comp. fiber to o.c. of tensile steel, d's=	5.19	in			
stress block factor, α1=	0.85				5.6.2.2
stress block factor, β1=	3.94				5.6.2.2
distance from extreme fiber to NA, c=	0.045	in			5.6.3.1.2-4
depth of equivalent stress block, a=	0.179	in			
Nominal flexural resistance, Mn=	1489	kip-ft			5.6.3.2.2-1
Factored flexural resistance, φMn=	1489	kip-ft	Capacity	Demand<Capacity OK	5.6.3.2.1-1
Factored Moment Load, Mu=	353	kip-ft	Demand		5.6.3.2.1-1

Minimum Reinforcement

correction factor for source of aggregate, K1=	1				5.4.2.4-1
unit weight of concrete for modulus of rupture, wc=	0.1452	kcf			Table 3.5.1-1
concrete density modification factor,λ=	1				C5.4.2.8 & 5.4.2.8-2
Modulus of rupture, fr=	0.547	ksi			5.4.2.6
flexural cracking vairability facgtor, γ1=	1.6				
ratio of specified yield to ultimate strength, γ3=	0.67				
section modulus, Sc=	64512	in3			
Cracking Moment, Mcr=	3154	kip-ft			5.6.3.3-1
1.33 times factored moment required, 1.33 x Mu=	470	kip-ft			5.6.3.3-1
Lesser of (Mu, Mcr)=	470	kip-ft	Demand	Demand<Capacity OK	5.6.3.3-1

Shear Design: Extreme, Transverse Bending

References:

User Input=					**AASHTO LFRD 9TH EDITION**
resistance factor, φ(shear)=	0.9				
design concrete compressive strength, f'c=	4	ksi			
yield strength of transverse reinforcement, fyt=	60	ksi			
adjusted width, bv=	168.0	in			5.7.2.8
effective shear depth, dv=	39.99	in			C5.7.2.8-1
Modulus of elasticity, Es=	29000	ksi			
Area of tensile steel, As=	6.6	in2			
Factored Moment Load, Mn=	10	k-ft			
Factored Axial Load, Nu=	2796	kips			
Factored Shear Load, Vu=	60	kips	Demand		
longitudinal tensile strain, εs=	0.007632				5.7.3.4.2-4
angle of inclination of diagonal compressive stresses, θ=	55.7	deg.			5.7.3.4.2-3
factor indicating ability of diagonally cracked concrete to transmit tension and shear, β=	0.71	for sections at least minimum shear reinforcement			5.7.3.4.2-2
concrete density modification factor, λ=	1				5.4.2.8
Nominal shear resistance of the concrete, Vc=	303	kips			5.7.3.3-3
Area of transverse steel, Av=	0.0	in2			Note: Capacity for steel rebars not considered
angle of inclination of transverse reinforcement to longitudinal axis, α=	90	deg.			
shear strength reduction factor accounting for the presence of PT ducts, λduct=	1				
spacing of transverse reinforcement, s=	8.3	in			
Nominal shear resistance of the steel, Vs=	0	kips			C5.7.3.3-1
Nominal shear resistance, Vn=	303	kips			5.7.3.3-1
factored shear resistance, φVn=	273	kips	Capacity	Demand<Capacity OK	5.7.2.1-1
Max nominal shear capacity, Vn2=	6719	kips			5.7.3.3-2

Pier 2 Footing - Design Check
(Bending about both axis)

PROJECT:	BR 17.7 - PIER 2 (Median Pier)
SUBJECT:	Footing Design: Bi-Axial
BY:	TOTG
CHK:	RSGR

Purpose:
Design check bearing pressure for strength and extreme limit states under bi-axial loading.

Methodology:
1. Check per WSHDOT BDM and AASHTO LRFD 9th Edition.

Asumptions:

Bearing Stress [Bi-Axial]

Compile transverse and longitudinal demands:

	P* (kips)		V2 (kips)		V3 (kips)		T (kip-ft)		M2 (kip-ft)		M3 (kip-ft)	
	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min	Max	Min
Strength	2727	1607	112	21	56	32	48	41	1985	1149	1352	968
Extreme	2796	1357	502	387	250	221	70	100	6581	5523	8790	10528

*Addition of EV above footing included

Bi-Axial Bearing Pressure

User Input=		
length of footing in direction 2, L2=	28	ft
length of footing in direction 3, B3=	14	ft
thickness of footing, h=	4	ft
Ultimate soil pressure under seismic loads, q(str)=	47.9	ksf
Ultimate soil pressure under seismic loads, q(ext)=	95.9	ksf

Case	Factored Axial Load, P (kips)	Factored Moments, M2 Direction (k-ft)	Factored Moments, M3 Direction (k-ft)	Factored Effective Moments, M_R(k-ft)	Eccentricity $e_o=M_R/P$ (ft)	Resultant angle, θ=Atan(M2/M3) Deg.	Effective Length, L' (ft)	Effective Width, B' (ft)	Maximum bearing pressure, σ (ksf)	C/D	
Strength	2727	1985	1352	2401	0.88	55.7	27.01	12.54	8.0	**6.0**	**OK**
Extreme	1357	6581	10528	12416	9.15	32.0	12.48	4.30	25.3	**3.8**	**OK**

Overturning Stability [Bi-Axial]

Overturning Stability [Strength]

L2/3=	9.33	ft		
eccentricity in dir 2, e2=	0.50	ft	<Lp/3 OK	10.6.3.3 (LRFD)
B3/3=	4.67	ft		
eccentricity in dir 3, e2=	0.73	ft	<Lp/3 OK	10.6.3.3 (LRFD)

Overturning Stability [Extreme]

for yeq=0, e<2/3*L2/2=	9.33		
for yeq=1, e<8/10*L2/2=	11.2		
linear interpolate, e:	10.27		11.6.5 (LRFD)
At yeq=0.5, e2=	7.76	OK	11.6.5 (LRFD)
for yeq=0, e<2/3*B3/2=	4.67		
for yeq=1, e<8/10*B3/2=	5.6		
linear interpolate, e:	5.13		11.6.5 (LRFD)
At yeq=0.5, e3=	4.85	OK	11.6.5 (LRFD)

Pier 2 Top Dowel - EXT Case Design Check

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 2 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI
JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Purpose:
Design the hinge rebar (dowel) and Shear Key at Pier 2 to EXT Loads (DL; LL; EQ)
Design Code:
AASHTO LFRD 9th Edition Chapter 5.13 which refers to ACI 318 - 14 Chapter 17
AASHTO LFRD 9th Edition Chapter 5.7.4 for Interface shear
AASHTO SEISMIC DESIGN GUIDE
Asumptions:
1. Hinge rebar is a shear connector. It is designed to transfer shear force between superstructure and substructure inc. transfer of torsion from supersturcture as tension through the hinge rebars.
2. Total loads from superstructure/substructure are distributed per dowel to calculated max. load effect per dowel. i.e. Max Shear and Max. Tension in the critical dowel which is the dowel at the end
3. Provided no. of dowels are verified so that load demand per dowel is less than dowel capacity. The following failure modes are checked:
a) Steel failure in tension and shear
b) Concrete failure due to tension and shear
4. The transfer of shear force between superstructure and substructure will als ooccur via concrete interface shear in the shear key
Hence, concrete shear key is then checked to have enough capacity for the interface shear transfer

input

1.0 DIMENSIONS & PARAMETERS

nominal f'c	4	ksi	expected f'ce	5.20	ksi		concrete compressive strength (nominal and expected)
wc =	0.155	kcf					Unit weight of weight concrete
K1 =	1						Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4028	ksi					Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
fy =	60	ksi	fye =	68.00	ksi		Yield strength of reinforcement (nominal and expected)
f,uta=	90	ksi	f,e uta=	95	ksi	AASHTO seismic Table 8.4.2.1	Ultimate tensile strength of reinforcement (nominal and expected)
Es =	29000	ksi					Modulus of elasticity of reinforcement
η =	7.20						Modular ratio
β_1 =	0.85						Compression Zone Factor
ϕv=	1						
ϕf=	1						
Dowel Qty., N=	47	Nos.					

2.0 Spacing of dowels

Length of Wall=	24	ft	
Edge distance from CL of anchor=	8	in	the smallest edge distance in Trans. Direction, this value is 15in in Long. direction for the edge most bar
Spacing of anchors=	6	in	
Use=	6	in	

3.0 Horizontal Force demands (between superstructure and substructure)

Seismic EtoW governs here

Main Shear force in Bridge Longitudinal Direction **100%L + 30%T**

LEP EQ Demand, L1=	42	kips	From Seismic earth pressure behind Pier 3
EDA EQ Demand, L2=	185	kips	From RSA **(Seismic EtoW)**, applied with a factor of 1.2
LS, EQ Demand, L3=	1.5	kips	From Live Load Surcharge behind Pier 3 (0.5 factor used)
Live Load LL, L4=	20	kips	Apply a factor of 0.5 in combination below

Associated Transverse shear forces (if any)

LEP Demand, T1=	44	kips	From Seismic earth pressure behind Pier 3
EDA Demand, T2=	216	kips	From RSA **(Seismic EtoW)**, applied with a factor of 1.2
LS Demand, T3=	1	kips	From Live Load Surcharge behind Pier 3 (0.5 factor used)
Live Load LL, T4=	44	kips	Apply a factor of 0.5 in combination below

Combined shear using SRSS

1=	61	kips
2=	284	kips
3=	2	kips
4=	24	kips
Total=	371	kips

Shear per dowel=	7.9	kips	Total shear load / nos. of dowels
Shear per dowel (transverse only)=	6.0	kips	
Shear per dowel (longitudinal only)=	5.1	kips	

Main Shear force in Bridge Transverse Direction **30%L + 100%T**

LEP Demand, T1=	53	kips
EDA Demand, T2=	510	kips
LS Demand, T3=	1	kips
Live Load LL, T4=	44	kips

Associated Longitudinal shear forces (if any) 30%L + 100%T

LEP EQ Demand, L1=	13	kips
EDA EQ Demand, L2=	63.6	kips
LS, EQ Demand, L3=	1.5	kips
Live Load LL, L4=	20	kips

Combined shear using SRSS

1=	54	
2=	514	
3=	2	
4=	24	
Total=	594	kips

Shear per dowel=	12.6	kips
Shear per dowel (transverse only)=	1.9	kips
Shear per dowel (longitudinal only)=	12.5	kips

4.0 Vertical Demands - Compression

DL=	994	kips	
LL=	0	kips	
EQ=	-950	kips	(- 30% V)
Min. Compression under EQ =	44.4	kips	
Compression per dowel=	0.94	kips	

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 2 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

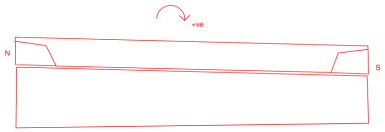
JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

5.0 Vertical Demands - Tension due to superstructure torsion

EQ (This can be + or -) =	700	k-ft
Seismic Earth pressure behind Pier 3=	120	k-ft
Live load surcharge behind Pier 3=	5	k-ft
DL=	150	k-ft
Max Torsion LL=	500	k-ft
Min Torsion, LL=	-700	k-ft

From RSA **(Seismic EtoW)**, applied with a factor of 1.2

Apply a factor of 0.5 in combination
Apply a factor of 0.5 in combination



Max Torsion, Tmax	1225	k-ft
Min Torsion, Tmin	-775	k-ft

Estimate the tension forces caused by torsion

Assumption: Torsion causes tension force in half of the dowel

Half nos. of dowels=	24	
Spacing=	6.0	in

Dowels experience tension force relative to their distance S from structure CL

Tension force in dowel = Tmax x Si / ∑(S^2); Tmax = 3603 kip/ft

	Si (ft)	S^2	Tension (kips)	Compression	Net Force (-ve is tension)	
S1	0.5	0.25	-0.500	0.94	0.445	
S2	1	1	-1.000	0.94	-0.055	
S3	1.5	2.25	-1.500	0.94	-0.555	
S4	2	4	-2.000	0.94	-1.055	
S5	2.5	6.25	-2.500	0.94	-1.555	
S6	3	9	-3.000	0.94	-2.055	
S7	3.5	12.25	-3.500	0.94	-2.555	
S8	4	16	-4.000	0.94	-3.055	
S9	4.5	20.25	-4.500	0.94	-3.555	
S10	5	25	-5.000	0.94	-4.055	
S11	5.5	30.25	-5.500	0.94	-4.555	
S12	6	36	-6.000	0.94	-5.055	
S13	6.5	42.25	-6.500	0.94	-5.555	
S14	7	49	-7.000	0.94	-6.055	
S15	7.5	56.25	-7.500	0.94	-6.555	
S16	8	64	-8.000	0.94	-7.055	
S17	8.5	72.25	-8.500	0.94	-7.555	
S18	9	81	-9.000	0.94	-8.055	
S19	9.5	90.25	-9.500	0.94	-8.555	
S20	10	100	-10.000	0.94	-9.055	
S21	10.5	110.25	-10.500	0.94	-9.555	
S22	11	121	-11.000	0.94	-10.055	
S23	11.5	132.25	-11.500	0.94	-10.555	
S24	12	144	-12.000	0.94	-11.055	Max. tension in edge dowel
						Max. tension in edge dowel

Total, ∑(S^2)= 1225 ft

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6.0 Verify Minimum Nos. of Dowel - Single Anchor shear capacity - Vsa

Dowel Size #=	9	#				
Dowel Area, Ad=	1	in2				
Dowel diam., d=	1.128	in				
Expected Dowel Utlimate Tensile Strength, f-ue=	95	ksi				
Shear Resistance Factor, φ dowel=	1					
Nominal Resistance, Vn=	57	kips		Max. Demand	D/C	=0.6 x Ad x futa ACI 318 eq 17.5.1.2b
Factored Resistance, Phi x Vn=	57	kips	>	13	22%	

7.0 Verify Minimum Nos. of Dowel - Single Anchor Tensile capacity = Nsa

Dowel Size #=	9	#				
Dowel Area, Ad=	1	in2				
Dowel diam., d=	1.128	in				
Expected Dowel Utlimate Tensile Strength, f-ue=	95	ksi				
Tensile Resistance Factor, φ dowel=	1.00					
Nominal Resistance, Tn=	95	kips		Max. Demand	D/C	= Ad x futa ACI 318 eq 17.4.1.2
Factored Resistance, Phi x Vn=	95	kips	>	11	12%	

8.0 Verify Layout of Anchors in Concrete - Single Anchor

8.1 Concrete breakout due to tension - Ncb

C,a1=	8	in	Min. edge distance in transverse direction	
H,ef used.=	5	in	Use this for tension breakout calcs	ACI 318 cl17.4.2.3
H,ef prov.=	36	in	embedment length into concrete	
A, Nc=	256	in2	Effective concrete area around the edge anchor	ACI 318 cl17.4.2.1c
A, Nco=	256	in2	Projected area of single anchor = 9 x hef x hef	ACI 318 Fig R17.4.2.1
Nb	21	kips	Basic breakout strength of single anchor in tension	ACI 318 CL17.4.2.2a
kc	24		cast in anchor	
gamma a	1		aggregate factor	
sqrt f'c	72.111			
hef	5.333	in		
ψ, ed,N=	1		ca, min = 8in $\geq$ 1.5 h,ef = 7.5	
ψ, c,N=	1.25		cast-in-anchors	
ψ, cp,N=	1		cast-in-anchors	
Factor=	1.00		Max. Demand	D/C
Factored Concrete breakout, Ncb=	27	kips	> 11	41%

8.2 Concrete breakout due to Shear at edge dowel Vcb

Shear in bridge transverse direction

Ca1=	15	in	min. distance to edge parallel to edge	
Ca2=	8	in	min. distance to edge perpendicular to edge	
Avco=	288	in2	use smaller Ca	ACI 318 eq 17.5.2.1c
le=	9	in	le=he=load bearing length of dowel	
da=	1.128	in		diameter of dowel
sqrt f'c=	72.11			
Vb=	18.39	kips	use smaller Ca	ACI 318 eq 17.5.2.2a
Limit Vb=	14.69	kips	use smaller Ca	ACI 318 eq 17.5.2.2b
Use limit				
hef=	12	in		effective embedment for shear breakout = 1.5 x Min (Ca1, Ca2)
ha=	40	in		Bearing height
Avc=	288	h	single anchor at edge	
ψ, ed,V=	1			
ψ, c,V=	1.2		Cracking at ext loads	
ψ, h,V=	1		ha=1.5ca	

Shear Resistance Factor, φ = 1

Max. Trans. Demand D/C: 13 | 72%

Factored Concrete breakout, Vcb= 17.6 kips

ACI 318 CL17.5.2.1a

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8.3 Concrete pryout due to Shear at edge dowel Vcp

Vcp=kcp x Ncb

kcp=	2	
Ncb=	26.6	kips
Factor, φ=	1	
Factored Vcp=	53.3	kips
Factored Demand, Vu=	12.6	kips
D/C=	24%	

ACI 318-14 eq. 17.5.3.1a

8.4 Shear and Tension Interaction

Max Shear D/C, Vdc=	72%		
Max Tension D/C, Tdc=	41%		
	0.806	OK	$Vdc^{5/3} + Tdc^{5/3} < 1.0$

ACI 318-14 R17.6

9.0 Verify Layout of Anchors in Concrete - Group Anchor

9.1 Concrete breakout due to tension - Ncbg

C,a1=	8	in	Min. edge distance in longitudinal direction	
H,ef used.=	5.333	in	Use this for tension breakout calcs	ACI 318-14 cl17.4.2.3
H,ef prov.=	36	in	embedment length into concrete	
A, Nc=	4672	in2	Effective concrete area around the edge anchor	ACI 318-14 cl17.4.2.1c
A, Nco=	256	in2	Projected area of single anchor = 9 x hef x hef	ACI 318-14 Fig R17.4.2.1
Nb	21	kips	Basic breakout strength of single anchor in tension	ACI 318-14 CL17.4.2.2a
kc	24		cast in anchor	
gamma a	1		aggregate factor	
sqrt f'c	72.111			
hef	5.333	in		
ψ, ed,N=	1		ca, min = 8in $\geq$ 1.5 h,ef = 7.5in	
ψ, ec,N=	1		cast-in-anchors	
ψ, cp,N=	1		cast-in-anchors	
ψ, c,N=	1.25			
Factor=	1.00			

			Max. Tension Demand	D/C
Factored Concrete breakout, Ncb=	389	kips	> 105	27%

9.2 Concrete breakout due to Shear at edge dowel Vcbg - Shear in bridge long. Direction

Shear in bridge long. direction

Ca2=	15	in	min. distance to edge parallel to edge	
Ca1=	8	in	min. distance to edge perpendicular to edge	
s=	5.9	in		
Avco=	1012.5	in2		ACI 318 eq 17.5.2.1c
le=	9.024	in	le=he=load bearing length of dowel	
da=	1.128	in		diameter of dowel
sqrt f'c=	72.11			
Vb=	47.21	kips		ACI 318 eq 17.5.2.2a
Limit Vb=	37.70	kips		ACI 318 eq 17.5.2.2b
Use limit				
he=	12	in	note that the actual embedment is 40"	effective embedment for shear breakout = 1.5 x Ca1
Avc=	3852	in2	group of anchors	
ψ, ec,V=	1			
ψ, ed,V=	1		Ca2 24" > 1.5Ca1 18"	
ψ, c,V=	1.4		Cracking at ext loads	
ψ, h,V=	1		ha=1.5ca	

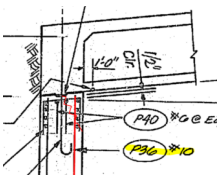
			Max. long. Demand	D/C
Shear Resistance Factor, φ =	1			
Factored Concrete breakout, Vcb=	200.8	kips	594	296%

ACI 318 CL17.5.2.1b

Check for main wall reinforcement at wall face to resist concrete breakout

Bar #	10		#10 rebar at 12" spa. Throughout wall
Qty.	23		
Area of steel, As=	29.21	in2	
f'ye	68.000	ksi	expected steel yield strength
Nominal Steel Strength, Vs=	1986.28	kips	=f'ye x As
Factor, φ=	1		
Factored steel strength, f x Vs=	1986	kips	
Total Factored Strength, Concrete + Steel, =	2187.1	kips	

Max. Trans. Demand D/C: 594 27%



Hence, stirrups reinforcement will prevent concrete breakout in bridge long. Direction

10.0 Interface shear resistance (AASHTO LRFD BDS 2021, 5.7.4)

This happens in superstructure where f'c = 5 ksi

- For normal weight concrete placed monolithically:

c = 0.40 ksi
μ = 1.4
K_1 = 0.25
K_2 = 1.5 ksi

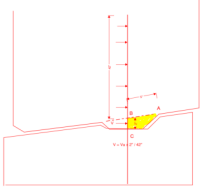
cohesion factor	c=	0.4	ksi	
Friction factor	u=	1.4		
Fraction of concrete strength	K1=	0.25		
Limiting interface shear resistance	K2=	1.5	ksi	
	b	5	in	(3" on top and 2" in height)
	L	288	in	Length of shear key failure plane
Area of concrete in interface	Acv=	1440	in2	b x L
Nominal Shear Resistance of interface	Vni=	576	kips	c x Acv
	fye=	0	ksi	Rebar Steel strength (nominal)
	Avf=	0	in2	Steel reinforcement area in shear key
	Pc=	0	kips	No compression assumed
Factor=		1		

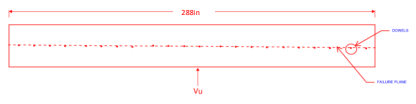
				Max. Demand		D/C
Factored resistance=	576	kips	>	132	=351 x 2"/le"	23%

Max Limits for resistance

				Max. Demand	D/C
AASHTO LRFD BDS 5.7.4.3-4	V1	1800	kips	> 132	7%
AASHTO LRFD BDS 5.7.4.3-5	V2	2160	kips		

Hence, concrete shear key shall will not fail and the failure will be governed by checks in section 6 to 9





Book: I8 CALC BOOK COMPLETION DATE: **2021 Dec 05**

Title: Existing Pier 3 (East Abutment) - Design Checks

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I8-0	-	I8-0	Calc Register (this sheet)	N/A	N/A	
2	I8-1	-	I8-1	Pier 3 Check Methodology	RSGR	MWBM	
3	I8-2	-	I8-9	Point of Fixity Analysis (Static Case)	RSGR	MWBM	
4	I8-10	-	I8-17	Point of Fixity Analysis (Seismic Case)	RSGR	MWBM	
5	I8-18	-	I8-22	Construction Analysis : Max force to restrain Pier 3	RSGR	MWBM	
6	I8-23	-	I8-27	Construction Analysis : Locked in stress in due to 1" Pier 3 movement	RSGR	MWBM	
7	I8-28		I8-29	Reinforcement arrangement along the shaft	RSGR	MWBM	
8	I8-30		I8-35	Reinforcement Design Check (EXT Case)	RSGR	MWBM	
9	I8-36		I8-42	Reinforcement Design Check (STR Case)	RSGR	MWBM	
10	I8-43		I8-46	Check AASHTO SEISMIC Guide Requirements	RSGR	MWBM	
11	I8-47		I8-49	Check WSDOT BDM Requirements	RSGR	MWBM	
12	I8-50		I8-55	Top Shear Key Design (EXT CASE)	RSGR	LDLI	
		-					
		-					
		-					

1. The fixity of Pier 3 shafts in model were set at 47ft below top of pile cap. Per NAVD88, top of pile cap is at EL107 in the middle. Hence, the bottom of shafts are set fixed at EL60. The point of fixity was decided by converging Moment and Shear diagrams for the part of shafts above i405 level (~EL 74) between CSI model and Geo-engineers PY-Wall model.

2. The section properties and loadings were made consistent between the CSI model and PY-Wall model. Full height of shaft is modeled in PY-Wall starting at top of pile cap. (i.e. 0 is set at top of pile cap in PY-Wall).

3. For static case, pier 3 top does not show significant displacement in CSI model (~0.05"). Hence, top displacement in PY-Wall Pier 3 model is set to 0. The moment and shear diagrams for similar loading conditions are then matched by adjusting the point of fixity in CSI model. Similar process is repeated for the seismic case. However, for seismic case, the bridge provides some lateral restraint at top of pier 3. This force is also applied and adjusted at pier 3 top in PY-Wall in order to converge moment and shear diagram with CSI model.

4. Pier 3 shafts are then checked for flexure, shear and torsion. For East to West EQ, demands above i405 level are taken as maximum of the values between CSI model and PY-wall model. Demands at the point of fixity are taken from PY-wall model as they are more realistic and account for soil-structure interaction.

5. For West to East EQ, demands are directly read from CSI model which include passive springs behind Pier 3 and lateral earth pressure and live load surcharge behind Pier 1 (as discussed in book i6 page 8 and 9.

6. Pier 3 is also analyzed for construction stage. This stage is checked to estimate the amount of lateral restraint needed at Pier 3 top to restrict any movement due to earth pressure behind Pier 3 and about 20ft of excavation behind existing Pier 1. This restraining force is estimated to be around 850 kips.

7. Although Pier 3 top should is restrained, there will still be some deflection resulting from elastic deformation of support structure. Hence, 1" allowance is set and checked to estimate the amount of locked-in stress such a movement would induce at the point of fixity. These locked in stress at the base is added to the shaft demands in all STR and EXT case.

PIER 3 : Point of Fixity Analysis (Static case)

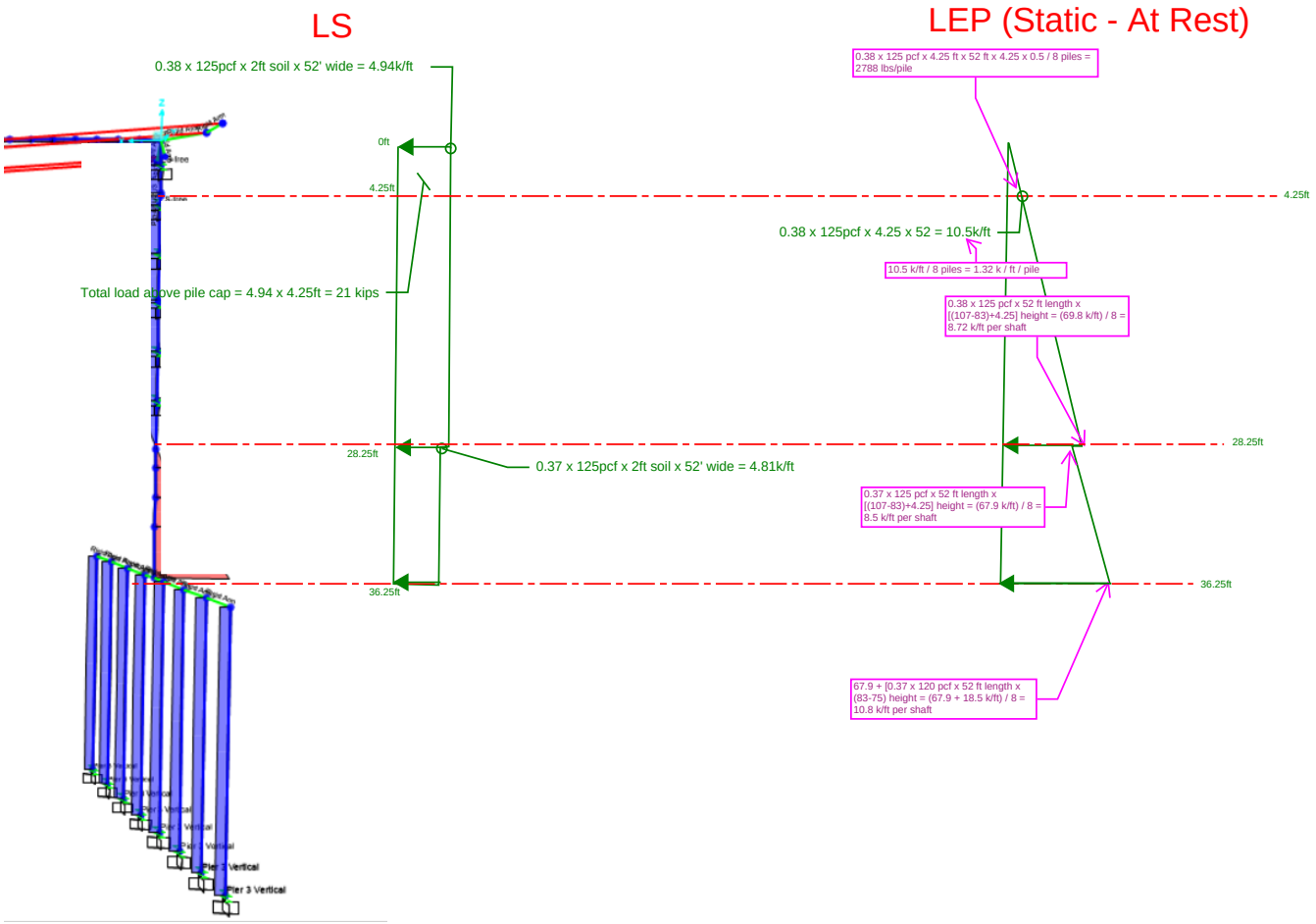
Pier 3 - LEP (Static at rest) AND LS LOADS

The increase in horizontal pressure due to live load surcharge may be estimated as:

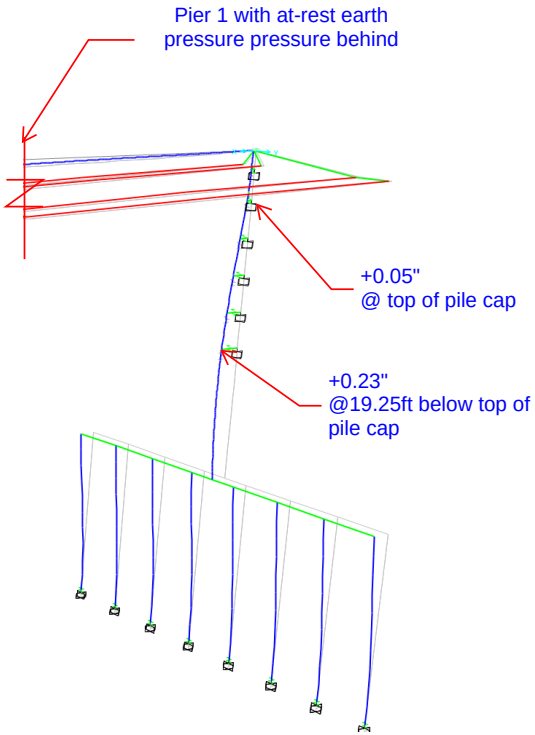
$\Delta_p = k\gamma_s h_{eq}$ (3.11.6.4-1)

where:

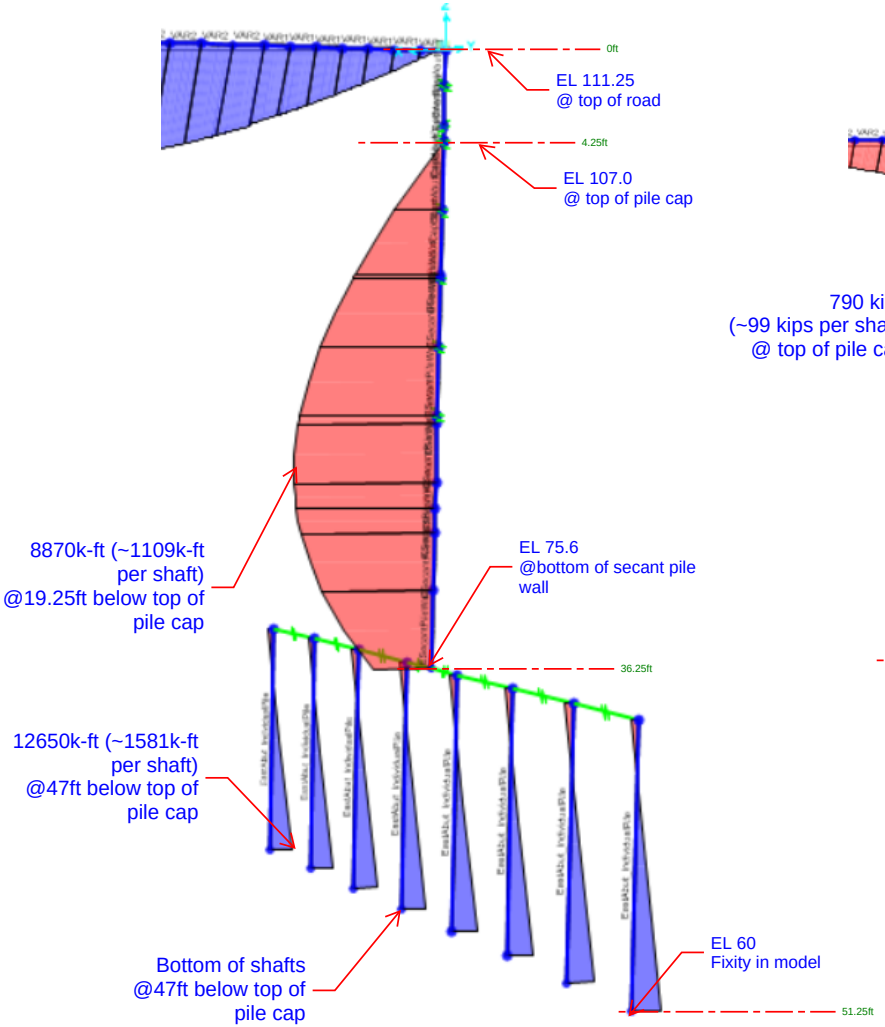
Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft) = 2ft (consider 2ft of SER case as LS factor=1)



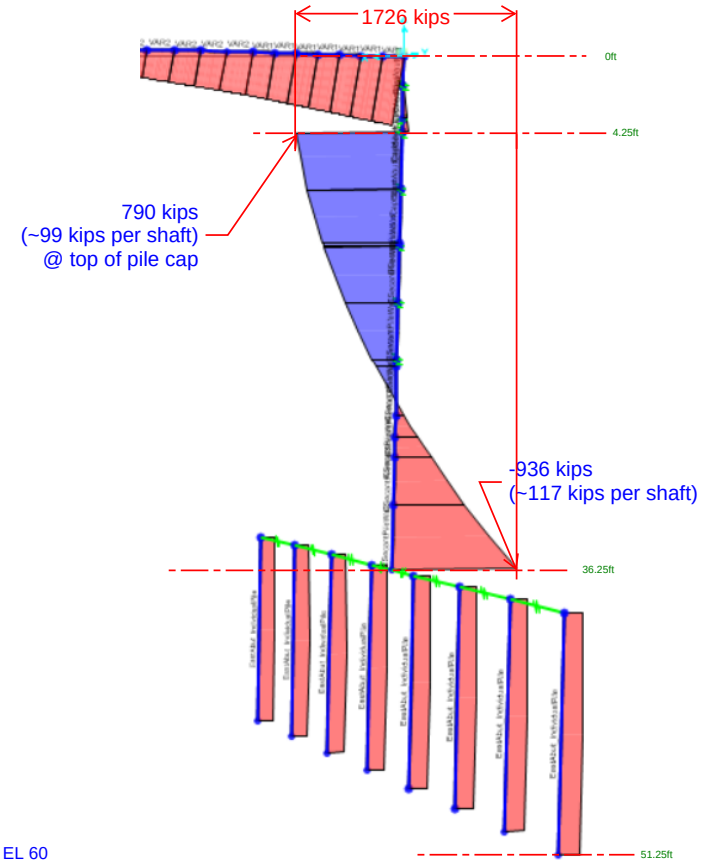
Pier 3 - At rest Pressure (Static) + LS



Deflection Diagram
(positive towards pier 2)

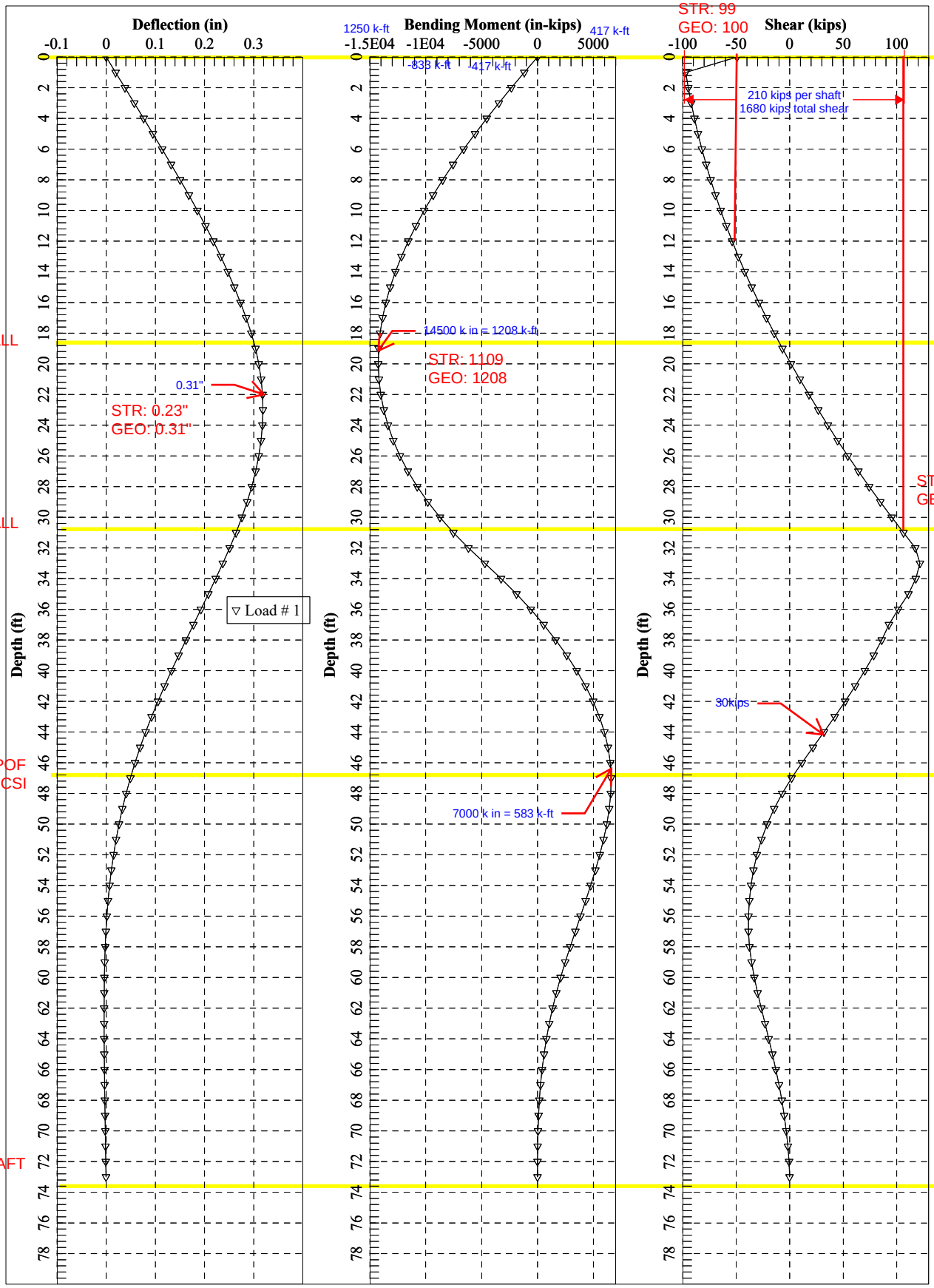


Moment Diagram
(about the weak axis of pier)



Shear Diagram

PIER 3 - STATIC
AT REST LEP + LIVE SURCHARGE OF 2'
11-23-2021
22.3kips TOTAL LATERAL LOAD AT TOP OF PILES (2.8kip/pile) DUE TO LEP ON END DIAPHRAGM
K_at-rest APPLIED TO SOIL LAYERS ABOVE I-405, AUTOMATIC CALCULATION OF LEP BELOW I-405
RESULTS ARE PER PILE, ASSUMED PILE SPACING = 6'-8"



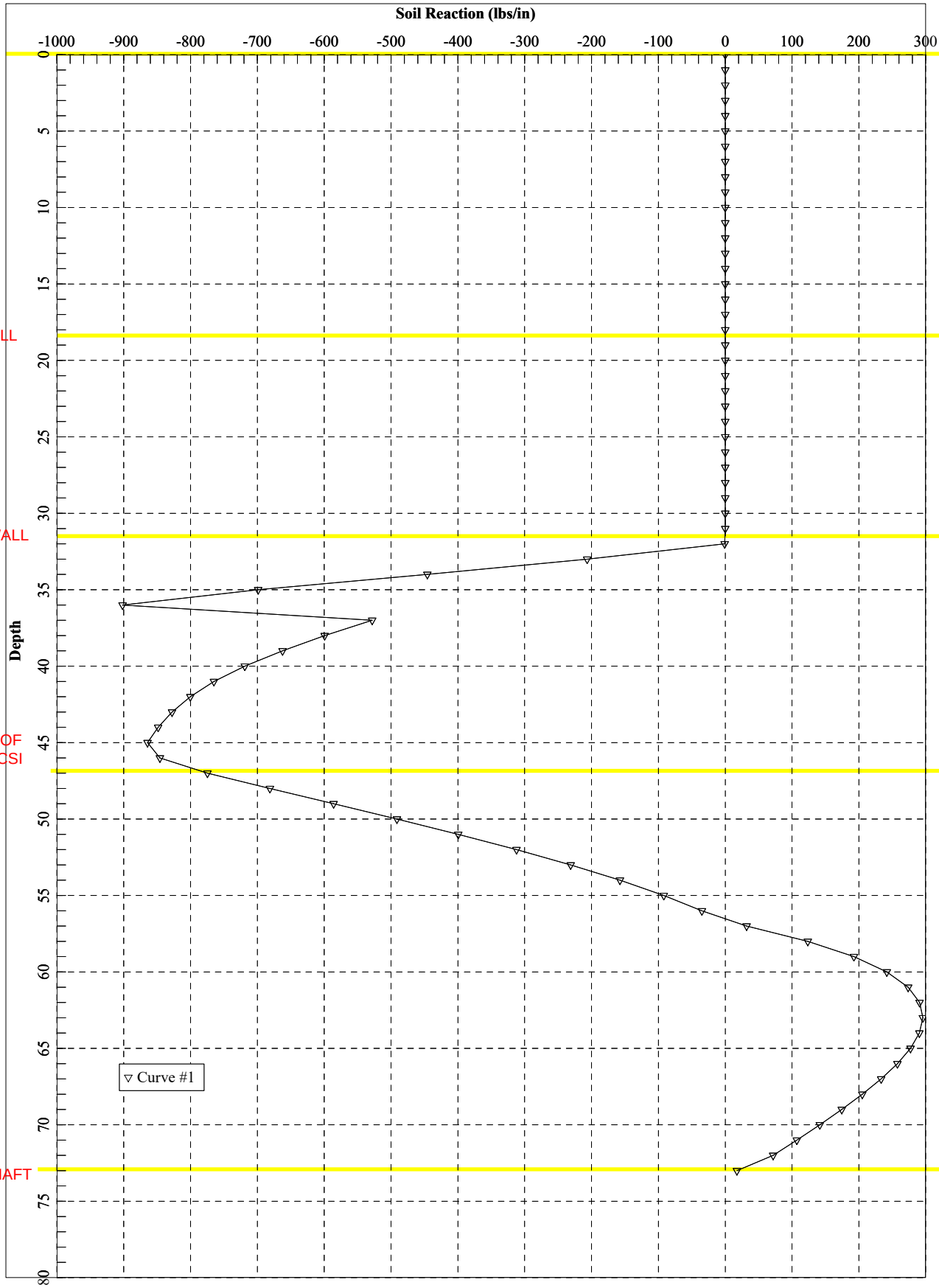
HINGE LINE
EL 107.0

B/ FASCIA WALL
EL 88.5

B/ SECANT WALL
EL 75.6

EQUIVALENT POF
MODELLED IN CSI
BRIDGE
EL 60

B/ DRILLED SHAFT
EL 34.6



HINGE LINE
EL 107.0

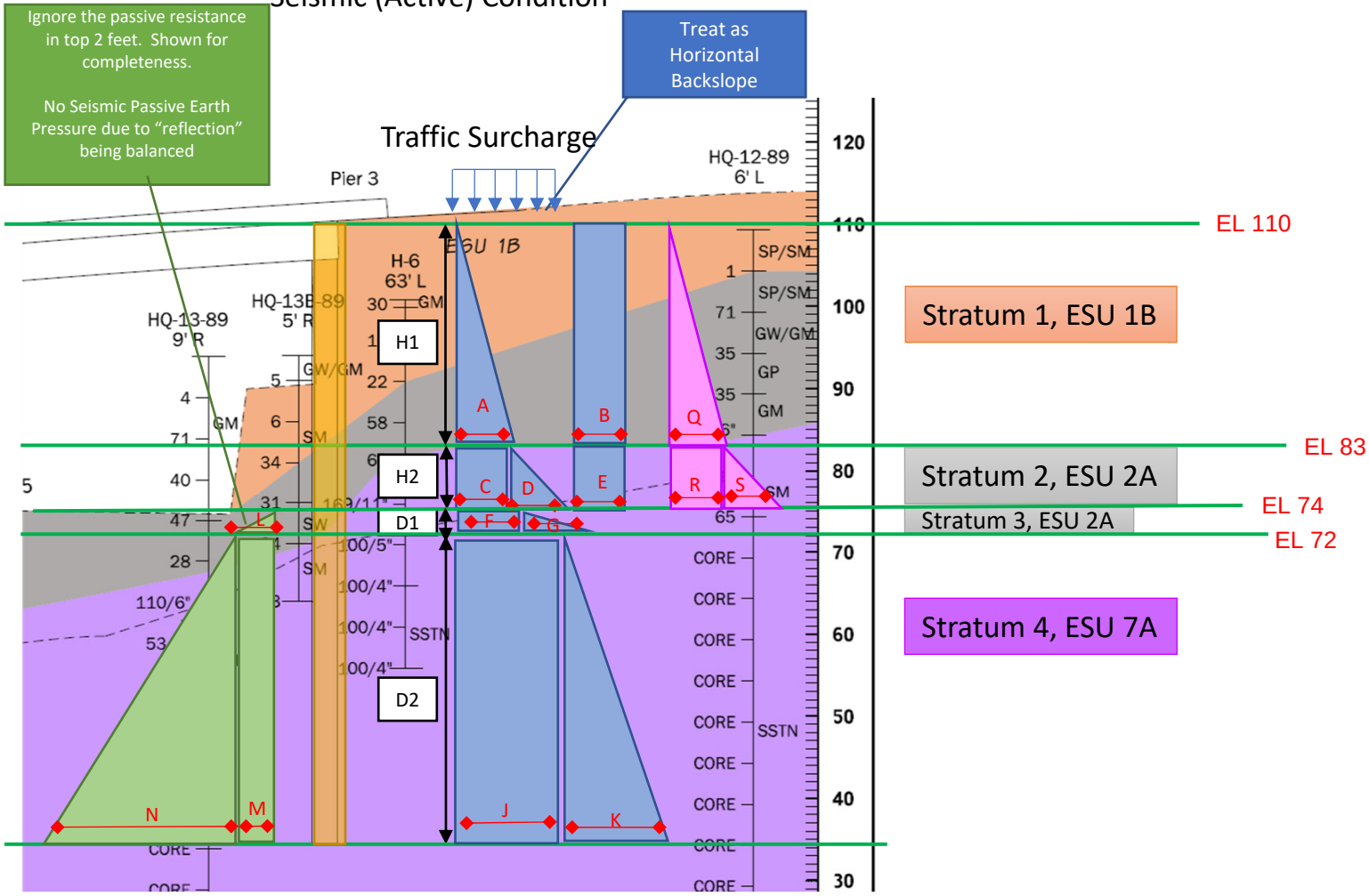
B/ FASCIA WALL
EL 88.5

B/ SECANT WALL
EL 75.6

EQUIVALENT POF
MODELLED IN CSI
BRIDGE
EL 60

B/ DRILLED SHAFT
EL 34.6

Bridge 17.7
Pier 3 Lateral Earth Pressures
Seismic (Active) Condition



NOT TO SCALE

To Calculate LEP for each stratum, sum the areas indicated.
For Example, Stratum 2, C+D+E+R+S = LEP

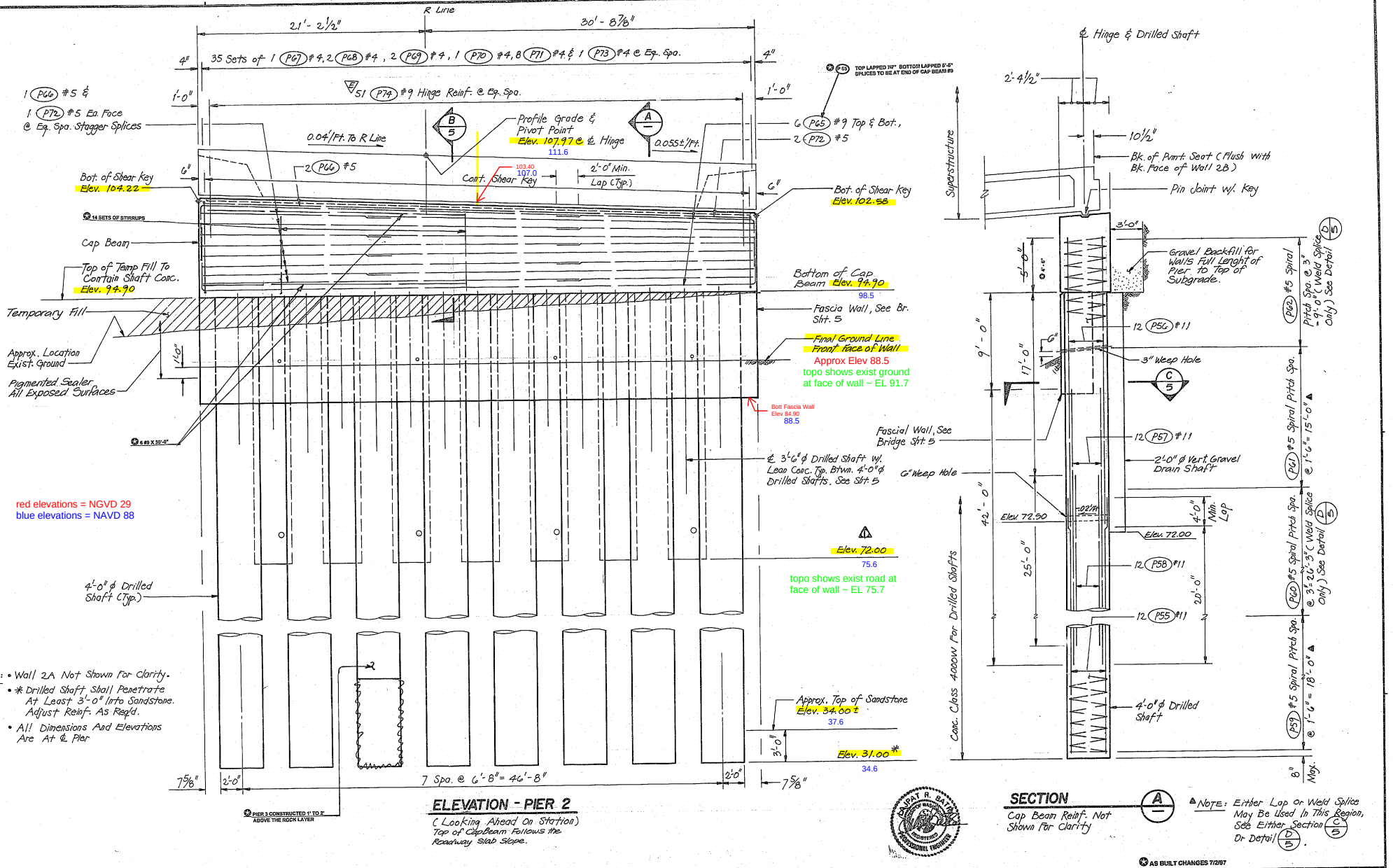
Seismic (Active) Lateral Earth Pressures

Ground Surface Elevation at Top of Wall	110	ft	H1	27	ft
Ground Surface Elevation at NB 405	74	ft	H2	9	ft
Wall Toe Elevation	35	ft	D1	2	ft
Groundwater Level Elevation	72	ft	D2	37	ft
Traffic Surcharge, Static	250	psf			
Traffic Surcharge, Seismic	125	psf			

Interval	Layer Description	Elevation, Top of Layer ft	Elevation, Bottom of Layer ft	Friction Angle deg	Soil Total Unit Weight pcf	Soil Effective Unit Weight pcf	Active Lateral Earth Pressure Coefficient	At Rest Lateral Earth Pressure Coefficient	Passive Lateral Earth Pressure Coefficient	Seismic Lateral Earth Pressure Coefficient
Stratum 1	ESU 1B, above water table	110	83	38	125	125	0.22	0.38	4.20	0.41
Stratum 2	ESU 2A, above water table	83	74	39	120	120	0.21	0.37	4.40	0.40
Stratum 3	ESU 2A, above water table	74	72	39	120	120	0.18	0.37	4.40	0.40
Stratum 4	ESU 7A, below water table	72	35	42	135	72.6	0.28	0.33	5.04	0.38

	Seismic (active) (psf)								
	A	B	C	D	E	F	G	J	K
Stratum	1	1	2	2	2	3	3	4	4
Description	earth pressure, H1	traffic surcharge	earth pressure, H1	earth pressure, H2	traffic surcharge	earth pressure, H1 and H2	earth pressure, D1	earth pressure, H1, H2, D1	earth pressure, D2
Recommended Earth Pressures	732	27	703	225	26	820	89	1291	739
To calculate	27.1 * H1	27	26.02 * H1	24.98 * H2	26	23 * H1 + 22.08 * H2	22.08 * D1	34.38 * H1 + 33 * H2 + 33 * D1	19.97 * D2

	Passive (psf)			Seismic (psf)		
	L	M	N	Q	R	S
Stratum	3	4	4	1	2	2
Description	earth pressure, D1	earth pressure, D1	earth pressure, D2	seismic earth pressure, H1	seismic earth pressure, H1	seismic earth pressure, H2
Recommended Earth Pressures	0	1211	13551	1399	1364	436
To calculate	527.46 * (D1 - 2 ft)	605.36 * D1	366.24 * D2	51.82 * H1	50.52 * H1	48.5 * H2



DRAWN	P. Lee				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
DESIGN	J. King				10	WASH	ACI-405-3(780)		
CHECK	K. Pirbazari				JOB NUMBER				
PROJ. ENGR.					90W069				
DIST. ADM.		97-90	1	Revised Elev.	RJS	CK	CONTRACT NO.		
		DATE		REVISION	BY	APP'D	3854		

APPROVED

Washington State Department of Transportation

INCA ENGINEERS INC.

SR 405
TUKWILA TO SUNSET BLVD
CEDAR RIVER PIPELINES-RELOCATION
RENTON AVENUE U'XING 405/17B

PIER 2

SHEET 4 OF 317 SHEETS

Book 18 Page 9 of 55

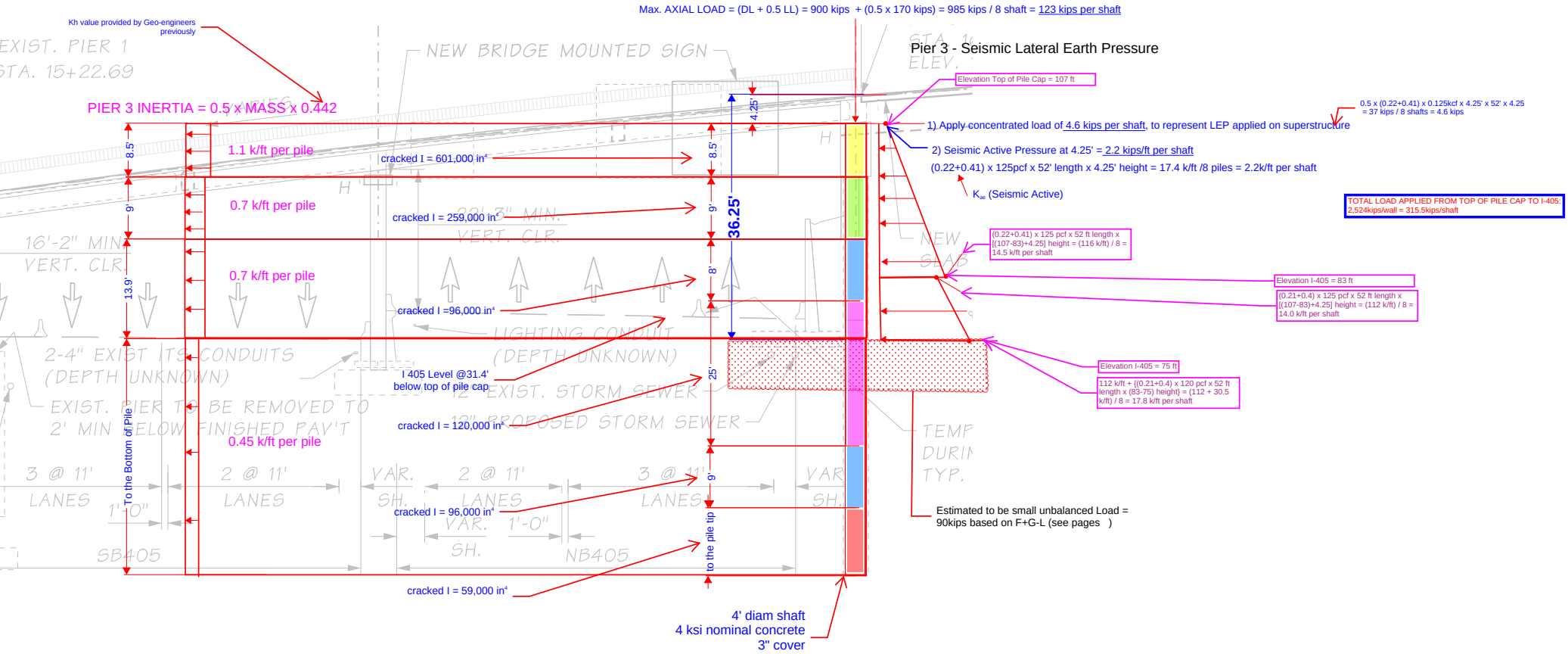
SR 405/13.1

2/20/21

DOT FORM 221-012
Revised 10/83

PIER 3 : Point of Fixity Analysis (Seismic case)

Pier 3 - LEP and Inertial Loads



NOTES FOR PY WALL MODEL :

1. All loads and section properties are given to represent single-shaft section. There are a total 8 shafts.
Hence, all load and section properties values should be multiplied by 8 if the group of pile is modeled for analysis.

2. Appropriate p-Y multipliers need to be applied to account for group effects, unless group of 8 piles are modeled.

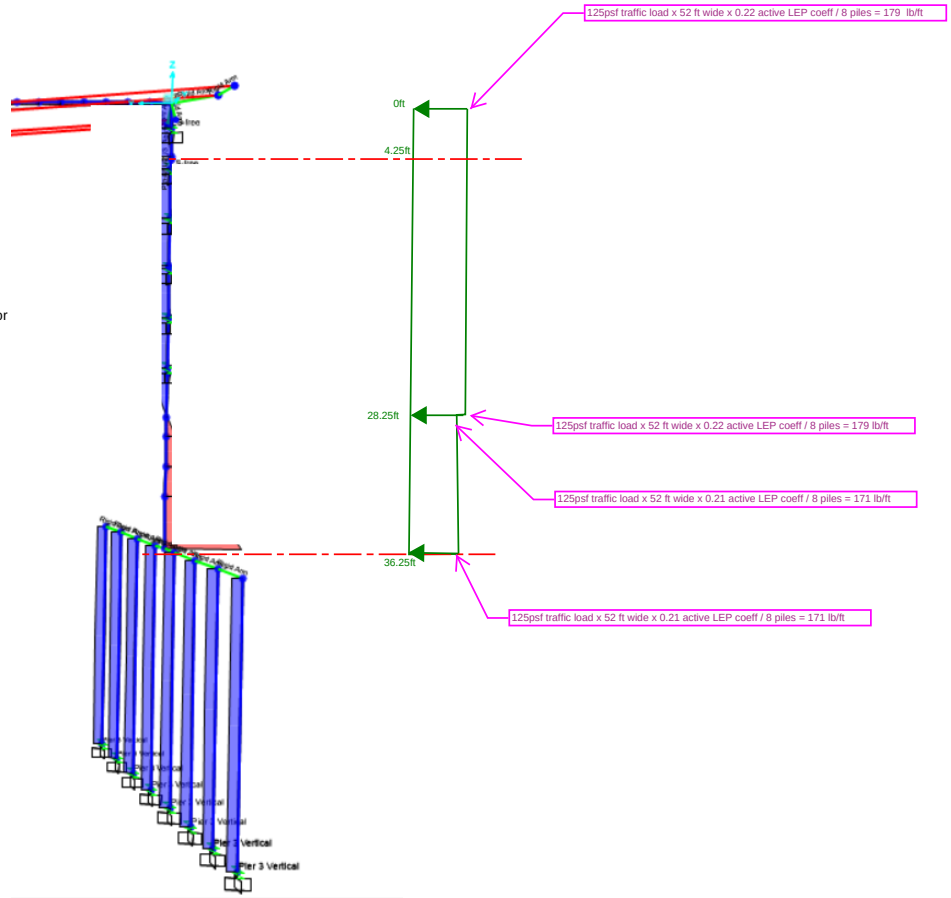
Pier 3 - Live Load Surcharge

The increase in horizontal pressure due to live load surcharge may be estimated as:

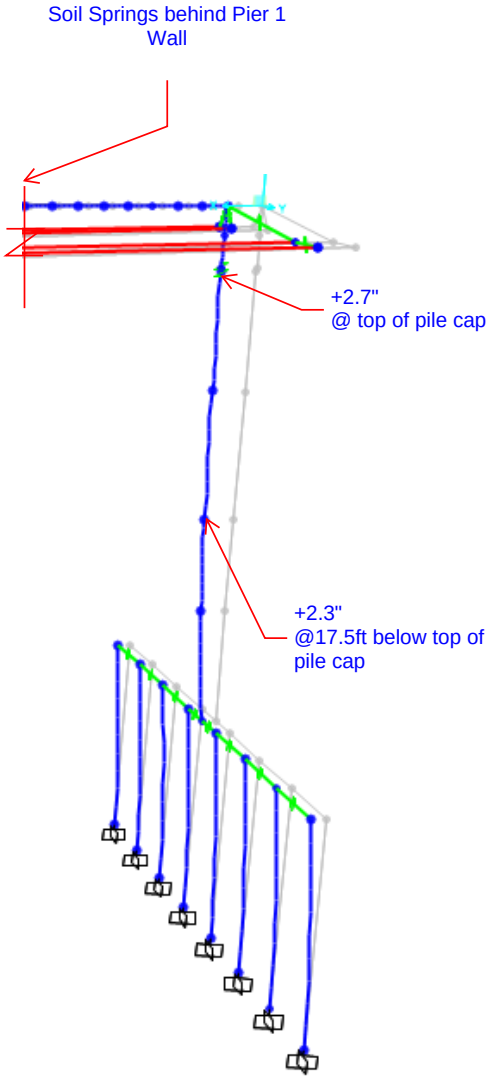
$\Delta_p = k\gamma_s h_{eq}$ (3.11.6.4-1)

where:

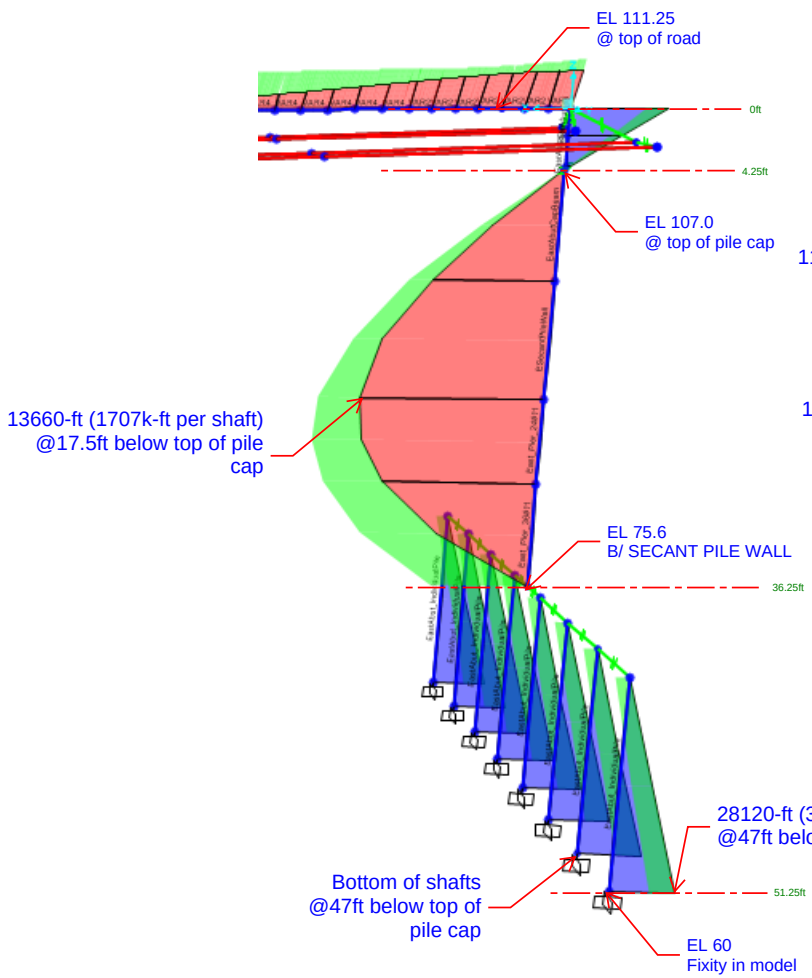
Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft) = 2ft (consider 1ft for EXT case as LS factor =0.5)



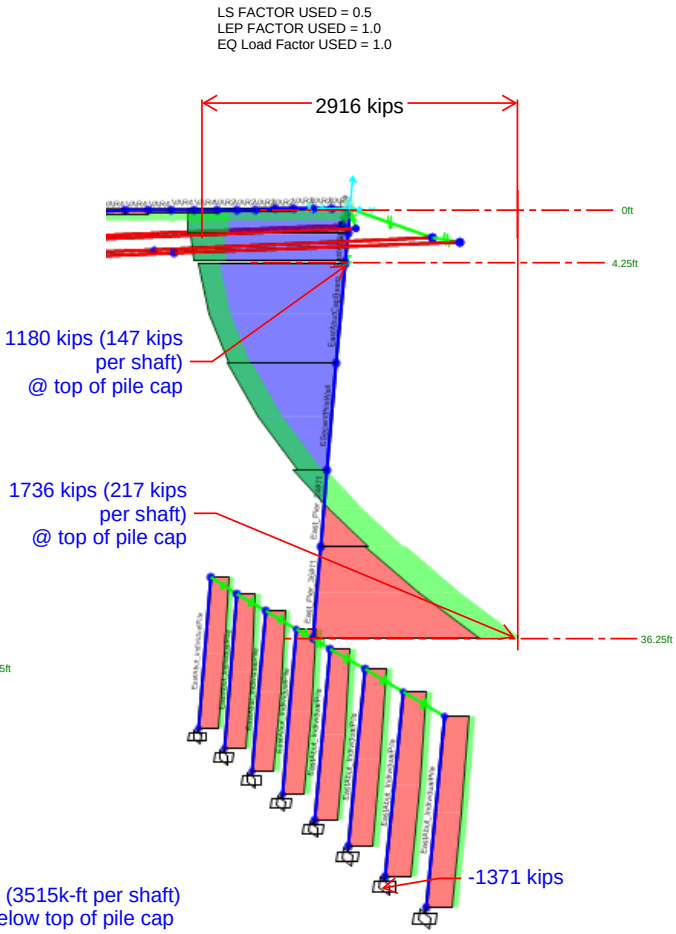
Pier 3 - Seismic Results EtoW (LEP + LS)



Deflection Diagram
(positive towards pier 2)



Moment Diagram
(about the weak axis of pier)



Shear Diagram

PIER 3 - DYNAMIC
SEISMIC ACTIVE + 0.5 LIVE SURCHARGE
11-23-2021
37kips TOTAL LEP LOAD AT TOP OF PILES (4.6 kip/pile) DUE TO LEP ON END DIAPHRAGM
K_seismic APPLIED TO SOIL LAYERS ABOVE I-405, AUTOMATIC CALCULATION OF LEP BELOW I-405
RESULTS ARE PER PILE, ASSUMED PILE SPACING = 6'-8"

Geo-engineers should also add LS on end-diaphragm = 6 kips of total LS load i.e. 0.75 kips per shaft

GEO resistance force at top 150 kips
STR: 158 kips

STR: 147
GEO: 140

~-140 kips per pile
~-1120 kips for wall

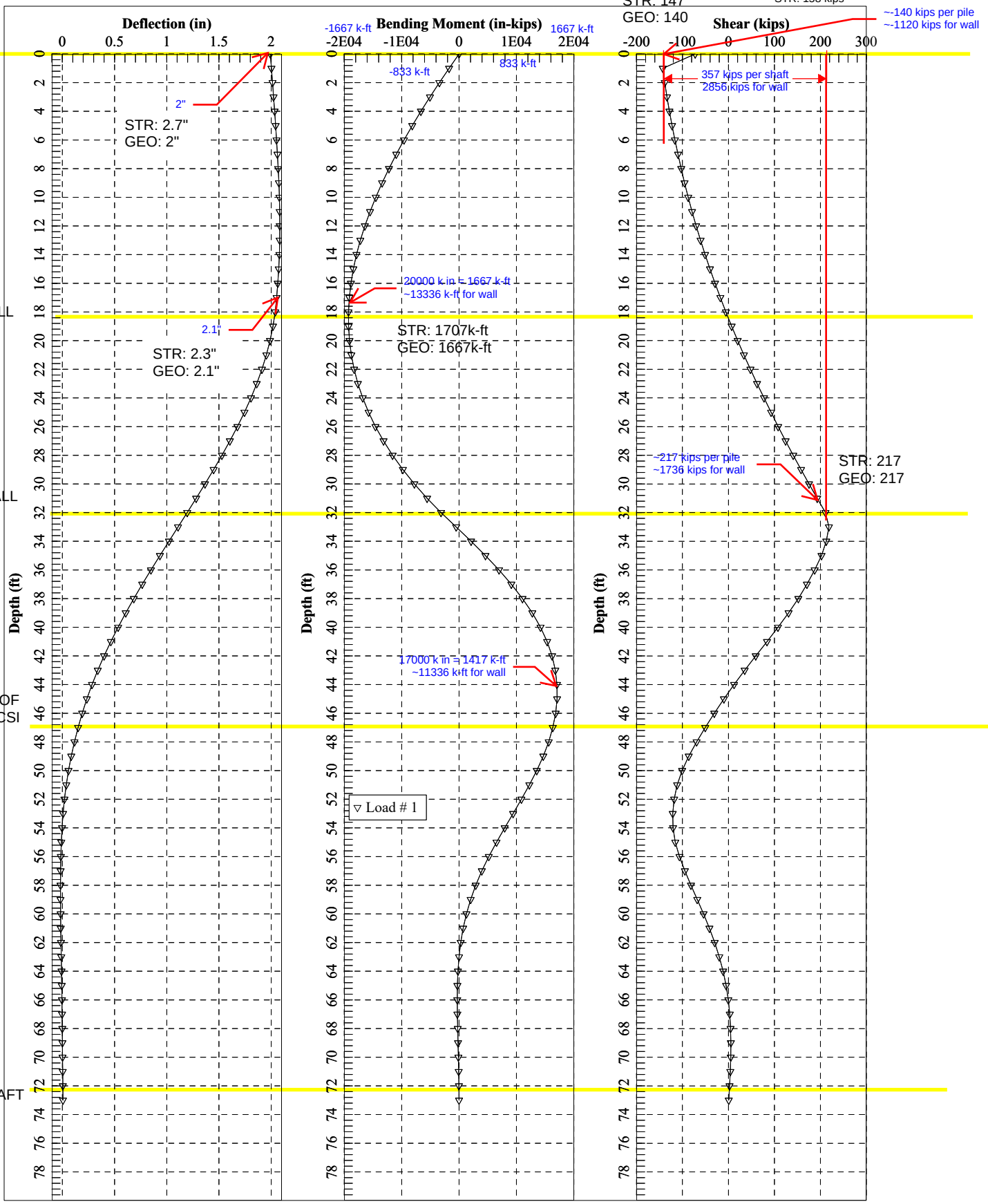
HINGE LINE
EL 107.0

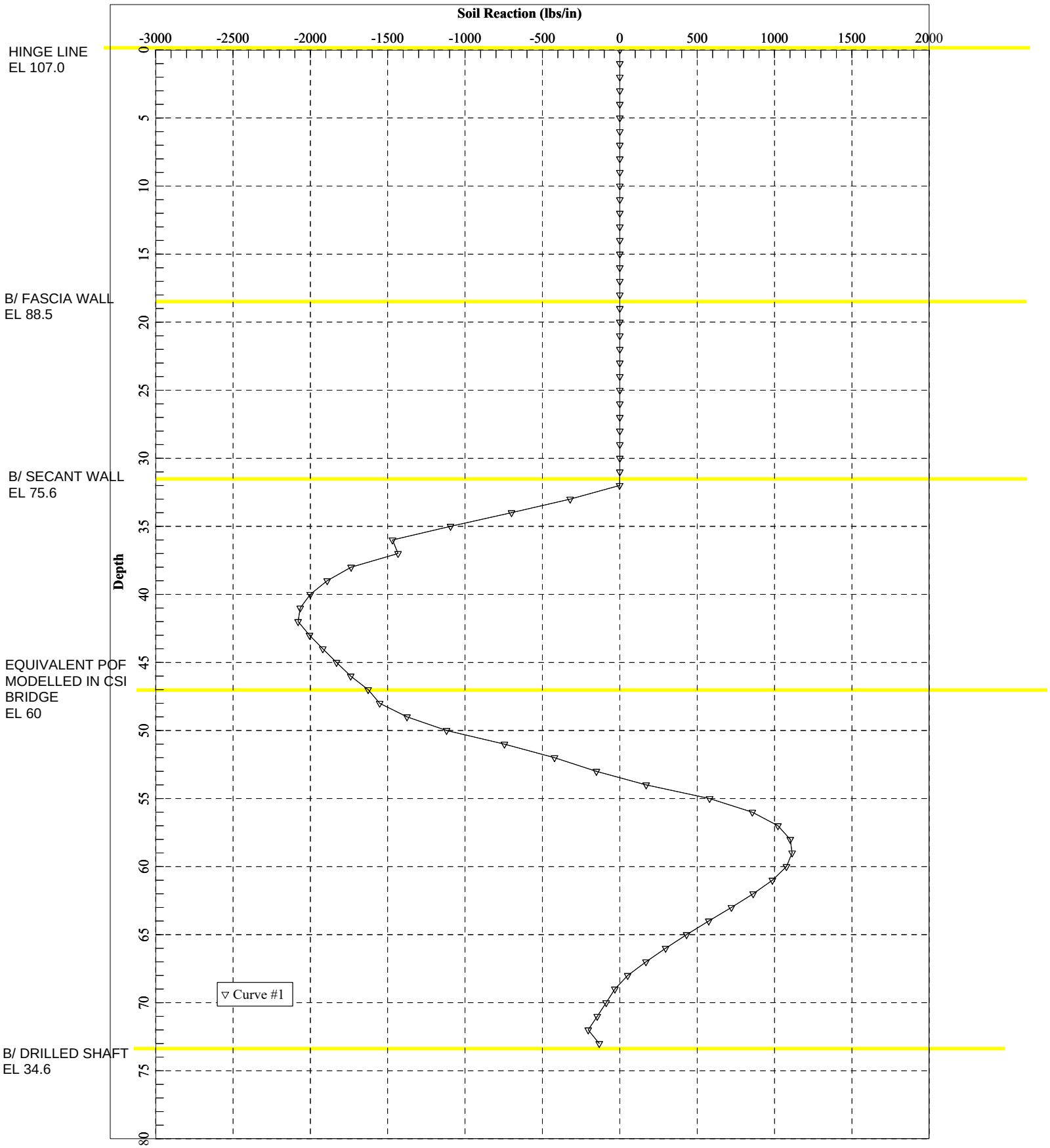
B/ FASCIA WALL
EL 88.5

B/ SECANT WALL
EL 75.6

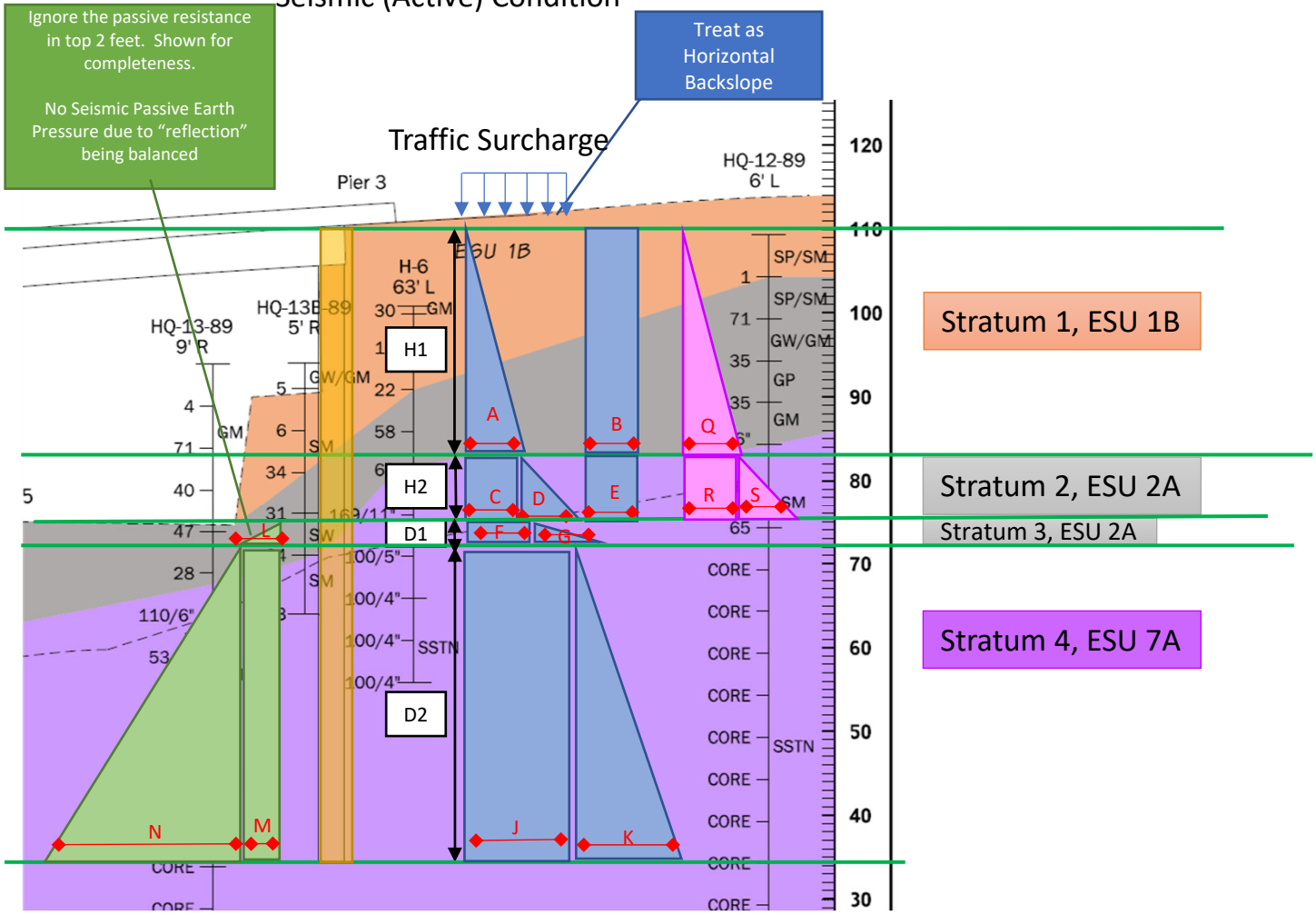
EQUIVALENT POF MODELLED IN CSI BRIDGE
EL 60

B/ DRILLED SHAFT
EL 34.6




Soil Reaction (lbs/in)
-3000
-2500
-2000
-1500
-1000
-500
0
500
1000
1500
2000
Depth
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
HINGE LINE
EL 107.0
B/ FASCIA WALL
EL 88.5
B/ SECANT WALL
EL 75.6
EQUIVALENT POF
MODELLED IN CSI
BRIDGE
EL 60
B/ DRILLED SHAFT
EL 34.6
▽ Curve #1

Bridge 17.7
Pier 3 Lateral Earth Pressures
Seismic (Active) Condition



NOT TO SCALE

To Calculate LEP for each stratum, sum the areas indicated.
For Example, Stratum 2, C+D+E+R+S = LEP

Seismic (Active) Lateral Earth Pressures

Ground Surface Elevation at Top of Wall	110	ft	H1	27	ft
Ground Surface Elevation at NB 405	74	ft	H2	9	ft
Wall Toe Elevation	35	ft	D1	2	ft
Groundwater Level Elevation	72	ft	D2	37	ft
Traffic Surcharge, Static	250	psf			
Traffic Surcharge, Seismic	125	psf			

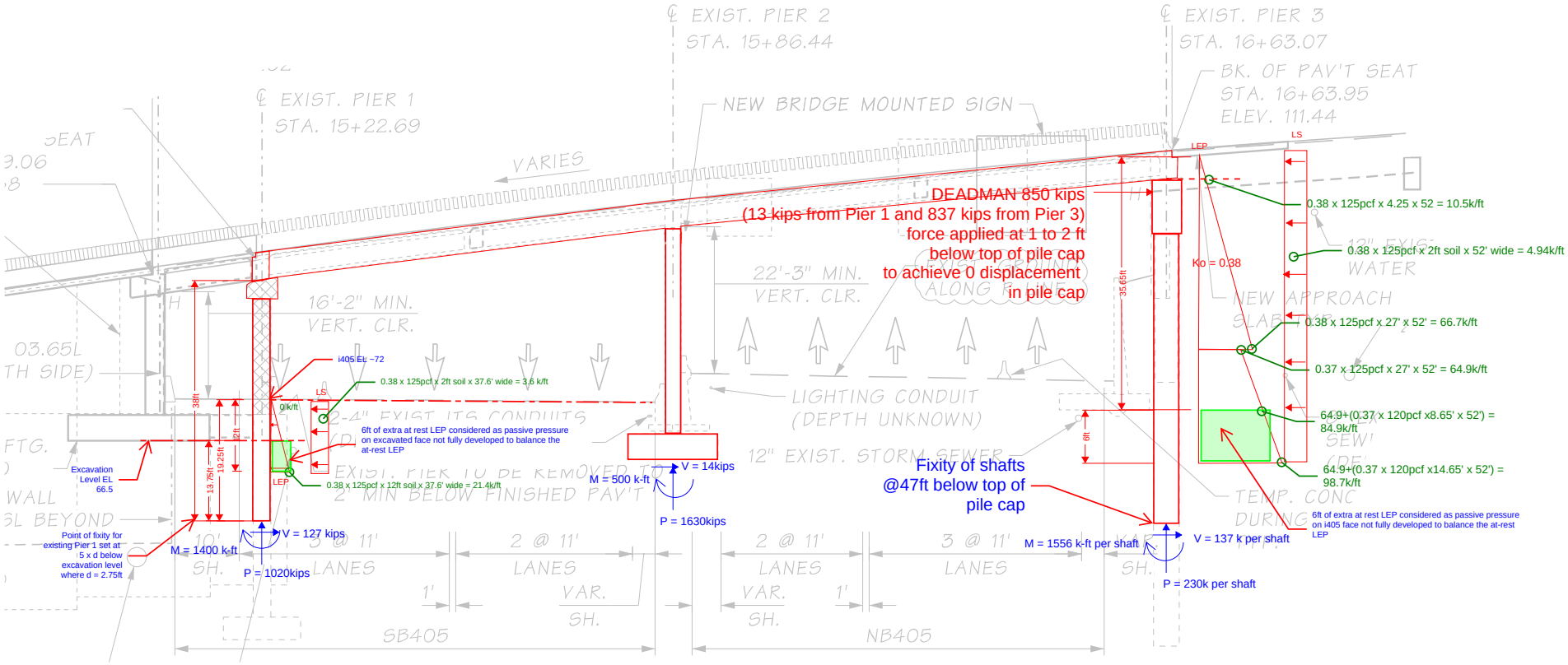
Interval	Layer Description	Elevation, Top of Layer ft	Elevation, Bottom of Layer ft	Friction Angle deg	Soil Total Unit Weight pcf	Soil Effective Unit Weight pcf	Active Lateral Earth Pressure Coefficient	At Rest Lateral Earth Pressure Coefficient	Passive Lateral Earth Pressure Coefficient	Seismic Lateral Earth Pressure Coefficient
Stratum 1	ESU 1B, above water table	110	83	38	125	125	0.22	0.38	4.20	0.41
Stratum 2	ESU 2A, above water table	83	74	39	120	120	0.21	0.37	4.40	0.40
Stratum 3	ESU 2A, above water table	74	72	39	120	120	0.18	0.37	4.40	0.40
Stratum 4	ESU 7A, below water table	72	35	42	135	72.6	0.28	0.33	5.04	0.38

	Seismic (active) (psf)								
	A	B	C	D	E	F	G	J	K
Stratum	1	1	2	2	2	3	3	4	4
Description	earth pressure, H1	traffic surcharge	earth pressure, H1	earth pressure, H2	traffic surcharge	earth pressure, H1 and H2	earth pressure, D1	earth pressure, H1, H2, D1	earth pressure, D2
Recommended Earth Pressures	732	27	703	225	26	820	89	1291	739
To calculate	27.1 * H1	27	26.02 * H1	24.98 * H2	26	23 * H1 + 22.08 * H2	22.08 * D1	34.38 * H1 + 33 * H2 + 33 * D1	19.97 * D2

	Passive (psf)			Seismic (psf)		
	L	M	N	Q	R	S
Stratum	3	4	4	1	2	2
Description	earth pressure, D1	earth pressure, D1	earth pressure, D2	seismic earth pressure, H1	seismic earth pressure, H1	seismic earth pressure, H2
Recommended Earth Pressures	0	1211	13551	1399	1364	436
To calculate	527.46 * (D1 - 2 ft)	605.36 * D1	366.24 * D2	51.82 * H1	50.52 * H1	48.5 * H2

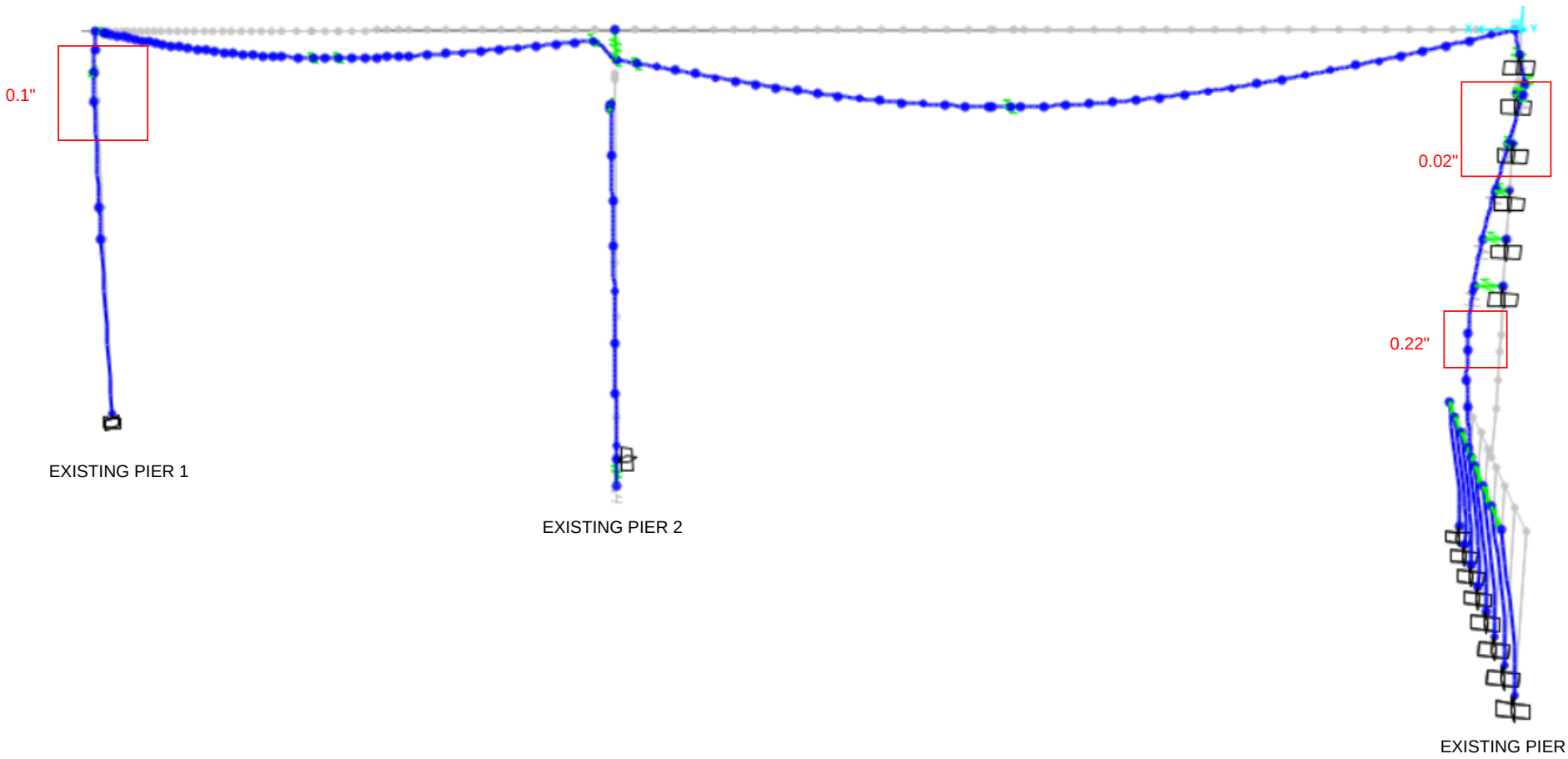
PIER 3 : Construction Stage Analysis : Max Force estimate to restraint Pier 3 top movement

LOADING

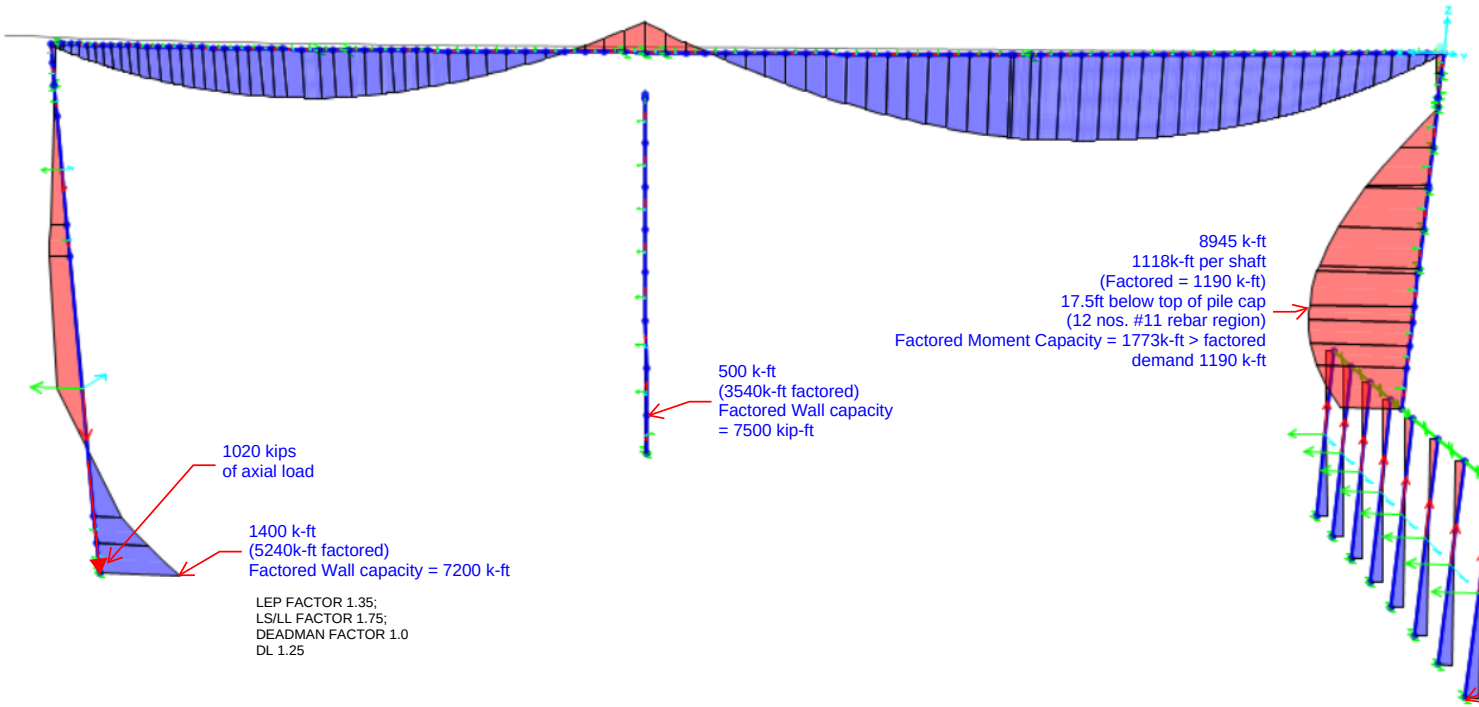


1. All loads in loading diagrams and reactions shown here are un-factored.
2. For structural design, use 1.35 factor for LEP and 1.75 for LS and 1.0 for deadman force
3. 10psf of construction live load applied on bridge.
4. Cracked moment of inertia modifier of 0.5 used for all piers.
5. Footing springs used at Pier 2 as provided by Geo-Engineers.
6. Pier 3 bottom is fixed at point of fixity 47ft below top of pile cap.
7. Pier 1 bottom is fixed at point of fixity 38ft below top of pile cap.

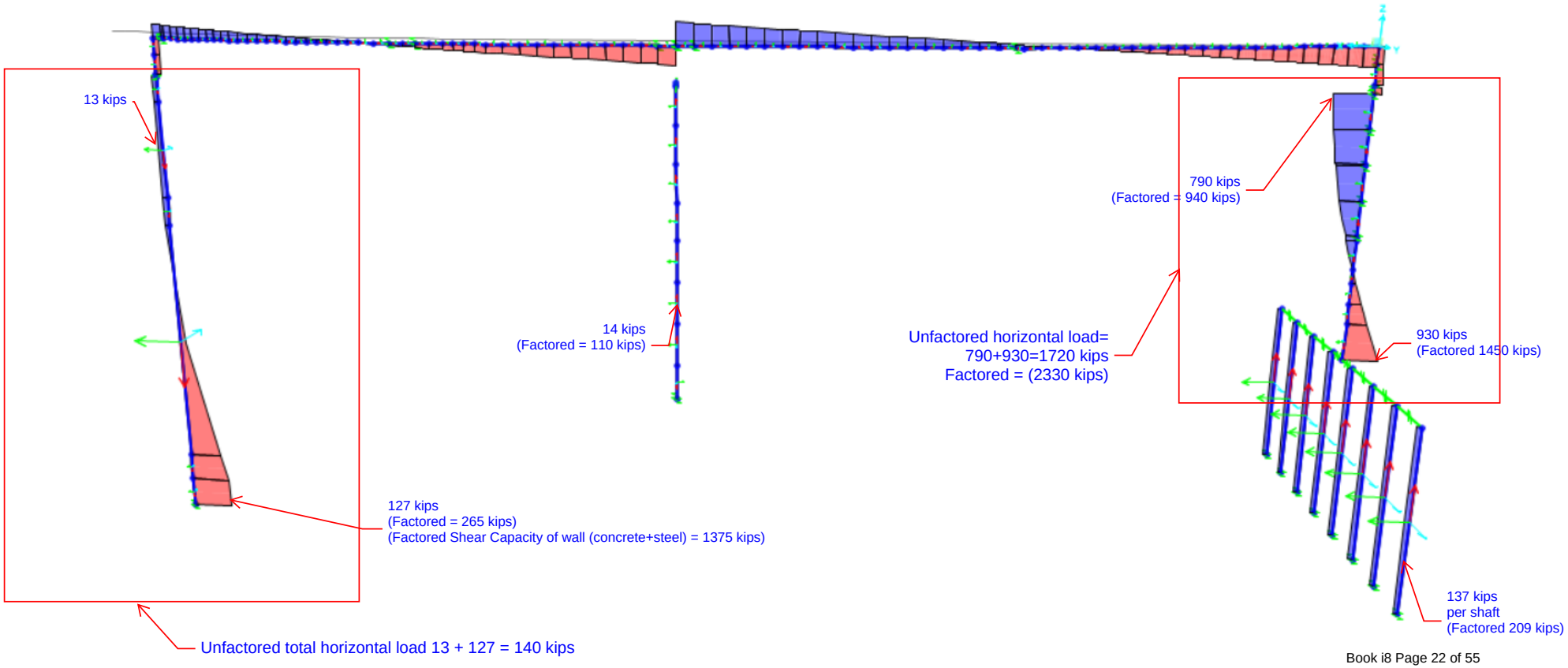
DEFLECTION



MOMENT DIAGRAM
(Unfactored plot shown)

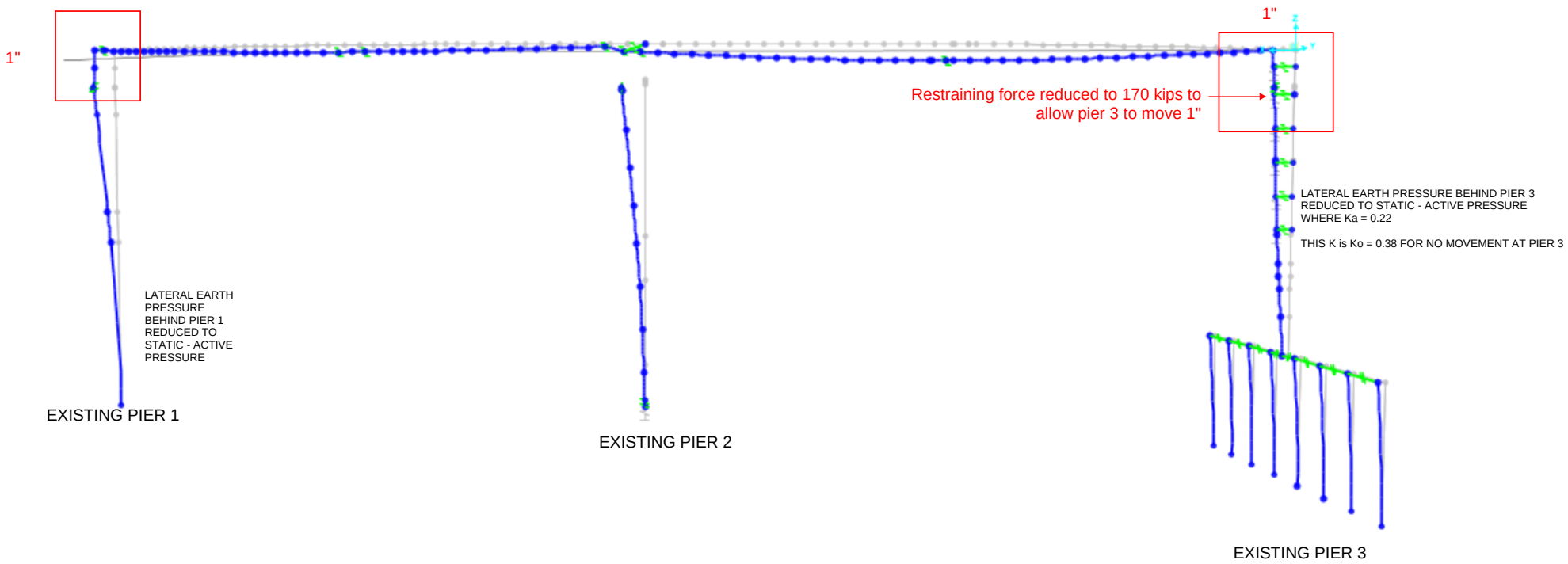


SHEAR DIAGRAM
(Unfactored plot shown)

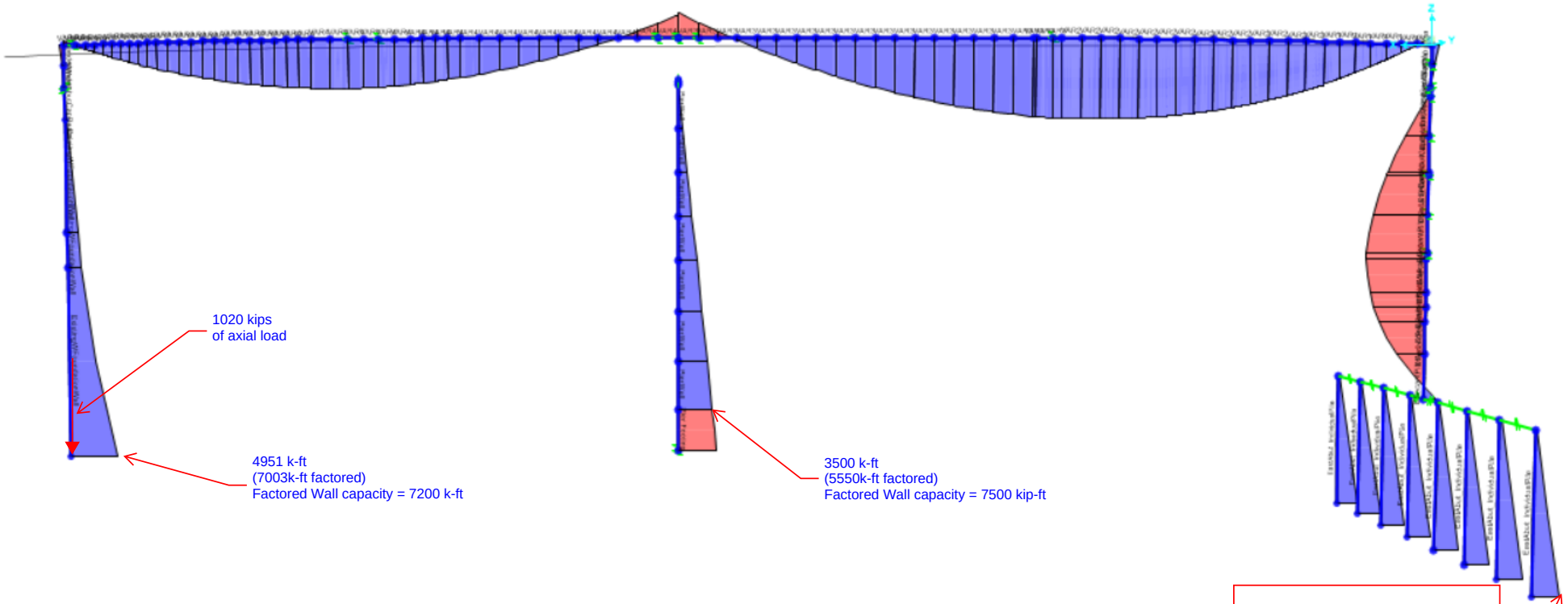


PIER 3 : Construction Stage Analysis : Locked-in stress at point of fixity for 1" movement at pier 3 top

DEFLECTION

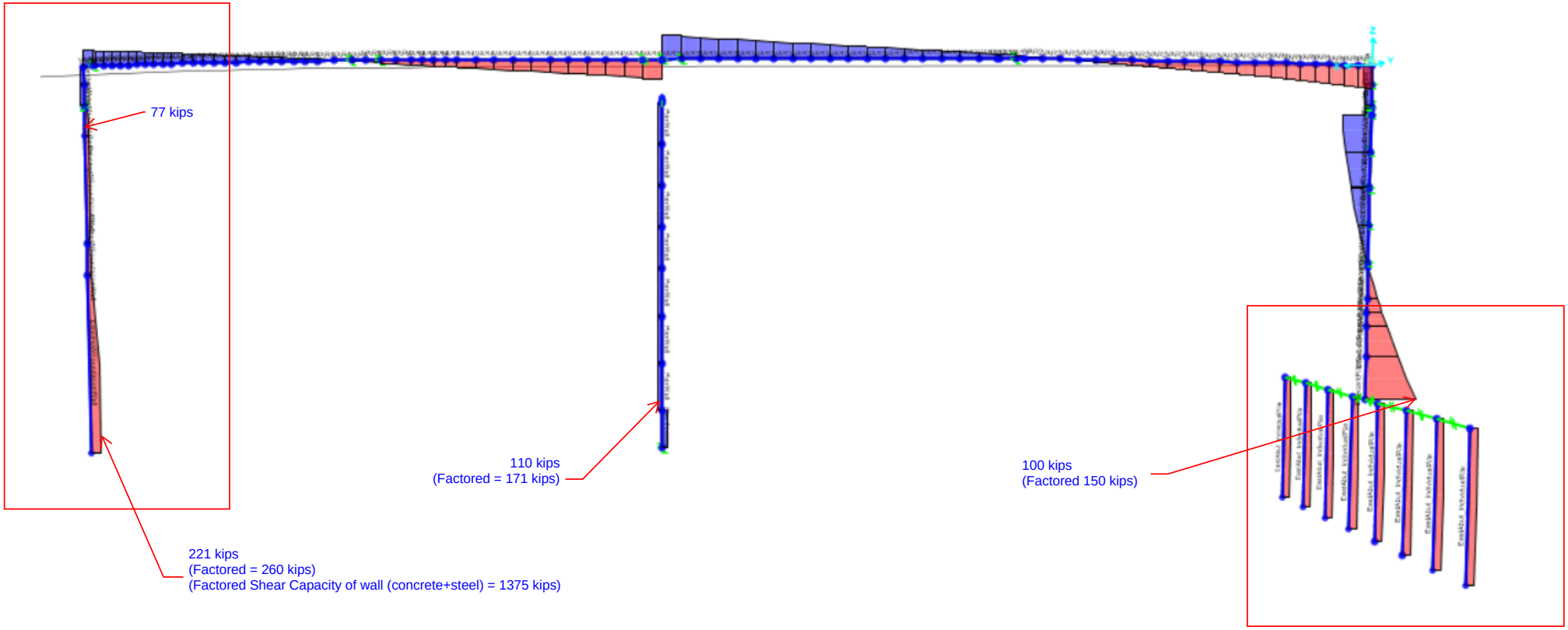

1"
1"
Restraining force reduced to 170 kips to allow pier 3 to move 1"
LATERAL EARTH PRESSURE BEHIND PIER 3 REDUCED TO STATIC - ACTIVE PRESSURE WHERE Ka = 0.22
THIS K is Ko = 0.38 FOR NO MOVEMENT AT PIER 3
LATERAL EARTH PRESSURE BEHIND PIER 1 REDUCED TO STATIC - ACTIVE PRESSURE
EXISTING PIER 1
EXISTING PIER 2
EXISTING PIER 3

MOMENT DIAGRAM
(Unfactored plot shown)

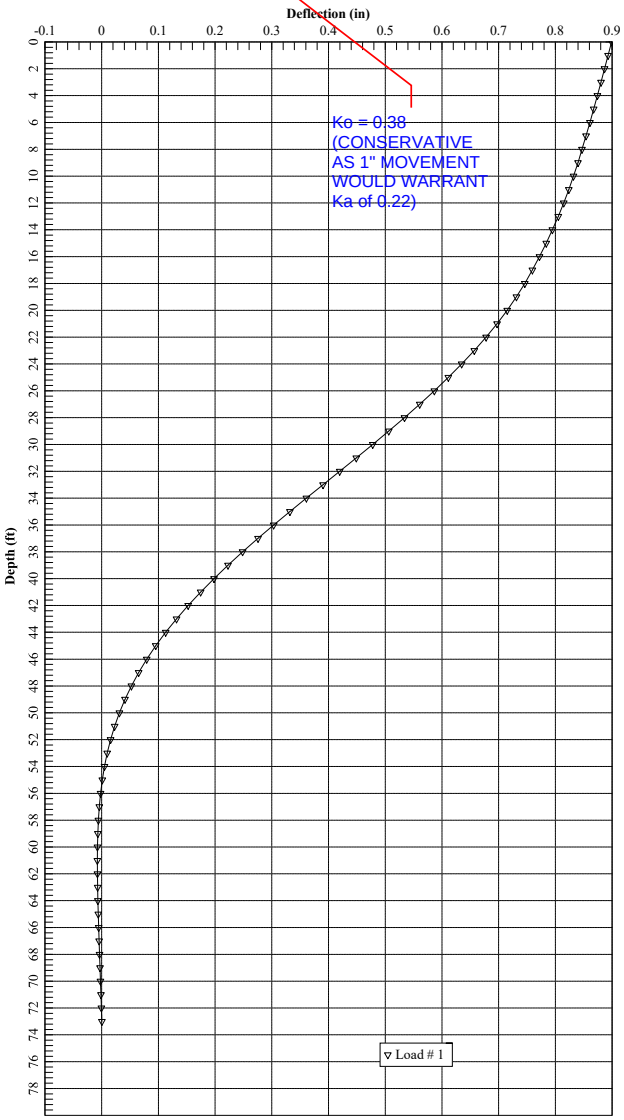


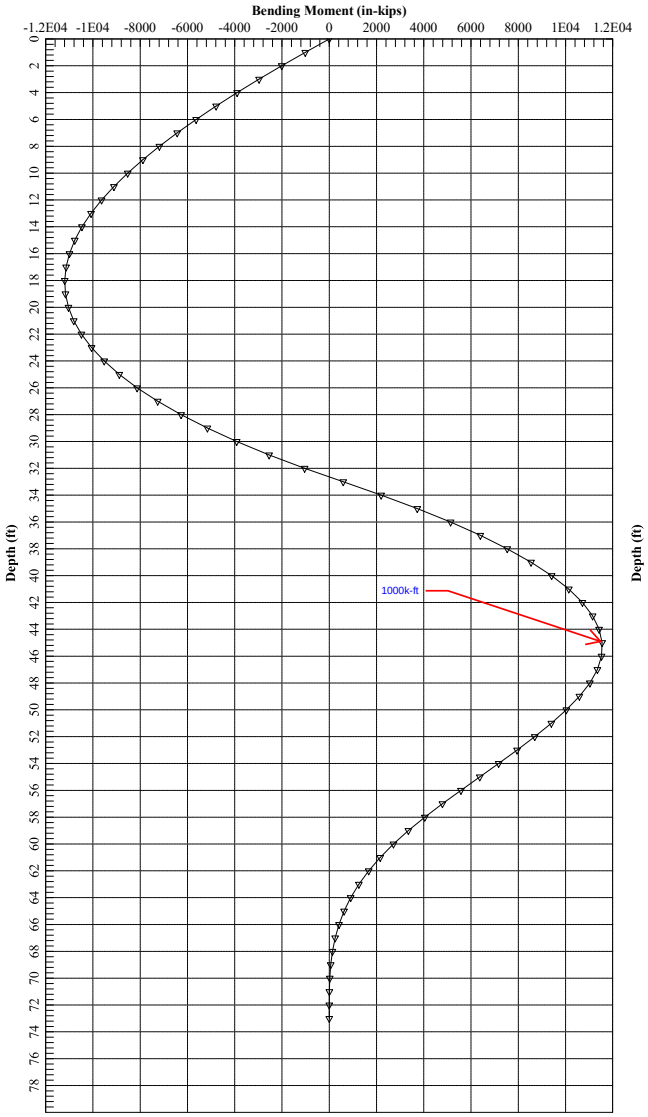
NOTE : THIS LOCKED IN VALUE IS SMALLER USING PY WALL MODEL. SEE PAGE 27

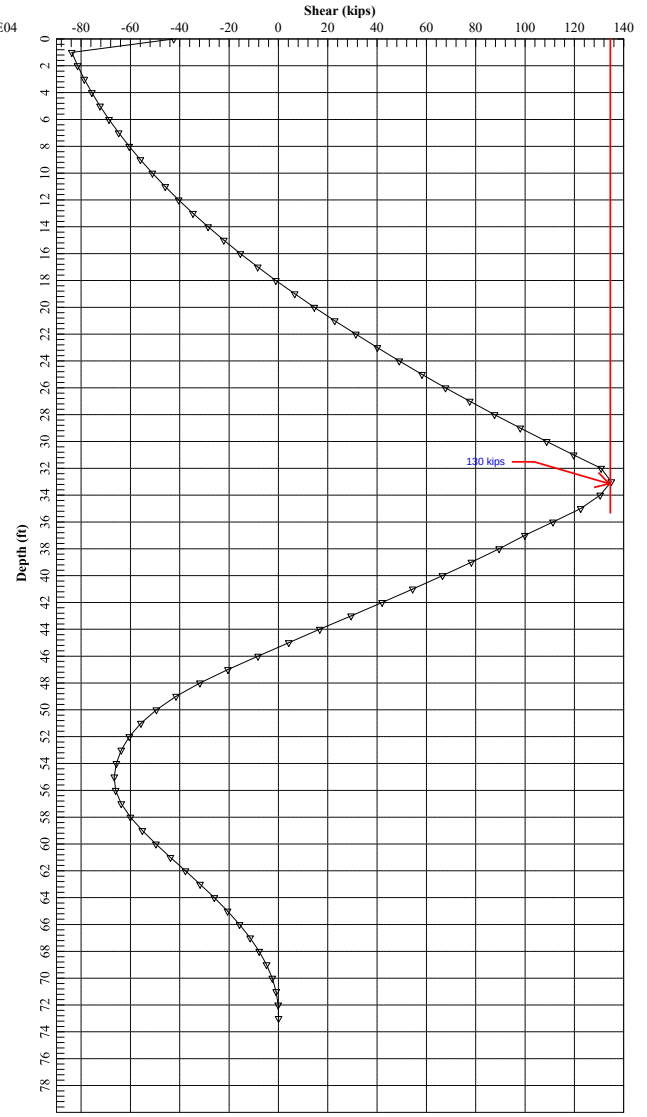
SHEAR DIAGRAM
(Unfactored plot shown)



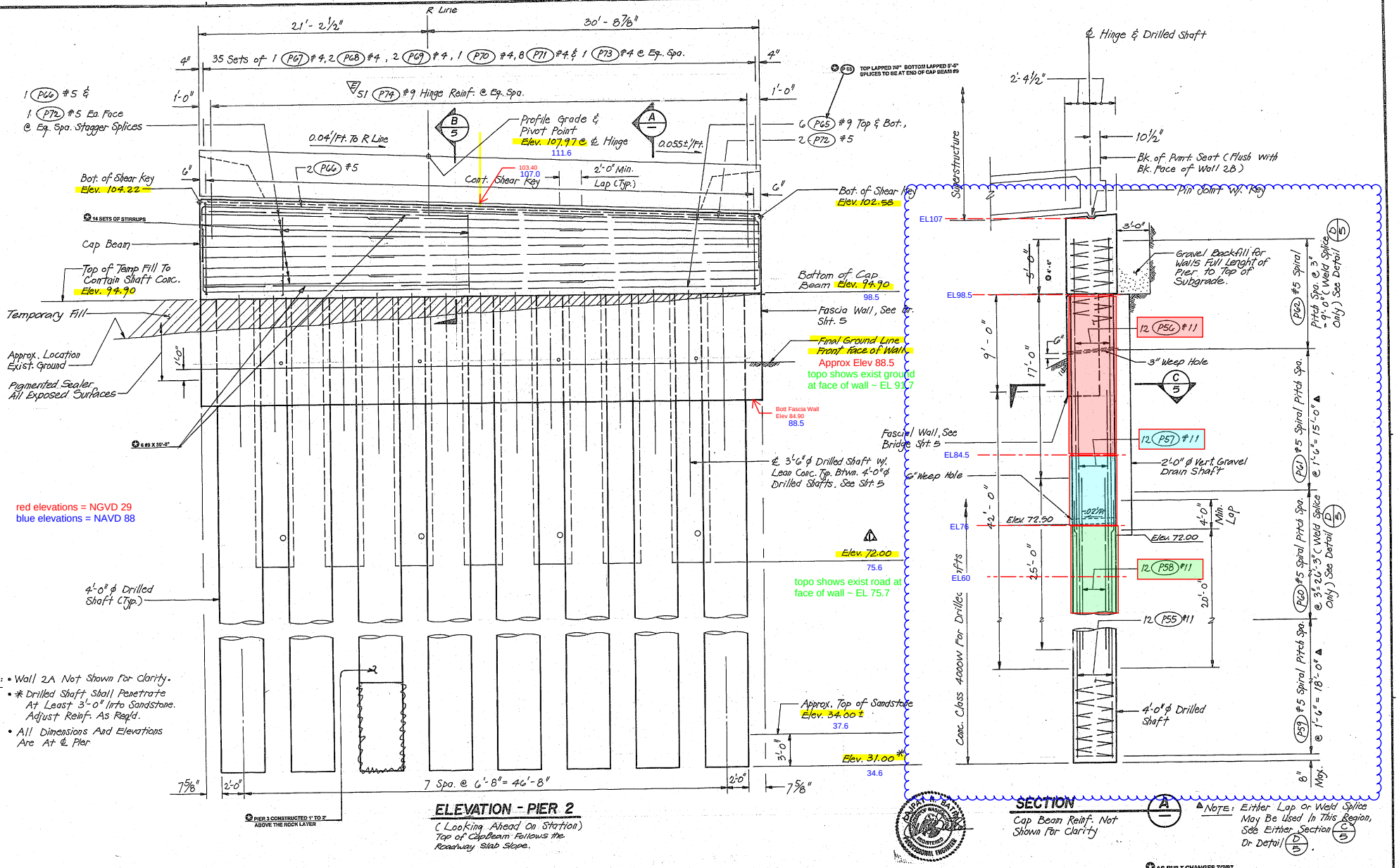
Restraining lateral load of 88 kips per shaft, acting at Elevation 107 (Depth = 0ft), Bottom of Shear Key
Deflection < 1 inch towards I-405
At-Rest Lateral Earth Pressures used







PIER 3 : Reinforcement arrangement along the shaft



DRAWN	P. Lee				REGION NO.	STATE	FED. AID PROJ. NO.	SHEET NO.	TOTAL SHEETS
DESIGN	J. King				10	WASH	ACI-405-3(780)		
CHECK	K. Pirbazari				JOB NUMBER				
PROJ. ENGR.					90W069				
DIST. ADM.		97-90	1	Revised Elev.	CONTRACT NO.				
		DATE		REVISION	BY	APP'D	3854		

APPROVED

Washington State Department of Transportation

INCA ENGINEERS INC.

SR 405
TUKWILA TO SUNSET BLVD
CEDAR RIVER PIPELINES-RELOCATION
RENTON AVENUE U'XING 405/17B

PIER 2

SHEET 4 OF 271 SHEETS

Book 18 Page 29 of 55

SR 405/131

2/20/21

DOT FORM 221-012
Revised 10/83

PIER 3 : REINFORCEMENT DESIGN CHECK (EXT CASE)

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Reinforcement (EXT)
BY: RSGR
CHK: MWBM

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Purpose:
Check the flexure and shear capacity for the Pier 3 shafts for EXT cases
Notes:
1. Shafts are not expected to develop a plastic hinge during EXT case.
2. Unfactored demands for lateral earth pressure and live load surcharge behind Pier 3 are taken as the highest value between geotech's PY-Model and bridge FEM model above the i405 level.
3. Unfactored demands for lateral earth pressure and live load surcharge behind Pier 3 are from geotech's PY-Model below i405 level.
4. Shaft have varying flexural and shear capacity along its length due varying reinforcement layout.
5. Design checks are carried out on 'per-shaft' basis.
6. Locked-in stress due to construction stage is added to the shear and moment demands to point of fixity. Locked-in stress are estimated by letting Pier 3 move 1" towards the West during construction stage. This load includes LEP with a Ka factor of 0.22. Note that LEP demands from SEE EQ case already include a K factor of 0.69 out of which 0.22 is from Ka. Hence the locked in stresses are highly conservative.
7. The shafts can resist the demands from lengthened bridge without retrofitting the pier.

Shaft Reinforcment along the length

	height below top of pile cap		EL NAV88	height below top of pile cap	EL NAV88	
Assuming 0ft at top of pile cap	0	ft	107	0	107	
Bottom of pile cap	8.5	ft below top of pile cap	98.5	8.5	98.5	
end of 12 nos. #11 section (inc. splice length)	22.5	ft below top of pile cap	84.5	13	94	#5 spiral 3"SPA from EL 98.5 to 94
end of 24 nos. #11 section (inc. splice length)	31	ft below top of pile cap	76	28	79	#5 spiral 18"SPA from EL 94 to 79
end of 36 nos. #11 section (inc. splice length)	47	ft below top of pile cap	60	47	60	#5 spiral 3"SPA from EL 79 to EL60

Note that the Shaft is only modelled upto 47ft below top of pile cap (EL 56.5) for STR and EXT case in bridge FEM models
i405 level is at ~EL74 (or 33ft below top of pile cap)

DIMENSIONS & PARAMETERS

f'c	4	ksi	concrete compressive strength
wc =	0.155	kcf	Unit weight of weight concrete
K1 =	1		Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4028	ksi	Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
fy =	60	ksi	Yield strength of reinforcement
Es =	29000	ksi	Modulus of elasticity of reinforcement
η =	7.20		Modular ratio
β_1 =	0.85		Compression Zone Factor
φf=	1		flexural resistance factor for EXT case
φv=	0.9		shear resistance factor

LOADING IN PIER 3 Wall (Per Shaft) - Lateral Earth Pressure due to SEE event

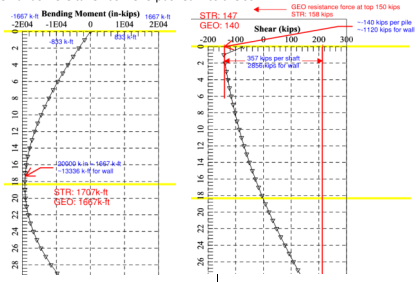
LEP (Seismic)+Live Load Surcharge and Inertial demands from elastic dynamic analysis compared with analysis from PY-Wall model

Taken from Geo-engineers PY-Wall model
These are only applicable to EtoW earthquake, For WtoE earthquake, bridge FEM model demands were taken as Pier 3 pushed into the soil

ABOVE i405 Level :

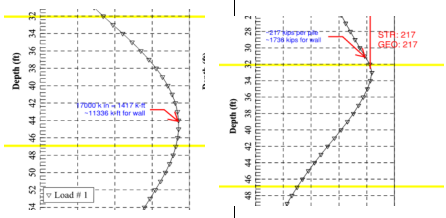
	V2	M3	
	Kip	Kip-ft	
EL 98.5 to EL 89.5	147	1707	Max top 20ft

These are combined with other load effects at pier 3



AT OR BELOW i405 Level :

EL 89.5 to EL 81.5	100	1707	Max 20 to 30ft below top of pile cap
EL 81.5 to EL 56.5	217	-1417	Max 30 ft to 44ft below top of pile cap



EL : Locked in Construction Stress at the point of fixity

These are from the case when the Pier 3 is allowed to move 1" to the west
These locked in demands include LEP with Ko of 0.38 which is then combined with EQ demands that already has LEP with Ka of 0.22 + Kae 0.47
Hence, this demand is more than double counting the LEP active pressure effect of Ka 0.22 which is conservative

V2	M3
Kips	Kip-ft
130	-1000

These demands are added to 36-#11 sections

COWI

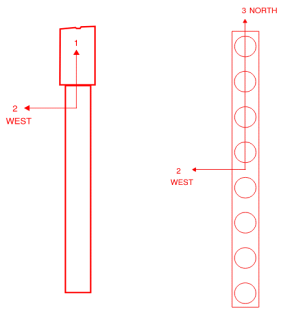
PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Reinforcement (EXT)
BY: RSGR
CHK: MWBM

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

LOADING IN PIER 3 WALL (Per Shaft) (8.5ft to 17.5ft below top pile cap)

			P	V2*	V3*	T*	M2	M3	
			Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
DC, Supr+pile cap+self weight of shaft			-133	1	1	0	0	-26	
		CR&SH	-9	1	0	0	0	24	
		PS	-2	2	0	1	0	26	
	Max	LL	-21	5	4	1	0	-91	IM included; BR and CE included
	Min	LL	2	5	4	1	0	83	IM included; BR and CE included
100%L+30 %T-30%Z	EtoW	LEP + 0.5 LS + EQ	27	147	5	16	278	1707	LEP and LS here are behind pier 3 for EtoW EQ
30%L+100 %T-30%Z	EtoW	0.3 LEP + 0.5 LS + EQ	27	44	13	51	628	512	
100%L+30 %T-30%Z	WtoE	0.5 LS	0	0	0	0	-1	9	LEP and LS here are behind pier 1 for WtoE EQ
		LEP	-1	5	0	2	-14	120	
		EQ	57	72	7	33	-498	1006	
30%L+100 %T-30%Z	WtoE	0.5 LS	0	0	0	0	-1	9	
		0.3 LEP	0	2	0	1	-4	36	
		EQ	37	46	16	43	-600	693	

Note:
* Absolute max absolute values of shear and torsion force selected for design check



LOADING ON THE PIER 3 WALL (per shaft) (17.5ft to 25.5ft below top pile cap)

			P	V2*	V3*	T*	M2	M3	
			Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
DC, Supr+pile cap+self weight of shaft			-164	1	1	0	0	41	
		CR&SH	-9	1	0	0	0	-38	
		PS	-2	2	0	1	0	-41	
	Max	LL	-21	5	4	1	0	-143	IM included; BR and CE included
	Min	LL	2	5	4	1	0	130	IM included; BR and CE included
100%L+30 %T-30%Z	EtoW	LEP + 0.5 LS + EQ	27	100	4	16	332	1707	LEP and LS here are behind pier 3 for EtoW EQ
30%L+100 %T-30%Z	EtoW	0.3 LEP + 0.5 LS + EQ	27	30	9	51	636	512	
100%L+30 %T-30%Z	WtoE	0.5 LS	0	1	0	0	-1	4	LEP and LS here are behind pier 1 for WtoE EQ
		LEP	-1	12	0	2	-13	56	
		EQ	57	114	5	-33	-482	516	
30%L+100 %T-30%Z	WtoE	0.5 LS	0	1	0	0	-1	4	
		0.3 LEP	0	4	0	1	-4	17	
		EQ	38	72	11	-43	-611	326	

Note:
* Absolute max absolute values of shear and torsion force selected for design check

LOADING ON THE PIER 3 WALL (per shaft) (25.5ft to 50.5ft below top pile cap)

			P	V2*	V3*	T*	M2	M3	
			Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
DC, Supr+pile cap+self weight of shaft			-213	2	1	0	0	-67	
		CR&SH	-11	2	0	0	0	64	
		PS	-5	3	0	1	0	73	
	Max	LL	-21	7	1	0	0	309	IM included; BR and CE included
	Min	LL	2	5	0	0	0	-222	IM included; BR and CE included
		EL	0	130	0	0	0	-1000	
100%L+30 %T-30%Z	EtoW	LEP + 0.5 LS + EQ	31	217	2	1	-23	-1417	LEP and LS here are behind pier 3 for EtoW EQ
30%L+100 %T-30%Z	EtoW	0.3 LEP + 0.5 LS + EQ	58	65	6	2	-47	-425	
100%L+30 %T-30%Z	WtoE	0.5 LS	0	0	0	0	0	-2	LEP and LS here are behind pier 1 for WtoE EQ
		LEP	0	3	0	0	1	-30	
		EQ	51	29	4	1	30	-292	
30%L+100 %T-30%Z	WtoE	0.5 LS	0	0	0	0	0	-2	
		0.3 LEP	0	1	0	0	0	-9	
		EQ	78	20	6	2	45	-194	

* Absolute max absolute values of shear and torsion force selected for design check

	DC	LL	CR SH PS	EL	EQ	LEP	LS	
EtoW	1	0.5	1	1	1.2	1	0.5	100%L + 30%T - 30%Z
EtoW	1	0.5	1	1	1.2	1	0.5	30%L + 100%T - 30%Z
WtoE	1	0.5	1	1	1.2	1	0.5	100%L + 30%T - 30%Z
WtoE	1	0.5	1	1	1.2	1	0.5	30%L + 100%T - 30%Z

12nos. #11; (EL 98.5 to EL 84.5); #5 Spiral at ~3" (EL 98.5 to EL 94)

								Combined SRSS	
		P	V2*	V3*	T*	M2	M3	V	M
Case		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	kips	Kip-ft
1	EtoW	-110	183	9	21	334	2114	183	2140
2		-110	60	18	63	754	680	62	1015
3	WtoE	-76	98	11	43	-612	1311	99	1446
4		-99	63	22	54	-725	938	67	1185

100%L + 30%T - 30%Z	
30%L + 100%T - 30%Z	
100%L + 30%T - 30%Z	
30%L + 100%T - 30%Z	

24nos. #11; (EL 84.5 to EL 76); #5 Spiral at ~3" (EL 94 to EL 79)

								Combined SRSS	
		P	V2*	V3*	T*	M2	M3	V	M
Case		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	kips	Kip-ft
1	EtoW	-141	127	8	21	398	2076	127	2114
2		-141	43	14	63	763	642	45	997
3	WtoE	-106	156	9	-36	-592	705	157	920
4		-128	98	16	-50	-738	437	99	858

100%L + 30%T - 30%Z	
30%L + 100%T - 30%Z	
100%L + 30%T - 30%Z	
30%L + 100%T - 30%Z	

36nos. #11; (EL 76 to EL 60); #5 Spiral at ~3" (EL 79 to EL 60)

								Combined SRSS	
		P	V2*	V3*	T*	M2	M3	V	M
Case		Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	kips	Kip-ft
1	EtoW	-191	400	3	2	-28	-2741	400	2742
2		-158	218	8	3	-56	-1551	218	1552
3	WtoE	-167	177	6	2	37	-1422	177	1423
4		-134	164	8	3	54	-1284	165	1285

100%L + 30%T - 30%Z	
30%L + 100%T - 30%Z	
100%L + 30%T - 30%Z	
30%L + 100%T - 30%Z	

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Reinforcement (EXT)
BY: RSGR
CHK: MWBM

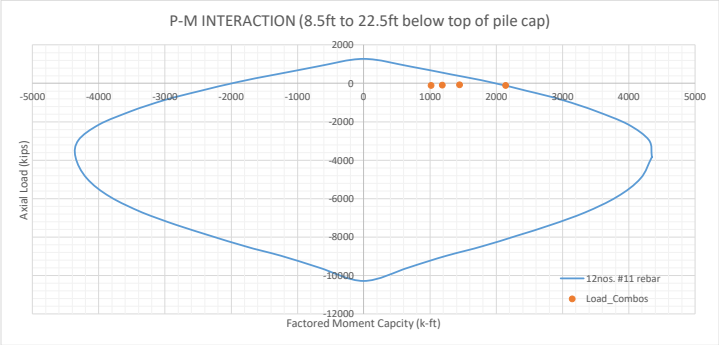
JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Axial - Flexure Design Check

AXIAL P - MOMENT M INTERACTION CURVE FOR PIER 3 SHAFTS

Height: 8.5ft to 22.5ft below pile cap
Rebar: 12 nos. #11 section

fye = 68ksi
f'ce = 5.2ksi phi = 1.0

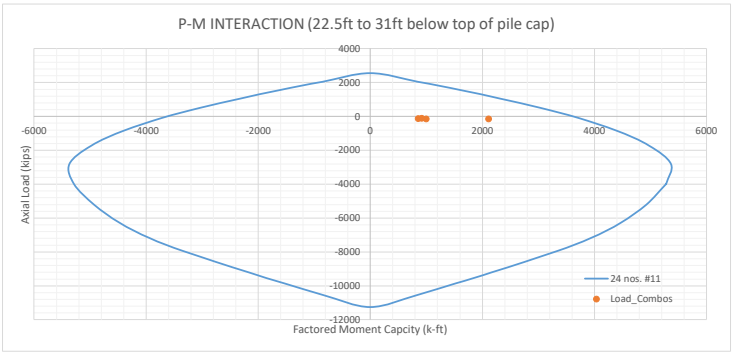


All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 1809 = 543 kips

	Case	M Capacity	P	M demand	D/C
EtoW	1	2120	-110	2140	101%
	2	2120	-110	1015	48%
WtoE	3	2079	-76	1446	70%
	4	2106	-99	1185	56%

22.5ft to 31ft below pile cap
24 nos. #11 section

fye = 68ksi
f'ce = 5.2ksi phi = 1.0

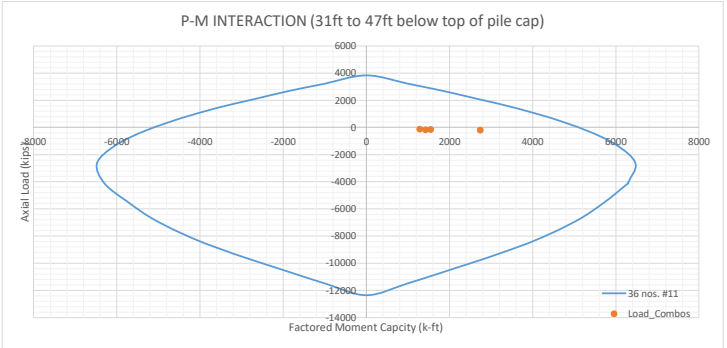


All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 1809 = 543 kips

	Case	M Capacity	P	M demand	D/C
EtoW	1	3753	-141	2114	56%
	2	3753	-141	997	27%
WtoE	3	3721	-106	920	25%
	4	3741	-128	858	23%

31ft to 47ft below pile cap
36 nos. #11 section

fye = 68ksi
f'ce = 5.2ksi phi = 1.0



All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 1809 = 543 kips

	Case	M Capacity	P	M demand	D/C
EtoW	1	5274	-191	2742	52%
	2	5252	-158	1552	30%
WtoE	3	5258	-167	1423	27%
	4	5236	-134	1285	25%

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Reinforcement (EXT)
BY: RSGR
CHK: MWBM

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Shear Design Check - top section with #5 spiral at 18" spa.

V, Demand	Vu	157	kips	
V, Threshold	Vt	134	kips	AASHTO LRFD BDS Eq C5.7.2.3-1

Limit

dv=	32.87	in	AASHTO LRFD BDS Eq C5.7.2.8-2
bv=	48	in	
f'c=	4	ksi	
V, limit=	1578	kips	AASHTO LRFD BDS Eq C5.7.3.3-2

Concrete Strength

tensile strain, es=	0.0008		AASHTO LRFD BDS Eq 5.7.3.4.2-4
β=	2.99		AASHTO LRFD BDS Eq 5.7.3.4.2-1
λ=	1		
Vc, Nominal concrete shear strength=	298	kips	AASHTO LRFD BDS Eq 5.7.3.3-3

Steel Tie Strength - #5 bar at 18in vertical and horizontal spacing

s=	18	in	
Av=	0.31	in2	#5 within distance 18"
Θ=	31.83	degrees	AASHTO LRFD BDS Eq 5.7.3.4.2-3
Vs, Steel shear strength=	55	kips	AASHTO LRFD BDS Eq 5.7.3.3-4

				demand	
Total factored shear strength, V=	317	kips	>	157	OK

If this section works, other sections are Ok by inspection as max shear demand is 400 kips for spiral spacing at 3"
Steel spiral at 3" has a Vs of 300 kips

Torsion Design Check (AASHTO LRFD ARTICLE 5.7.2.1-3)

P=	133	kips	unfactored DL per shaft at the top
Ag=	1809	in2	
fpc=	0.074	ksi	
K=	1.14		
Acp=	1809	in2	
pc=	151	in	
Tcr	518	k-ft	
Torsion threshold, T=	117	k-ft	
T, Demand Tu	63	k-ft	

Since Tu < Threshold, Torsional effects need not be investigated

Torsional effects shall be investigated where:

$T_u > 0.25\phi T_{cr}$ (5.7.2.1-3)

- For solid shapes:

$T_{cr} = 0.126K\lambda\sqrt{f'_c}\,\frac{A_{cp}^2}{p_c}$ (5.7.2.1-4)

Minimum Reinforcement Checks - Seismic Detailing (AASHTO LRFD BDS Article 5.11.4)

5.11.4.1.1—Longitudinal Reinforcement

The area of longitudinal reinforcement shall not be less than 0.01 or more than 0.04 times the gross cross-section area, A_g.

This requirement is met
Min. area of steel is 12nos. #11 in shaft

ρs, min=	0.0105	OK
ρs, max=	0.031	OK

5.11.3.2—Concrete Piles

Concrete piles shall be anchored to the pile footing or cap by either embedment of reinforcement or anchorages to develop uplift forces. The embedment length shall not be less than the development length required for the reinforcement specified in Article 5.10.8.2.

Embedment length provided=	4.50	ft	
Embedment length required=	3.42	ft	OK

Tension Development Length for #11 bar per AASHTO LRFD BDS Article 5.10.8.2

5.11.4.5.4—Cast-in-Place Piles

For cast-in-place piles, longitudinal steel shall be provided for the full length of the pile. In the upper two thirds of the pile, the longitudinal steel ratio, provided by not less than four bars, shall not be less than 0.75 percent. For piles less than 24.0 in. in diameter, spiral reinforcement or equivalent ties of not less than No. 3 bars shall be provided at a pitch not exceeding 9.0 in., except that the pitch shall not exceed 4.0 in. within a length below the pile cap reinforcement of not less than 4.0 ft and where the volumetric ratio and splice details shall conform to Articles 5.11.4.1.4, 5.11.4.1.5, and 5.11.4.1.6.

MET

5.11.4—Seismic Zones 3 and 4

5.11.4.1—Column Requirements

For the purpose of this article, a vertical support shall be considered to be a column if the ratio of the clear height to the maximum plan dimensions of the support is not less than 2.5. For a flared column, the maximum plan dimension shall be taken at the minimum section of the flare. For supports with a ratio less than 2.5, the provisions for piers of Article 5.11.4.2 shall apply.

A pier may be designed as a pier in its strong direction and a column in its weak direction.

Clear Height of shafts to point of fixity, H=	47	ft	height of pier 3 to point of fixity
Max. plan dimension of support, L=	52	ft	length of Pier 3
Ratio H/L=	0.904	Not treated as Column	

PIER 3 : REINFORCEMENT DESIGN CHECK (STR AND SER CASE)

COWI

PROJECT:	I-405 R2B Final Design - BR 17.7	JOB NO.:	A207833
SUBJECT:	Pier 3 Reinforcement (SER and STR)		
BY:	RSGR	DATE:	2021/12/05
CHK:	MWBM	DATE:	2021/12/05

Purpose:
Check the flexure and shear capacity for the Pier 3 shafts for SER and STR cases

Notes:
1. Unfactored demands for earth pressure and live load surcharge behind Pier 3 are taken from geotech's PY-Model which shows a good match to CSI model demands where there bottom of shafts are modelled at a point of fixity of 47ft below top of pile cap
2. Shaft has varying flexural and shear capacity along its length due varying reinforcement layout.
3. Design checks are carried out on 'per-shaft' basis.
4. Locked-in stress due to construction stage is added to the shear and moment demands to point of fixity. Locked-in stress are estimated by letting Pier 3 move 1" towards the West during construction stage.
5. The shafts can resist the demands from lengthened bridge without retrofitting the pier.

Shaft Reinforcment along the length

	height below top of pile cap		EL NAV88	height below top of pile cap	EL NAV88	
Assuming 0ft at top of pile cap	0	ft	107	0	107	
Bottom of pile cap	8.5	ft below top of pile cap	98.5	8.5	98.5	
end of 12 nos. #11 section (inc. splice length)	22.5	ft below top of pile cap	84.5	13	94	#5 spiral 3"SPA from EL 98.5 to 94
end of 24 nos. #11 section (inc. splice length)	31	ft below top of pile cap	76	28	79	#5 spiral 18"SPA from EL 94 to 79
end of 36 nos. #11 section (inc. splice length)	47	ft below top of pile cap	60	47	60	#5 spiral 3"SPA from EL 79 to EL60

Note that the Shaft is only modelled upto 47ft below top of pile cap (EL 56.5) for STR and EXT case in bridge FEM models
The moment and shear demands at the bottom of shaft are to be taken from Geo-tech's PY-Wall model
i405 level is at ~EL74 (or 33ft below top of pile cap)

DIMENSIONS & PARAMETERS

f'c	4	ksi	concrete compressive strength
wc =	0.155	kcf	Unit weight of weight concrete
K1 =	1		Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4028	ksi	Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
fy =	60	ksi	Yield strength of reinforcement
Es =	29000	ksi	Modulus of elasticity of reinforcement
η =	7.20		Modular ratio
β_1 =	0.85		Compression Zone Factor
φf=	0.9		flexural resistance factor
φv=	0.9		shear resistance factor

LOADING IN PIER 3 Wall (Per Shaft) - Lateral Earth Pressure and Live load Surcharge

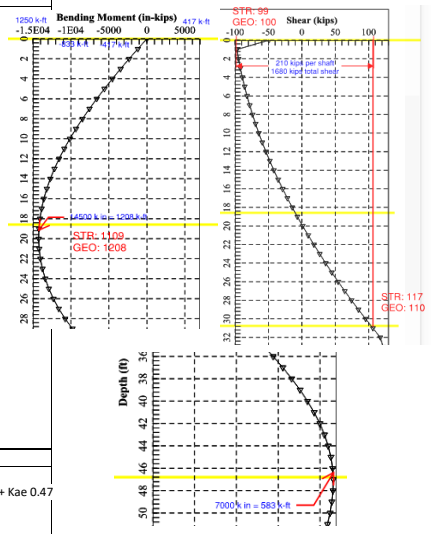
LEP and LS (per shaft)

Taken from Geo-engineers PY-Wall model

	V2	M3	
	Kip	Kip-ft	
Max top 20ft	70	1208	The contribution of of LS load is 8% and LEP is 92%
Max 20 to 30ft	50	1208	
Max 30 ft to 47ft	110	-583	

The contribution of of LS load is 8% and LEP is 92% in the above table

These are combined with other load effects at pier 3 from when the bridge had finished construction



EL : Locked in Construction Stress at the point of fixity

These are from the case when the Pier 3 is allowed to move 1" to the west
These locked in demands include LEP with Ko of 0.38 which is then combined with EQ demands that already has LEP with Ka of 0.22 + Kae 0.47
Hence, this demand is more than double counting the LEP active pressure effect of Ka 0.22 which is conservative

V2	M3	
Kips	Kip-ft	
130	-1000	These demands are added to 36-#11 sections

COWI

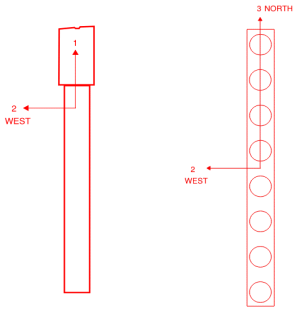
PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Reinforcement (SER and STR)
BY: RSGR
CHK: MWBM

JOB NO.: A207833
DATE: 2021/12/05
DATE: 2021/12/05

LOADING IN PIER 3 WALL (Per Shaft) (8.5ft to 17.5ft below top pile cap)

	P	V2*	V3*	T*	M2	M3	
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
DC, Supr+pile cap+self weight of shaft	-133	1	1	0	0	26	
CR&SH	-9	1	0	0	0	-24	
PS	-2	2	0	1	0	-26	
TU-	0	7	1	0	0	-121	
TU+	0	5	0	0	0	87	
TG-	2	0	0	0	0	-5	
TG+	-6	1	1	0	0	17	
WL	0	0	3	0	0	1	
WS (SER)	0	1	1	0	0	4	
WS (STR3)	-2	1	3	0	0	8	
WS (STR5)	0	1	2	0	0	6	
Max LL	-21	5	4	1	0	91	IM included; BR and CE included
Min LL	2	5	4	1	0	-83	IM included; BR and CE included
LEP	0	64	0	0	0	1111	These are behind Pier 3
LS	0	6	0	0	0	97	
SE	2	1	0	0	0	-10	

Note:
* max absolute values of shear and torsion force selected for design check



LOADING ON THE PIER 3 WALL (per shaft) (17.5ft to 25.5ft below top pile cap)

	P	V2*	V3*	T*	M2	M3	
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
DC, Supr+pile cap+self weight of shaft	-164	1	1	0	0	-41	
CR&SH	-9	1	0	0	0	38	
PS	-2	2	0	1	0	41	
TU-	0	7	1	0	0	191	
TU+	0	5	0	0	0	-137	
TG-	2	0	0	0	0	8	
TG+	-6	1	1	0	0	-27	
WL	0	0	3	0	0	-2	
WS (SER)	0	1	1	0	0	-11	
WS (STR3)	-2	2	3	0	0	-21	
WS (STR5)	0	2	2	0	0	-14	
Max LL	-21	5	4	1	0	-143	IM included; BR and CE included
Min LL	2	5	4	1	0	130	IM included; BR and CE included
LEP	0	46	0	0	0	1111	These are behind Pier 3
LS	0	4	0	0	0	97	
SE	2	1	0	0	0	15	

Note:
* max absolute values of shear and torsion force selected for design check

LOADING ON THE PIER 3 WALL (per shaft) (25.5ft to 50.5ft below top pile cap)

	P	V2*	V3*	T*	M2	M3	
	Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	
DC, Supr+pile cap+self weight of shaft	-213	2	1	0	0	-67	
CR&SH	-11	2	0	0	0	64	
PS	-5	3	0	1	0	73	
TU-	-1	7	1	0	0	309	
TU+	0	5	0	0	0	-222	
TG-	3	0	0	0	0	14	
TG+	-10	1	1	0	0	-45	
WL	-11	1	3	0	0	-6	
WS (SER)	-5	1	1	0	0	-32	
WS (STR3)	-16	3	3	0	0	-61	
WS (STR5)	-7	2	2	0	0	-42	
Max LL	-53	6	4	1	0	-235	IM included; BR and CE included
Min LL	14	6	4	1	0	214	IM included; BR and CE included
LEP	0	101	0	0	0	-536	These are behind Pier 3
LS	0	9	0	0	0	-47	
SE	3	1	0	0	0	25	
EL	0	130	0	0	0	-1000	

Note:
* max absolute values of shear and torsion force selected for design check

		DC	LL	CR SH PS	WS	WL	TU	SE	TG	LEP	LS	EL
STR I	Max	1.25	1.75	1	0	0	1	1	0	1.35	1.75	1
	Min	0.9	1.75	1	0	0	1	1	0	1.35	1.75	1
STR III	Max	1.25	0	1	1	0	1	1	0	1.35	0	1
	Min	0.9	0	1	1	0	1	1	0	1.35	0	1
STR V	Max	1.25	1.35	1	1	1	1	1	0	1.35	1.35	1
	Min	0.9	1.35	1	1	1	1	1	0	1.35	1.35	1
SER	Max	1	1	1	1	1	1	1	1	1	1	1
	Min	1	1	1	1	1	1	1	1	1	1	1

12nos. #11; (EL 98.5 to EL 84.5); #5 Spiral at ~3" (EL 98.5 to EL 94)

									Combined SRSS	
			P	V2*	V3*	T*	M2	M3	V	M
Case			Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	kips	Kip-ft
1	STR I	Max	-212	117	9	3	0	1888	118	1888
2		Min	-125	118	9	3	0	1584	119	1584
3	STR III	Max	-177	100	5	1	0	1569	100	1569
4		Min	-131	100	5	1	0	1569	100	1569
5	STR V	Max	-204	114	13	2	0	1820	115	1820
6		Min	-126	115	12	2	0	1586	116	1586
7	SER	Max	-169	89	11	2	0	1375	89	1375
8		Min	-138	89	11	2	0	1201	90	1201

24nos. #11; (EL 84.5 to EL 76); #5 Spiral at ~3" (EL 94 to EL 79)

									Combined SRSS	
			P	V2*	V3*	T*	M2	M3	V	M
Case			Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	kips	Kip-ft
1	STR I	Max	-251	90	9	3	0	1325	90	1325
2		Min	-152	91	9	3	0	1803	91	1803
3	STR III	Max	-216	77	5	1	0	1385	77	1385
4		Min	-158	77	5	1	0	1385	77	1385
5	STR V	Max	-242	88	12	2	0	1328	89	1328
6		Min	-153	89	11	2	0	1696	89	1696
7	SER	Max	-200	69	11	2	0	942	70	942
8		Min	-168	69	11	2	0	1215	70	1215

36nos. #11; (EL 76 to EL 60); #5 Spiral at ~3" (EL 79 to EL 60)

									Combined SRSS	
			P	V2*	V3*	T*	M2	M3	V	M
Case			Kip	Kip	Kip	Kip-ft	Kip-ft	Kip-ft	kips	Kip-ft
1	STR I	Max	-373	308	9	3	0	-2361	308	2361
2		Min	-180	306	8	3	0	-1575	306	1575
3	STR III	Max	-296	285	5	1	0	-1929	285	1929
4		Min	-221	283	4	1	0	-1929	283	1929
5	STR V	Max	-370	305	13	2	0	-2296	305	2296
6		Min	-204	303	12	2	0	-1690	303	1690
7	SER	Max	-306	264	11	2	0	-2028	264	2028
8		Min	-225	261	9	2	0	-1520	261	1520

COWI

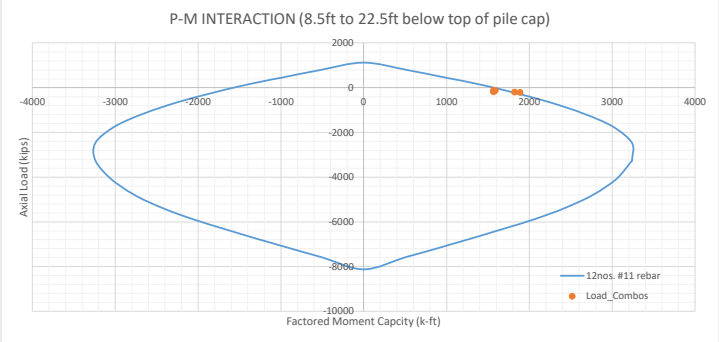
PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Reinforcement (SER and STR)
BY: RSGR
CHK: MWBM

JOB NO.: A207833
DATE: 2021/12/05
DATE: 2021/12/05

Axial - Flexure Design Check

AXIAL P - MOMENT M INTERACTION CURVE FOR PIER 3 SHAFTS (AASHTO LRFD BDS Article 5.6.4.5)

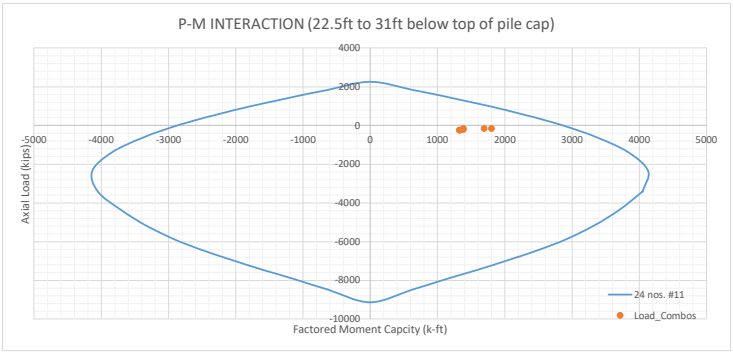
Height: 8.5ft to 22.5ft below pile cap fy = 60ksi
Rebar: 12 nos. #11 section f'c = 4ksi phi = 0.9



All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 1809 = 543 kips

	Case	M Capacity	P	M demand	D/C
STR I	1	1796	-212	1888	105%
	2	1700	-125	1584	93%
STR III	3	1758	-177	1569	89%
	4	1706	-131	1569	92%
STR V	5	1787	-204	1820	102%
	6	1701	-126	1586	93%

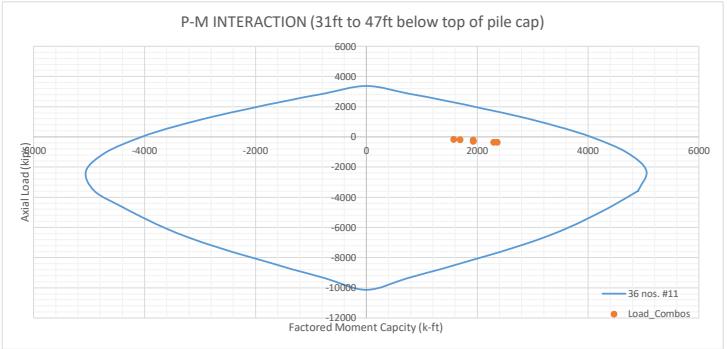
22.5ft to 31ft below pile cap fy = 60ksi
24 nos. #11 section f'c = 4ksi phi = 0.9



All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 1809 = 543 kips

	Case	M Capacity	P	M demand	D/C
STR I	1	3069	-251	1325	43%
	2	2987	-152	1803	60%
STR III	3	3040	-216	1385	46%
	4	2992	-158	1385	46%
STR V	5	3062	-242	1328	43%
	6	2987	-153	1696	57%

31ft to 47ft below pile cap fy = 60ksi
36 nos. #11 section f'c = 4ksi phi = 0.9



All Axial demands are less than
Limit = 0.10 x phi x f'c x Ag = 0.10 x 0.75 x 4 x 1809 = 543 kips

	Case	M Capacity	P	M demand	D/C
STR I	1	4290	-373	2361	55%
	2	4171	-180	1575	38%
STR III	3	4242	-296	1929	45%
	4	4196	-221	1929	46%
STR V	5	4288	-370	2296	54%
	6	4185	-204	1690	40%

COWI

PROJECT:	I-405 R2B Final Design - BR 17.7	**JOB NO.:**	A207833
SUBJECT:	Pier 3 Reinforcement (SER and STR)		
BY:	RSGR	**DATE:**	2021/12/05
CHK:	MWBM	**DATE:**	2021/12/05

Service Design Check - Critical section at top with 12 #11 rebars (max service moment above i405 level)

Check Cracking using SER demands

M, SER=	16497	k-in			
Ye =	1.00	Class 1 exposure			
thickness of member, h=	48	in			
Thickness of concrete cover, dc=	4.33	in	=cover 3"+#5 spiral 0.625" + 0.5x #11 bar 0.5x1.41		
β=	1.14				
Tensile Reinforcement Ratio, ρ =	0.01046	Tensile reinforcement for 12 #11 rebar region			
η =	7.20				
k=	0.320	$k = \sqrt{2n\rho + (n\rho)^2} - n\rho$			
j=1-k/3=	0.893			0.6 x f'y	
fss=	23	ksi	<	36	ksi
max. spacing of reinforcement, s=	18	in			
provided, s=	10	in	OK		

Shear Design Check - top section with #5 spiral at 18" spa.

V, Demand	Vu	91	kips	
V, Threshold	Vt	112	kips	AASHTO LRFD BDS Eq C5.7.2.3-1
Limit				
dv=		32.87	in	AASHTO LRFD BDS Eq C5.7.2.8-2
bv=		48	in	
f'c=		4	ksi	
V, limit=		1578	kips	AASHTO LRFD BDS Eq C5.7.3.3-2

Concrete Strength

tensile strain, εs=	0.0012		AASHTO LRFD BDS Eq 5.7.3.4.2-4
β=	2.49		AASHTO LRFD BDS Eq 5.7.3.4.2-1
λ=	1		
Vc, Nominal concrete shear strength=	248	kips	AASHTO LRFD BDS Eq 5.7.3.3-3

Steel Tie Strength - #5 bar at 18in vertical and horizontal spacing

s=	18	in	
Av=	0.31	in2	#5 within distance 18"
Θ=	33.34	degrees	AASHTO LRFD BDS Eq 5.7.3.4.2-3
Vs, Steel shear strength=	52	kips	AASHTO LRFD BDS Eq 5.7.3.3-4

					demand	
Total factored shear strength, V=	270	kips	>	91	OK	

If this section works, other sections are Ok by inspection as max shear demand is 308 kips for spiral spacing at 3"
Steel spiral at 3" has a Vs of 300 kips

COWI

PROJECT:	I-405 R2B Final Design - BR 17.7	JOB NO.:	A207833
SUBJECT:	Pier 3 Reinforcement (SER and STR)		
BY:	RSGR	DATE:	2021/12/05
CHK:	MWBM	DATE:	2021/12/05

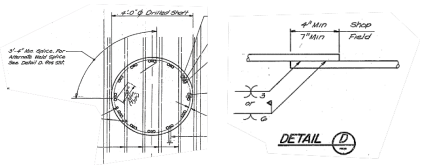
Checks Spiral detailling AASHTO LRFD BDS 5.10.4

Anchorage of spiral reinforcement shall be provided by 1.5 extra turns of spiral bar or wire at each end of the spiral unit. For Seismic Zones 2, 3, and 4, the extension of transverse reinforcement into connecting members shall meet the requirements of Article 5.11.4.3.

Splices in spiral reinforcement may be one of the following:

- Lap splices of 48.0 uncoated bar diameters, 72.0 coated bar diameters, or 48.0 wire diameters;
- approved mechanical connectors; or
- approved welded splices.

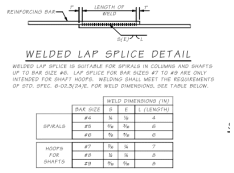
No Details about anchorage provided



No Details on whether done in shop or field.

WSDOT BDM WELDED LAP SPLICE DETAIL

Figure 7.3.5-6 Welded Spiral Splice and Butt Splice Details



Minimum Transverse Reinforcement (AASHTO LRFD BDS Article 5.7.2.5)

$$A_v \geq 0.0316\,\lambda\sqrt{f'_c}\,\frac{b_v\,s}{f_y} \qquad (5.7.2.5\text{-}1)$$

λ=	1			
f'c=	4	ksi		
bv=	48	in		
s=	18	in		
f'y=	60	ksi		
Av, required=	0.91	in2		
Av, provided=	0.21	in2	**NOT OK**	#5 spiral @ 18" Spacing

The shaft does not have minimum required transverse reinforcement for areas with #5 at 18" spacing

Spacing Check AASHTO LRFD BDS 5.7.2.6

shear stress, vu=	0.06	<	0.125*f'c = 0.50	ksi
s, max=	24	in		
provided, s=	18	in	**OK**	Max spiral spacing

PIER 3 : CHECK AASHTO SEISMIC GUIDE REQUIREMENTS

COWI

Project	I-405	**Project No**	A207833
Title	Disp.Capacity - Pier 3	**Date**	12/5/2021
Author	RSGR	**Checker**	MWBM

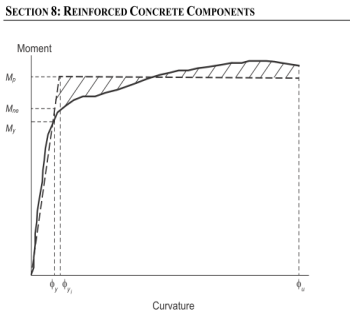
The plastic hinge is assumed to occur at point of fixity level where there are 36 - #11 bars

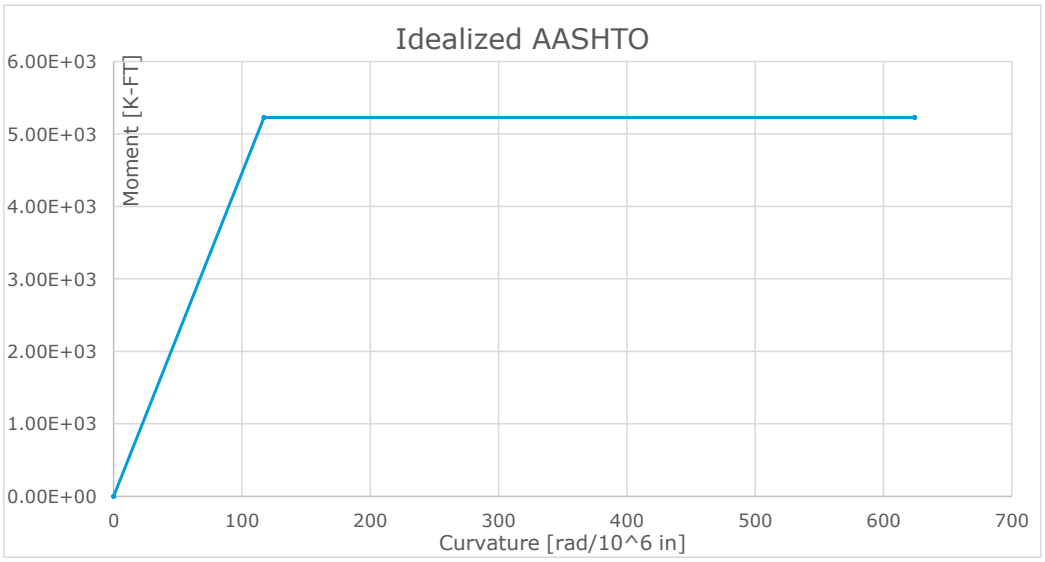
φ_first rebar yield=	8.66E+01	(rad/10^6 in)	From M-curvature analysis, see previous page
M_first rebar yield =	3859	k-ft	From M-curvature analysis, see previous page
EI_first rebar yield =	4.46E+01	k-ft/(rad/10^6in)	
φu =	6.24E+02	(rad/10^6 in)	From M-curvature analysis, see previous page

Moment -curvature

Mp =	5227	k-ft
φyi=	117.2423	(rad/10^6 in)
φu =	6.24E+02	(rad/10^6 in)
φpd=	506.758	(rad/10^6 in)

Φ	M_idealized
rad/10^6	k-ft
0	0.00E+00
1.17E+02	5.23E+03
6.24E+02	5.22658E+03





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COWI

Project	I-405	**Project No**	A207833
Title	Disp.Capacity - Pier 3	**Date**	12/5/2021
Author	RSGR	**Checker**	MWBM

Longitudinal displacement capacity of shaft

Elastic displacement

L =	47.00	ft, Height from top of pile cap to point of fixity	
φyi=	0.0001	(rad/10^6 in), from AASHTO curve	
Δy =	12.43	in, φyi x L^2/3 elastic displacement	AASHTO Seismic Eq.C4.9-3

Plastic displacement

Longitudinal

Analytical plastic hinge

L =	47.0	ft	
	564.0	in	
fye =	68	ksi	AASHTO Seismic Table 8.4.2-1
dbl =	1.56	in	#11rebar
0.08L=	45.1	in	
0.15fye*dbl =	15.91	in	
Lp =	61.0	in	AASHTO Seismic Eq. 4.11.6-1
Lp(min)=	31.82	in	
Lp=	61.03	in	
φpd=	0.0005	(rad/10^6 in)	
θp =	0.031	rad, φpdxL	
L - Lp/2=	533.48	in,	
Δp=	16.50	in,	

Total displacement capacity

Δy =	12.43	in,
Δp=	16.50	in,
Δ.total =	28.93	in,

Results: 5% Drift

Ductility Capacity=	2.33	=1+(Δp/Δy)
Total Displacement Capacity of Shaft	28.93	in

Overstrength Results:

M0	7317	k-ft	(1.4*Mp AASHTO Seismic Eq 8-5.1); Grade A615
H	47.00	ft	From top of pile cap to bottom of point of fixity
Vo	156	Kips	(M0/H)

Ductility demand=

Max. Pier 3 movement, Δ=	2.75	in	From EtoW analysis of bridge		
θ col =	0.0052	=Δ"/(L - Lp/2)			
θ elastic =	0.0233	=Δy"/(L - Lp/2)			
Δp=	0.0000			Ductility Capacity	
Ductility demand=	1.00	=1+Δp/Δy	<	2.33	OK

4.9—MEMBER DUCTILITY REQUIREMENT FOR SDC D

For multiple-column bents:

$\mu_D \le 6$ (4.9-2)

Ductility Capacity =	2.33	MET
Ductility Demand =	1	

Shear Design Check - Overstrength EQ demands as per AASHTO SEISMIC 8.6

Overstrength Shear, Vpo=	311	kips	= 2 x Vpo in long. Direction	
Ag=	1810	in2		
Ae=	1448	in2	$A_e = 0.8A_g$	
8.6.2—Concrete Shear Capacity f'c=	4	ksi		
Pu=	150	kips	unfactored DL	
Asp=	0.31	in2	in 1ft high section	1 nos. #5 spirals in 3in spacing
b=	48.0	in	width of colum	
s=	3	in	vertical direction spacing	
ρs=	0.010	>	Limit 0.005 Per AASHTO Seismic eq. 8.6.5-3	
fs=	0.60	>	Limit 0.35, Hence, use 0.35	
fs=	0.35			
μd=	3.0		For SDC C, the concrete shear capacity of a section within the plastic hinge region shall be determined using $\mu_D = 3$.	
α'=	3.0			

			limit		
concrete shear stress strength, vc=	0.20	ksi	0.22		
Concrete Shear strength, Vc=	290	kips	limit		
Stirrup shear strength, Vs=	403	kips	723.82	kips	AASHTO seismic eq. 8.6.4-1
Factored 0.9=	623	kips	>	311 kips demand	OK

For rectangular columns with ties:

$\alpha' = \frac{f_w}{0.15} + 3.67 - \mu_D$ (8.6.2-8)

$f_w = 2\rho_w f_{yh} \le 0.35$ (8.6.2-9)

$\rho_w = \frac{A_v}{bs}$ (8.6.2-10)

If P_u is compressive:

$$v_c = 0.032\alpha'\left(1 + \frac{P_u}{2A_g}\right)\sqrt{f_c'} \le \min\begin{cases} 0.11\sqrt{f_c'} \\ 0.047\alpha'\sqrt{f_c'} \end{cases}$$ (8.6.2-3)

$V_c = v_c A_e$

8.7—REQUIREMENTS FOR DUCTILE MEMBER DESIGN

8.7.1—Minimum Lateral Strength

The minimum lateral flexural capacity of each column shall be taken as:

$M_{ne} \ge 0.1 P_{trib}\left(\frac{H_h + 0.5D_s}{\Lambda}\right)$ (8.7.1-1) MET

P=	150	kips
Hh=	47	ft
Ds=	4.25	ft
Λ=	1	Pin - Fixed
Mne=Mp=	5227	k-ft

8.8—LONGITUDINAL AND LATERAL REINFORCEMENT REQUIREMENTS

8.8.1—Maximum Longitudinal Reinforcement

The area of longitudinal reinforcement for compression members shall satisfy:

$A_t \le 0.04 A_g$ (8.8.1-1)

where:

A_g = gross area of member cross-section (in.^{2}) MET

A_t = area of longitudinal reinforcement in the member (in.^{2})

A(L)=	56	in2	Max steel	36 nos. #11 rebar
A(L)=	19	in2	min steel	12 nos. #11 rebar
0.04Ag=	72	in2	Max limit	
0.007Ag=	12.7	in2	Min limit	

8.8.2—Minimum Longitudinal Reinforcement

The minimum area of longitudinal reinforcement for compression members shall not be less than:

- For columns in SDCs B and C:

$A_t \ge 0.007 A_g$ (8.8.2-1) MET

8.8.3—Splicing of Longitudinal Reinforcement in Columns Subject to Ductility Demands for SDCs C and D

Splicing of longitudinal column reinforcement in SDC C or D shall be outside the plastic hinging region as defined in Article 4.11.7, except as permitted below.

For a pile or shaft in SDC D where liquefaction is anticipated, the design shall consider that the zone comprising the location of potential plastic hinging in the liquefied and nonliquefied cases can be large. For a pile or shaft in SDC D where splicing in the zone cannot be avoided, mechanical couplers that are capable of developing the expected tensile strength of the bars and as approved by the Owner shall be specified.

Spilicing of #11 rebar done without mechanical couplers throughout. But shafts are able to stay elastic under SEE level EQ.

PIER 3 : CHECK WSDOT REQUIREMENTS

7.8.2 *Structural Design and Detailing*

Standard Specifications Section 6-19 should be reviewed as part of the design of shafts. The structural design of shafts is similar to column design. The following guidelines shall be followed:

A. For shaft foundation supporting columns in any SDC C or D, the shaft nominal moment capacity shall be designed to resist 1.25 times the moment demand generated in the shaft by the overstrength column plastic hinge moment at the base of the column. **NOT APPLICABLE**

B. Concrete Class 5000P shall be specified for the entire length of the shaft for wet or dry conditions of placement. **NOT MET AS CLASS 4000 CONCRETE USED IN AS-BUILT SHAFTS**

C. When shafts are constructed in water, the concrete specified for the casing shoring seal shall be Class 4000W. **NOT APPLICABLE**

D. The assumed concrete compressive strength may be taken as $f'c$ for structural design of shafts. For seismic design, the expected compressive strength may be increased by 1.3 in accordance with AASHTO Seismic Section 8.4.4. **USED**

E. The presence of permanent steel casing shall be taken into account in the shaft design (i.e. for stiffness, and etc.), but the structural resistance of permanent steel casing shall not be considered for structural design of shafts unless the design conforms to Section 7.10. **NOT APPLICABLE**

F. Cover requirements vary depending on the shaft diameter and shall be as specified below:
- Diameter less than or equal to 3′-0″ = 3″
- Diameter greater than 3′-0″ and less than 5′-0″ = 4″
- Diameter greater than or equal to 5′-0″ = 6″

NOT MET AS 3" COVER USED IN AS-BUILT SHAFTS

Standard Specifications Section 6-19 lists exceptions to these cover requirements when permanent slip casings are used in column splice zones.

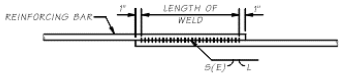
G. In general, shaft reinforcing shall be detailed to minimize congestion, facilitate concrete placement by tremie, and maximize consolidation of concrete. **NOT APPLICABLE**

H. The clear spacing between spirals and hoops shall not be less than 6″ or more than 9″, with the following exception. The clear spacing between spirals or hoops may be reduced in the splice zone in single column/single shaft connections because shaft concrete may be vibrated in this area, negating the need for larger openings to facilitate good flow of concrete through the reinforcing cage. **CLEAR SPACING AT SOME REGION IS MORE THAN 9" AND LESS THAN 6"**

I. The volumetric ratio and spacing requirements of the AASHTO Seismic Specifications for confinement need not be met. The top of shafts in typical WSDOT single column/single shaft connections remains elastic under seismic loads due to the larger shaft diameter (as compared to the column). Therefore this requirement does not need to be met. **OK**

J. Shaft transverse reinforcement may be constructed as hoops or spirals, or a combination of the two. Spiral reinforcement is preferred for shaft transverse reinforcement. However, if #6 spirals at 6″ (excluding the exception in 7.8.2.H) clear do not satisfy the shear design, circular hoops may be used. Circular hoops in shafts up to #9 bars may be lap spliced using a welded lap detail. Note: Welded lap splices for spirals are currently acceptable under the AWS D1.4 up to bar size #6. Recent testing has been performed by WSDOT for bar sizes #7 through #9. All tests achieved full tensile capacity (including 125 percent of yield strength.) Therefore, #7 through #9 welded lap spliced hoops are acceptable to use provided they are not located in possible plastic hinge regions. Circular hoops may also be fabricated using a manual direct butt weld, resistance butt weld, or mechanical coupler. Weld splicing of hoops for shafts shall be completed prior to assembly of the shaft steel reinforcing cage. Refer to Section 7.3.5F for additional discussion on circular hoops. Mechanical couplers may be considered provided cover and clearance requirements are accounted for in the shaft details. When welded hoops or mechanical couplers are used, the plans shall show a staggered pattern around the perimeter of the shaft so that no two adjacent welded splices or couplers are located at the same location. **WELDED LAP SPLICE LENGTH NOT ADEQUATE**

Figure 7.3.5-6 Welded Spiral Splice and Butt Splice Details



WELDED LAP SPLICE DETAIL

WELDED LAP SPLICE IS SUITABLE FOR SPIRALS IN COLUMNS AND SHAFTS UP TO BAR SIZE #6. LAP SPLICE FOR BAR SIZES #7 TO #9 ARE ONLY INTENDED FOR SHAFT HOOPS. WELDING SHALL MEET THE REQUIREMENTS OF STD. SPEC. 6-02.3(24)E. FOR WELD DIMENSIONS, SEE TABLE BELOW.

		WELD DIMENSIONS (IN)		
	BAR SIZE	S	E	L (LENGTH)
SPIRALS	#4	¼	⅛	4
	#5	⅝	⅜	6
	#6	⅝	⅜	6
HOOPS FOR SHAFTS	#7	⅞	¼	7
	#8	½	¼	8
	#9	⅞	⅝	8

K. In single column/single shaft configurations, the spacing of the shaft transverse reinforcement in the splice zone shall meet the requirements of the following equation, which comes from the TRAC Report titled, *"Noncontact Lap Splices in Bridge Column-Shaft Connections"*: **NOT APPLICABLE**

$$S_{max} = \frac{2\pi A_{sh} f_{ytr} l_s}{k A_l f_{ul}} \qquad (7.8.2\text{-}1)$$

L. Longitudinal reinforcement shall be provided for the full length of shafts. The minimum longitudinal reinforcement in the splice zone of single column/single shaft connections shall be the larger of 0.75 percent A_g of the shaft or 1.0 percent A_g of the attached column. The minimum longitudinal reinforcement beyond the splice zone shall be 0.75 percent A_g of the shaft. The minimum longitudinal reinforcement in shafts without single column/single shaft connections shall be 0.75 percent A_g of the shaft.

NOT APPLICABLE AS THIS IS APPLICABLE TO SINGLE COLUMN ON SINGLE SHAFT

M. The clear spacing between longitudinal reinforcement shall not be less than 6″ or more than 9″. If a shaft design is unable to meet this minimum requirement, a larger diameter shaft shall be considered. Alternatively, Grade 80 reinforcing steel could be considered. Where 3-bar bundles are used, the plans shall allow the contractor to construct the cage with two of the three bars located towards the center of the shaft. This allows the contractor flexibility in constructing the cage, but it reduces the flexural resistance of the shaft.

MET

N. Longitudinal reinforcing in shafts should be straight with no hooks to facilitate concrete placement and removal of casing. If hooks are necessary to develop moment at the top of a shaft (in a shaft cap situation) the hooks should be turned toward the center of the shaft while leaving enough opening to allow concrete placement with a tremie.

MET

O. Locations of longitudinal splices shall be shown in the contract plans. Mechanical splices shall be placed in low stress regions and staggered 2′-0″ minimum.

MET

P. Use of two concentric circular rebar cages shall be avoided.

MET

Q. Resistance factors for Strength Limit States shall be per the latest AASHTO LRFD. Resistance factors for Extreme Event Limit States shall be per the latest AASHTO *Seismic Specifications*. The resistance factor for shear shall conform to the AASHTO LRFD.

USED

R. The axial load along the shaft varies due to the side friction. It is considered conservative, however, to design the shaft for the full axial load plus the maximum moment. The entire shaft normally is then reinforced for this axial load and moment.

CURRENT ANALYSIS AND DESIGN CHECK DOES NOT INCLUDE AXIAL LOAD FROM SIDE FRICTION. INCLUSION OF AXIAL LOAD DUE TO SIDE FRICTION INTO LOADS CONSIDERED FROM STURCTURE WOULD INCREASE THE FLEXURAL CAPACITY AND HELP THE D/C RATIOS. HENCE, EXCLUSION OF THIS AXIAL LOAD IS CONSERVATIVE IN THE DESIGN CHECK

S. Access tubes for Crosshole Sonic Log (CSL) testing or thermal wires for Thermal Integrity Profiling (TIP) shall be provided in all shafts per *Standard Specifications* Section 6-19.3(6)B.

NOT APPLICABLE

T. Shafts shall be specified in English dimensions and shall be specified in sizes that do not preclude any drilling method. Shafts shall be specified in whole foot increments except as allowed here. The tolerances in *Standard Specifications* Section 6-19 accommodate metric casing sizes and/or oversized English casing sizes. Oversized English casings are often used so that tooling for drilling the shafts, which are the nominal English diameter, will fit inside the casing. There are a few exceptions, which will be discussed below. See Table 7.8.2-1 for casing sizes and tolerances.

NOT APPLICABLE

PIER 3 : PILE CAP - BRIDGE HINGE DOWEL / SHEAR KEY EXT CASE CHECK

COWI

PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

Purpose:
Design the hinge rebar (dowel) and Shear Key at Pier 3 to EXT Loads (DL; LL; EQ)
Design Code:
AASHTO LFRD 9th Edition Chapter 5.13 which refers to ACI 318 - 14 Chapter 17
AASHTO LFRD 9th Edition Chapter 5.7.4 for Interface shear
AASHTO SEISMIC DESIGN GUIDE
Asumptions:
1. Hinge rebar is a shear connector. It is designed to transfer shear force between superstructure and substructure inc. transfer of torsion from supersturcture as tension through the hinge rebars.
2. Total loads from superstructure/substructure are distributed per dowel to calculated max. load effect per dowel. i.e. Max Shear and Max. Tension in the critical dowel which is the dowel at the end
3. Provided no. of dowels are verified so that load demand per dowel is less than dowel capacity. The following failure modes are checked:
a) Steel failure in tension and shear
b) Concrete failure due to tension and shear
4. The transfer of shear force between superstructure and substructure will als ooccur via concrete interface shear in the shear key
Hence, concrete shear key is then checked to have enough capacity for the interface shear transfer

input

1.0 DIMENSIONS & PARAMETERS

nominal f'c	4	ksi	expected f'ce	5.20	ksi		concrete compressive strength (nominal and expected)
wc =	0.155	kcf					Unit weight of weight concrete
K1 =	1						Correction factor for source of aggregate, AASHTO 5.4.2.4
Ec =	4028	ksi					Modulus of elasticity of concrete, AASHTO Eqn 5.4.2.4-2
fy =	60	ksi	fye =	68.00	ksi		Yield strength of reinforcement (nominal and expected)
f,uta=	90	ksi	f,e uta=	95	ksi	AASHTO seismic Table 8.4.2.1	Ultimate tensile strength of reinforcement (nominal and expected)
Es =	29000	ksi					Modulus of elasticity of reinforcement
η =	7.20						Modular ratio
β_1 =	0.85						Compression Zone Factor
φv=	1						AASHTO SEISMIC Article 3.7
φf=	1						AASHTO SEISMIC Article 3.7
Dowel Qty., N=	51	Nos.					

2.0 Spacing of dowels

Length of Wall/Abut Diaphragm=	52	ft	
Edge distance from CL of anchor=	12	in	the smallest edge distance in Trans. Direction, this value is 24in in Long. direction for edge most bar
Spacing of anchors=	12.0	in	
Use=	12.0	in	

3.0 Horizontal Force demands (between superstructure and substructure)

Seismic WtoE governs here

Main Shear force in Bridge Longitudinal Direction **100%L + 30%T**

LEP EQ Demand, L1=	227	kips	From Seismic earth pressure behind Pier 3, factor of 1.2 for brittle faliure mode
EDA EQ Demand, L2=	2040	kips	From RSA **(Seismic WtoE)**, applied with a factor of 1.2 for brittle faliure mode
LS, EQ Demand, L3=	14	kips	From Live Load Surcharge behind Pier 3 (0.5 factor used)
Live Load LL, L4=	43	kips	Apply a factor of 0.5 in combination below

Associated Transverse shear forces (if any)

LEP Demand, T1=	2	kips	From Seismic earth pressure behind Pier 3, factor of 1.2 for brittle faliure mode
EDA Demand, T2=	966	kips	From RSA **(Seismic WtoE)**, applied with a factor of 1.2 for brittle faliure mode
LS Demand, T3=	0	kips	From Live Load Surcharge behind Pier 3 (0.5 factor used)
Live Load LL, T4=	35	kips	Apply a factor of 0.5 in combination below

Combined shear using SRSS

1=	227	kips
2=	2257	kips
3=	14	kips
4=	28	kips
Total=	2526	kips

Shear per dowel=	49.5	kips	Total shear load / nos. of dowels
Shear per dowel (transverse only)=	19.3	kips	
Shear per dowel (longitudinal only)=	45.1	kips	

Main Shear force in Bridge Transverse Direction **30%L + 100%T**

LEP Demand, T1=	2	kips
EDA Demand, T2=	1560	kips
LS Demand, T3=	0	kips
Live Load LL, T4=	35	kips

Associated Longitudinal shear forces (if any) 30%L + 100%T

LEP EQ Demand, L1=	68	kips
EDA EQ Demand, L2=	732	kips
LS, EQ Demand, L3=	14	kips
Live Load LL, L4=	43	kips

Combined shear using SRSS

1=	68	
2=	1723	
3=	14	
4=	28	
Total=	1833	kips

Shear per dowel=	35.9	kips
Shear per dowel (transverse only)=	16.4	kips
Shear per dowel (longitudinal only)=	31.0	kips

4.0 Vertical Demands - Compression

DL=	525	kips	
LL=	210	kips	Apply a factor of 0.5 in combination
EQ=	-240	kips	(- 30% V)
Min. Compression under EQ =	390	kips	
Compression per dowel=	7.65	kips	

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PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

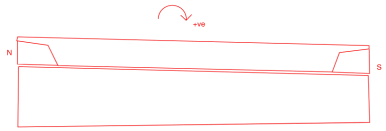
JOB NO.: A207833
DATE: 2021/11/29
DATE: 2021/11/29

5.0 Vertical Demands - Tension due to superstructure torsion

EQ (This can be + or -) =	2798	k-ft
Seismic Earth pressure behind Pier 1=	-36	k-ft
Live load surcharge behind Pier 1=	-30	k-ft
DL=	1020	k-ft
Max Torsion LL=	1500	k-ft
Min Torsion, LL=	-1800	k-ft

From RSA **(Seismic WtoE)**, applied with a factor of 1.2 for brittle faliure mode

Apply a factor of 0.5 in combination
Apply a factor of 0.5 in combination



Max Torsion, Tmax	4502	k-ft
Min Torsion, Tmin	-2744	k-ft

Estimate the tension forces caused by torsion

Assumption: Torsion causes tension force in half of the dowel

Half nos. of dowels= 25
Spacing= 12.0 in

Dowels experience tension force relative to their distance S from structure CL

Tension force in dowel = Tmax x Si / ∑(S^2); Tmax = 3603 kip/ft

	Si (ft)	S^2	Tension (kips)	Compression	Net Force (-ve is tension)	
S1	1	1	-0.815	7.65	6.832	
S2	2	4	-1.630	7.65	6.017	
S3	3	9	-2.445	7.65	5.202	
S4	4	16	-3.260	7.65	4.387	
S5	5	25	-4.075	7.65	3.572	
S6	6	36	-4.889	7.65	2.758	
S7	7	49	-5.704	7.65	1.943	
S8	8	64	-6.519	7.65	1.128	
S9	9	81	-7.334	7.65	0.313	
S10	10	100	-8.149	7.65	-0.502	
S11	11	121	-8.964	7.65	-1.317	
S12	12	144	-9.779	7.65	-2.132	
S13	13	169	-10.594	7.65	-2.947	
S14	14	196	-11.409	7.65	-3.762	
S15	15	225	-12.224	7.65	-4.577	
S16	16	256	-13.039	7.65	-5.392	
S17	17	289	-13.854	7.65	-6.206	
S18	18	324	-14.668	7.65	-7.021	
S19	19	361	-15.483	7.65	-7.836	
S20	20	400	-16.298	7.65	-8.651	
S21	21	441	-17.113	7.65	-9.466	
S22	22	484	-17.928	7.65	-10.281	
S23	23	529	-18.743	7.65	-11.096	
S24	24	576	-19.558	7.65	-11.911	
S25	25	625	-20.373	7.65	-12.726	Max. tension in edge dowel

Total, ∑(S^2)= 5525 ft

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SUBJECT: Pier 3 Shear Key at Top (EXT)
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JOB NO.: A207833
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6.0 Verify Minimum Nos. of Dowel - Single Anchor shear capacity - Vsa

Dowel Size #=	9	#				
Dowel Area, Ad=	1	in2				
Dowel diam., d=	1.128	in				
Expected Dowel Utlimate Tensile Strength, f-ue=	95	ksi				
Shear Resistance Factor, φ dowel=	1					AASHTO SEISMIC Article 3.7
Nominal Resistance, Vn=	57	kips		Max. Demand	D/C	=0.6 x Ad x futa ACI 318 eq 17.5.1.2b
Factored Resistance, Phi x Vn=	57	kips	>	50	87%	

7.0 Verify Minimum Nos. of Dowel - Single Anchor Tensile capacity = Nsa

Dowel Size #=	9	#				
Dowel Area, Ad=	1	in2				
Dowel diam., d=	1.128	in				
Expected Dowel Utlimate Tensile Strength, f-ue=	95	ksi				
Tensile Resistance Factor, φ dowel=	1.00					
Nominal Resistance, Tn=	95	kips		Max. Demand	D/C	= Ad x futa ACI 318 eq 17.4.1.2
Factored Resistance, Phi x Vn=	95	kips	>	13	13%	

8.0 Verify Layout of Anchors in Concrete - Single Anchor

8.1 Concrete breakout due to tension - Ncb

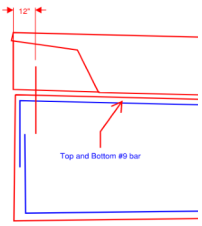
C,a1=	12	in	Min. edge distance in transverse direction	
H,ef used.=	8	in	Use this for tension breakout calcs	ACI 318 cl17.4.2.3
H,ef prov.=	36	in	embedment length into concrete	
A, Nc=	576	in2	Effective concrete area around the edge anchor	ACI 318 cl17.4.2.1c
A, Nco=	576	in2	Projected area of single anchor = 9 x hef x hef	ACI 318 Fig R17.4.2.1
Nb	39	kips	Basic breakout strength of single anchor in tension	ACI 318 CL17.4.2.2a
kc	24		cast in anchor	
gamma a	1		aggregate factor	
sqrt f'c	72.111			
hef	8.000	in		
ψ, ed,N=	1		ca, min = 12in $\geq$ 1.5 h,ef = 12in	
ψ, c,N=	1.25		cast-in-anchors	
ψ, cp,N=	1		cast-in-anchors	
Factor=	1.00			
Factored Concrete breakout, Ncb=	49	kips	> 13 (Max. Demand)	26% (D/C)

8.2 Concrete breakout due to Shear at edge dowel Vcb

Shear in bridge transverse direction

Ca1=	28.5	in	min. distance to edge parallel to edge	
Ca2=	12	in	min. distance to edge perpendicular to edge	
Avco=	648	in2	use smaller Ca	ACI 318 eq 17.5.2.1c
le=	18	in	le=he=load bearing length of dowel	
da=	1.128	in		diameter of dowel
sqrt f'c=	72.11			
Vb=	38.78	kips	use smaller Ca	ACI 318 eq 17.5.2.2a
Limit Vb=	26.98	kips	use smaller Ca	ACI 318 eq 17.5.2.2b
Use limit				
hef=	18	in		effective embedment for shear breakout = 1.5 x Min (Ca1, Ca2)
ha=	40	in		Bearing height
Avc=	837	in2	single anchor at edge	
ψ, ed,V=	1			
ψ, c,V=	1.2		Cracking at EXT loads	
ψ, h,V=	1		ha=1.5ca	
Shear Resistance Factor, φ =	1			
Factored Concrete breakout, Vcb=	41.8	kips		

Max. Trans. Demand 50 | D/C 118%
ACI 318 CL17.5.2.1a



Check supplementary rebar at edge

Bar #	9	2 nos. #9 bars		
Qty.	2	each side of dowel		
Area of steel, As=	2	in2		
f'ye	68.000	ksi	expected steel yield strength	
Nominal Steel Strength, Vs=	136	kips	=f'ye x As	
Factor, φ=	1			
Factored steel strength, f x Vs=	136	kips		
Total Factored Strength, Concrete + Steel, =	177.8	kips		

Max. Trans. Demand 50 | D/C 28%

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SUBJECT: Pier 3 Shear Key at Top (EXT)
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8.3 Concrete pryout due to Shear at edge dowel Vcp

Vcp=kcp x Ncb

kcp=	2		
Ncb=	49.0	kips	
Factor, φ=	1		
Factored Vcp=	97.9	kips	
Factored Demand, Vu=	49.5	kips	
D/C=	51%		

ACI 318-14 eq. 17.5.3.1a

8.4 Shear and Tension Interaction

Max Shear D/C, Vdc=	87%		
Max Tension D/C, Tdc=	26%		
	0.897	OK	$Vdc^{5/3} + Tdc^{5/3} < 1.0$

ACI 318-14 R17.6

9.0 Verify Layout of Anchors in Concrete - Group Anchor

9.1 Concrete breakout due to tension - Ncbg

C,a1=	12	in	Min. edge distance in longitudinal direction	
H,ef used.=	8.000	in	Use this for tension breakout calcs	ACI 318-14 cl17.4.2.3
H,ef prov.=	36	in	embedment length into concrete	
A, Nc=	14976	in2	Effective concrete area around the edge anchor	ACI 318-14 cl17.4.2.1c
A, Nco=	576	in2	Projected area of single anchor = 9 x hef x hef	ACI 318-14 Fig R17.4.2.1
Nb	39	kips	Basic breakout strength of single anchor in tension	ACI 318-14 CL17.4.2.2a
kc	24		cast in anchor	
gamma a	1		aggregate factor	
sqrt f'c	72.111			
hef	8.000	in		
ψ, ed,N=	1		ca, min = 12in $\geq$ 1.5 h,ef = 12in	
ψ, ec,N=	1		cast-in-anchors	
ψ, cp,N=	1		cast-in-anchors	
ψ, c,N=	1.25			
Factor=	1.00			

			Max. Tension Demand	D/C
Factored Concrete breakout, Ncb=	1018	kips	> 118	12%

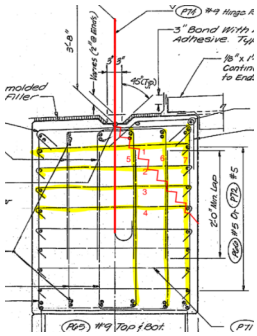
9.2 Concrete breakout due to Shear at edge dowel Vcbg - Shear in bridge long. Direction

Shear in bridge long. direction

Ca2=	28.5	in	min. distance to edge parallel to edge	
Ca1=	12	in	min. distance to edge perpendicular to edge	
s=	12.0	in		
Avco=	3655.125	in2		ACI 318 eq 17.5.2.1c
le=	10.160	in	le=he=load bearing length of dowel	
da=	1.128	in		diameter of dowel
sqrt f'c=	72.11			
Vb=	126.60	kips		ACI 318 eq 17.5.2.2a
Limit Vb=	98.74	kips		ACI 318 eq 17.5.2.2b
Use limit				
he=	18	in	note that the actual embedment is 40"	effective embedment for shear breakout = 1.5 x Ca1
Avc=	12339	in2	group of anchors	
ψ, ec,V=	1			
ψ, ed,V=	1		Ca2 24" > 1.5Ca1 18"	
ψ, c,V=	1.2		Cracking at EXT loads	
ψ, h,V=	1		ha=1.5ca	

Shear Resistance Factor, φ =	1	
Factored Concrete breakout, Vcb=	400.0	kips

Max. long. Demand D/C: 2526 | 631%
ACI 318 CL17.5.2.1b



Check for main wall reinforcement at wall face to resist concrete breakout

Bar #	4		atleast 7 sets of #4 stirrups at 18" spa. throughout
Qty.	243		
Area of steel, As=	49	in2	
f'ye	68.000	ksi	expected steel yield strength
Nominal Steel Strength, Vs=	3300.26667	kips	=f'ye x As
Factor, φ=	1		
Factored steel strength, f x Vs=	3300	kips	

Max. Trans. Demand D/C: 2526 | 68%

Total Factored Strength, Concrete + Steel, = 3700.3 kips

Hence, stirrups reinforcement will prevent concrete breakout in bridge long. Direction

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PROJECT: I-405 R2B Final Design - BR 17.7
SUBJECT: Pier 3 Shear Key at Top (EXT)
BY: RSGR
CHK: LDLI

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10.0 Interface shear resistance (AASHTO LRFD BDS 2017, 5.7.4)

This happens in superstructure where f'c = 5 ksi

- For normal weight concrete placed monolithically:

$c = 0.40$ ksi
$\mu = 1.4$
$K_1 = 0.25$
$K_2 = 1.5$ ksi

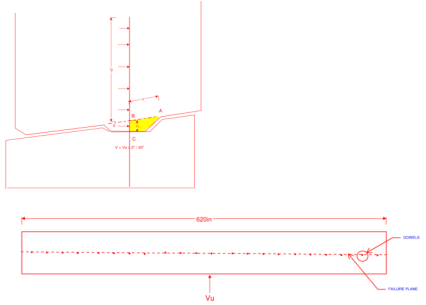
cohesion factor	c=	0.4	ksi	
Friction factor	u=	1.4		
Fraction of concrete strength	K1=	0.25		
Limiting interface shear resistance	K2=	1.5	ksi	
	b	5	in	(3" on top and 2" in height)
	L	620	in	Length of shear key failure plane
Area of concrete in interface	Acv=	3100	in2	b x L
Nominal Shear Resistance of interface	Vni=	1240	kips	c x Acv
	fye=	0	ksi	Rebar Steel strength (nominal)
	Avf=	0	in2	Steel reinforcement area in shear key
	Pc=	0	kips	No compression assumed
	Factor=	1		
	Factored resistance=	1240	kips	

	Max. Demand		D/C
>	495	=2526 x 2"/10.2"	40%

Max Limits for resistance

					Max. Demand	D/C
AASHTO LRFD BDS 5.7.4.3-4	V1	3875	kips	>	495	13%
AASHTO LRFD BDS 5.7.4.3-5	V2	4650	kips			

Hence, concrete shear key shall will not fail and the failure will be governed by checks in section 6 to 9



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PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I9

CALC BOOK COMPLETION DATE: **2021 Dec 05**

Title: Miscellaneous

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I9-0	-	I9-0	Calc Register (this sheet)	N/A	N/A	
2	I9-1	-	I9-1	Expansion Joint	FCCA	MWBM	
3	I9-2	-	I9-1				
4	I9-2	-	I9-1				
5	I9-2	-	I9-1				
6	I9-2	-	I9-1				
7	I9-2	-	I9-1				
8	I9-2	-	I9-1				
9	I9-2	-	I9-1				
10	I9-2	-	I9-1				
11	I9-2	-	I9-1				
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16	I9-2	-	I9-1				
17	I9-2	-	I9-1				
18	I9-2	-	I9-1				
19	I9-2	-	I9-1				
20	I9-2	-	I9-1				
		-					
		-					
		-					

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PROJECT I-405 R2B: Bridge 17.7 CONT A207833

SUBJECT Compression Seal Design

CHECKED BY MWBM DATE 11/1/2021 CALCULATIONS BY FCCA DATE 11/1/2021

Purpose: Determine if 2.5" Compression Seal can accommodate bridge uniform temperature, temperature gradient, shrinkage and creep translations.

Summary: 2 1/2" Compression Seal can be used at Bridge 17.7

Geometry

w_{CS} = 2.5 in Compression Seal Width

Parameters for Temperature Translation Calculations (WSDOT BDM 9.1.2-B)

T_{avg} =	64	deg	
T_L =	10	deg	AASHTO BDS Table 3.12.2.1-1, Moderate climate, Concrete
T_U =	80	deg	AASHTO BDS Table 3.12.2.1-1, Moderate climate, Concrete
LF =	1.2		Max. Load Factor for uniform temperature. AASHTO BDS Table 3.4.1-1
T_{min} =	3	deg	WSDOT 9.1.2-B
T_{max} =	87	deg	WSDOT 9.1.2-B

Translations obtained from bridge model, see calcbook I1: Loading/Analysis

Translation	Pier 1	Pier 3	
$\Delta_{S+C,nor}$ (in)	0.012	0.024	Creep + Shrinkage translation, normal
$\Delta_{TU,nor}$ (in)	0.320	0.515	Factored uniform temperature translation, normal
$\Delta_{TG,nor}$ (in)	0.064	0.078	Factored temperature gradient translation, normal
$\Delta_{S+C,par}$ (in)	0.003	0.000	Creep + Shrinkage translation, parallel
$\Delta_{TU,par}$ (in)	0.013	0.002	Factored uniform temperature translation, parallel
$\Delta_{TG,par}$ (in)	0.011	0.002	Factored temperature gradient translation, parallel

Total Translation

$\Delta_{T,nor,P1}$ =	0.396	in	Normal Shrinkage + Creep + Uniform Temperature + Temperature Gradient translation, Pier 1
$\Delta_{T,par,P1}$ =	0.027	in	Parallel Shrinkage + Creep + Uniform Temperature + Temperature Gradient translation, Pier 1
$\Delta_{T,nor,P3}$ =	0.617	in	Normal Shrinkage + Creep + Uniform Temperature + Temperature Gradient translation, Pier 3
$\Delta_{T,par,P3}$ =	0.004	in	Parallel Shrinkage + Creep + Uniform Temperature + Temperature Gradient translation, Pier 3

Motion Range Calculations

Reference [WSDOT BDM]:				Design Example 9.1.3-A							
-	Skew* (deg)	$\Delta_{T,nor}$ (in)	$\Delta_{T,par}$ (in)	$W_{1\text{-normal}}$ (in)	$W_{2\text{-par}}$ (in)	$W_{3\text{-min}}$ (in)	Min. Comp. Seal Width (in)	D/C	Const. Width @ 40° (in)	Const. Width @ 64° (in)	Const. Width @ 80° (in)
Pier 1	0	0.396	0.027	0.880	0.123	1.12	1.12	0.45	1.61	1.50	1.43
Pier 3	0	0.617	0.004	1.371	0.018	1.72	1.72	0.69	1.67	1.50	1.39

Notes:

* Translations obtained from bridge model are normal/parallel to compression seals at Piers 1 and 3, respectively. Therefore, skew can be taken as zero.

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PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I10

CALC BOOK COMPLETION DATE: **2021 Nov 28**

Title: Retaining Wall 03.66L

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I10-0	-	I10-0	Calc Register (this sheet)	N/A	N/A	
2	I10-1	-	I10-2	Geometry and Input	CEKH	MWBM	
3	I10-3	-	I10-7	Demands	CEKH	MWBM	
4	I10-8	-	I10-23	Design checks of 03.66L wall	CEKH	MWBM	
5	I10-24	-	I10-27	Check of Bridge 18W Pier 1 Footing	CEKH	MWBM	
6	I10-28	-	I10-29	Check of Bridge 18W Widening	CEKH	MWBM	
7	I10-30	-	I10-35	Check of Bridge 17.7 Existing Pier 1 Footing	CEKH	MWBM	
		-					
		-					
		-					

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PROJECT WSDOT I-405 R2B CONT A207833

SUBJECT Retaining Wall 03.66L Design PAGE I10-1

CHECKED BY MWBM DATE 2021DEC03 CALCULATIONS BY CEKH DATE 2021NOV20

WSDOT I-405
Retaining Wall 03.66L Design

By: CEKH

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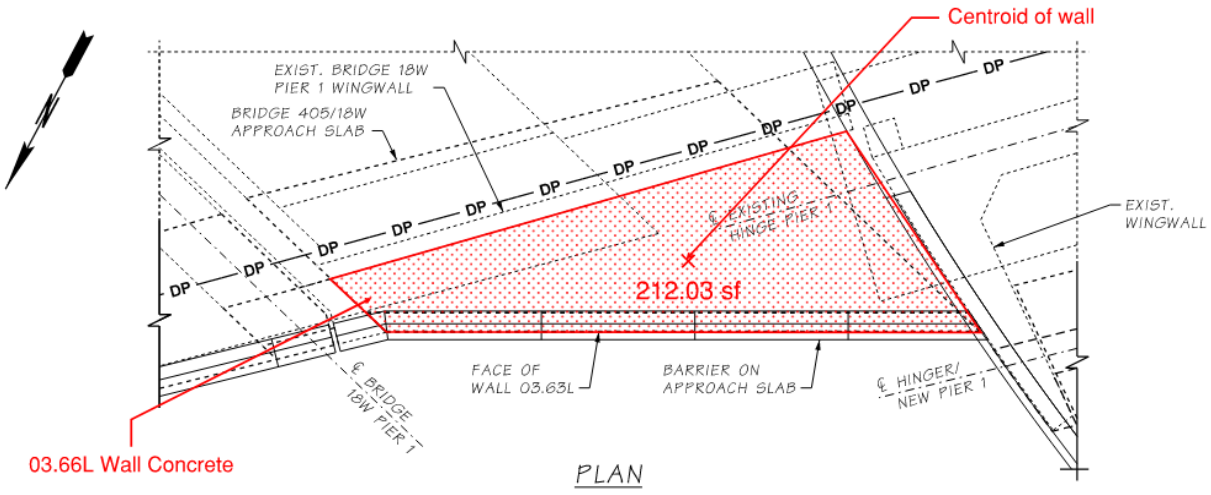
PROJECT WSDOT I-405 R2B CONT A207833

SUBJECT Retaining Wall 03.66L Design PAGE I10-2

CHECKED BY MWBM DATE 2021DEC03 CALCULATIONS BY CEKH DATE 2021NOV20

1. Geometry and Input

The space under the Bridge 18W approach slab extension past the Pier 1 Wingwall will be fillled with concrete. This concrete will extend from Bridge 18W Pier 1 to Bridge 17.7 existing wingwall, as shown in the Figure below.



It is expected that the "backfill" concrete and the wall itself will be monolithic. The resulting concrete wall will not rest on a separate footing. Instead, the wall will be connected to the existing Bridge 18W wingwall and existing Bridge 17.7 wingwall through dowels and to the widened portion of Bridge 18W Pier 1.

Area of concrete: $A_{conc} := 212.0\ ft^2$

Concrete wall height: $h := 23.54\ ft$

Concrete wall length: $l := 31\ ft + 1.625\ in = 31.14\ ft$

Unit weight of concrete: $\gamma_{conc} := 0.155\ \frac{kip}{ft^3}$

Unit weight of gravel backfill: $\gamma_{fill} := 0.1\ \frac{kip}{ft^3}$

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PROJECT WSDOT I-405 R2B CONT A207833

SUBJECT Retaining Wall 03.66L Design PAGE I10-3

CHECKED BY MWBM DATE 2021DEC03 CALCULATIONS BY CEKH DATE 2021NOV20

2. Demands

2.1 Self weight of 03.66L wall

Self weight of concrete wall: $W_{wall} := h \cdot A_{conc} \cdot \gamma_{conc} = 773.524\ kip$

2.2 Backfill Pressure

As mentioned previously, the wall face and the "backfill" concrete is expected to be poured monolithically. Therefore, no hydrostatic or backfill pressure will be experienced due to wet concrete behind the wall face.

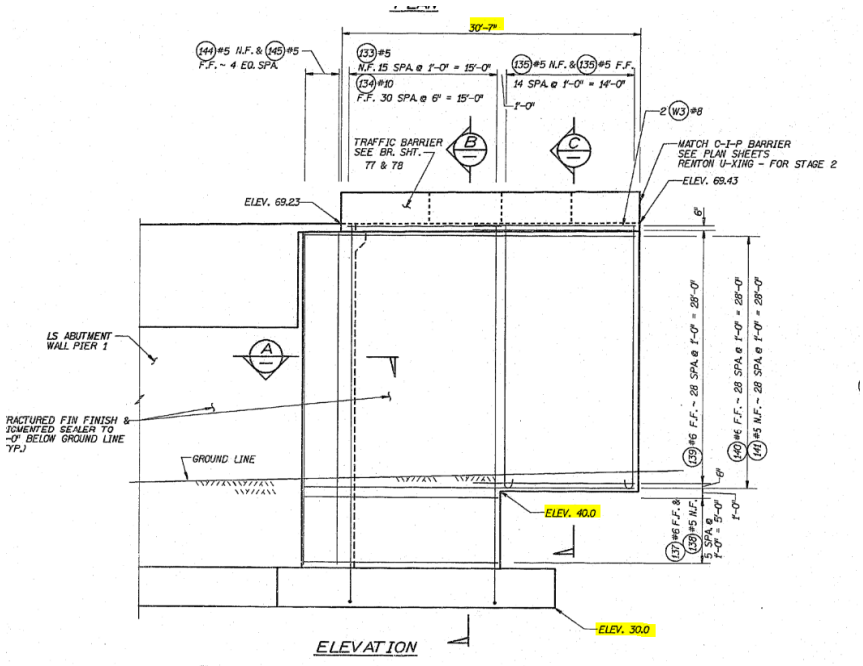
2.3 Seismic Load

Need to consider the inertial response of the wall.

Seismic horizontal acceleration coeffcient: $k_h := 0.56$ *As instructed by Ben Upsall in email dated 14 May 2021*

Horizontal seismic inertial force: $H_{IR.wall} := W_{wall} \cdot k_h = 433.17\ kip$

Need to consider the inertial response of Bridge 18W wingwall:



Length of wingwall over footing: $L_1 := 20\ ft + 3.25\ in = 20.27\ ft$

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PROJECT WSDOT I-405 R2B CONT A207833

SUBJECT Retaining Wall 03.66L Design PAGE I10-4

CHECKED BY MWBM DATE 2021DEC03 CALCULATIONS BY CEKH DATE 2021NOV20

Height of wingwall over footing: $H_1 := 69.23\ ft - 34\ ft = 35.23\ ft$

Average thickness of wingwall over footing: $t_1 := \frac{10\ in + \left(10\ in + 0.1 \cdot H_1\right)}{2} = 2.59\ ft$

Length of wingwall past footing: $L_2 := 30\ ft + 7\ in - L_1 = 10.31\ ft$

Height of wingwall past footing: $H_2 := 69.43\ ft - 40\ ft = 29.43\ ft$

Average thickness of wingwall past footing: $t_2 := \frac{10\ in + \left(10\ in + 0.1 \cdot H_2\right)}{2} = 2.3\ ft$

Self weight of wingwall: $W_{wingwall} := \left(\left(H_1 \cdot L_1 \cdot t_1\right) + \left(H_2 \cdot t_2 \cdot L_2\right)\right) \cdot \gamma_{conc} = 395.65\ kip$

Horizontal seismic inertial force: $H_{IR.wingwall} := W_{wingwall} \cdot k_h = 221.56\ kip$

Need to consider the active seismic earth pressure of gravel backfill retained by Bridge 18W:

Horizontal seismic active force: $P_{AE} := 0.5 \cdot \gamma_{fill} \cdot h^2 \cdot k_h \cdot \left(L_1 + L_2\right) = 474.52\ kip$

Chand Khushiram

From: Ben Upsall <bupsall@geoengineers.com>
Sent: Friday, May 14, 2021 2:58 PM
To: Matt Baughman
Cc: Ivy A. Harmon; Mehdi Dastfan; Chand Khushiram; A207833 - WSDOT I-405 R2B
Subject: RE: I-405 R2B : Wall 03.66L

I-405

Follow Up Flag: Follow up
Flag Status: Completed

And those are 0.56g and 0.27g for SEE and FEE.

Ben Upsall
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From: Ben Upsall
Sent: Friday, May 14, 2021 1:09 PM
To: Matt Baughman <mwbm@cowi.com>
Cc: Ivy A. Harmon <iharmon@geoengineers.com>; Mehdi Dastfan <medn@cowi.com>; Chand Khushiram <CEKH@cowi.com>; A207833 - WSDOT I-405 R2B <A207833-project@cowi.com>
Subject: RE: I-405 R2B : Wall 03.66L

Matt,
For a rigid wall like that you would use the full PGA from the PSHA from Hart Crowser's report for the Kh.

Ben

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From: Matt Baughman <mwbm@cowi.com>
Sent: Wednesday, May 12, 2021 1:03 PM
To: Ben Upsall <bupsall@geoengineers.com>
Cc: Ivy A. Harmon <iharmon@geoengineers.com>; Mehdi Dastfan <medn@cowi.com>; Chand Khushiram <CEKH@cowi.com>; A207833 - WSDOT I-405 R2B <A207833-project@cowi.com>
Subject: RE: I-405 R2B : Wall 03.66L

[EXTERNAL]

Ben, regarding the email string below, we've got a few updates for you. We're planning to support this wall off the existing Bridge 18W south abutment and existing Bridge 17.7 west abutment. As such, we won't need any bearing pressures or site class definition as previously requested. However, can you confirm the kh value we should be using? Does this change any recommendations you've given for Bridge 18W, or do you see anything else we would need to coordinate on? (I have in my head that with this system, we shouldn't need any Geotech input beyond what you've already given for new 18W-P1 and existing 17.7-P1.)

Thank you,
Matt

Matthew D. Baughman, P.Eng, P.E., S.E.
Senior Project Manager
COWI Bridge
Direct: 206 748 4034

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From: Matt Baughman
Sent: Tuesday, April 20, 2021 7:37 AM
To: Ben Upsall <bupsall@geoengineers.com>
Cc: 'Ivy A. Harmon' <iharmon@geoengineers.com>; Mehdi Dastfan <medn@cowi.com>; Chand Khushiram <CEKH@cowi.com>; A207833 - WSDOT I-405 R2B <A207833-project@cowi.com>
Subject: RE: I-405 R2B : Wall 03.66L

I forgot to attach the sketch.

Matt

From: Matt Baughman
Sent: Tuesday, April 20, 2021 7:24 AM
To: Ben Upsall <bupsall@geoengineers.com>
Cc: Ivy A. Harmon <iharmon@geoengineers.com>; Mehdi Dastfan <medn@cowi.com>; Chand Khushiram <CEKH@cowi.com>; A207833 - WSDOT I-405 R2B <A207833-project@cowi.com>
Subject: I-405 R2B : Wall 03.66L

Hey Ben, can you provide us geotechnical information for the design of Retaining Wall 03.66L? To start, we are looking for:

- Bearing Pressure/Capacity. Ground is around EL 50, so bottom of footing would be around EL 47. We believe this would be on backfill placed after the original construction of the Bridge 18W south abutment and Bridge 17.7 west abutment (just outside the 18W wingwall built between the two).
- Soil Site Class
- Kh

We still would be looking at two options:

- Just a mass concrete wall (fill the whole trapezoidal area with concrete). This is FLJV's preference if it works.
- CIP wall with LDCC / CLSM filling in the rest of the trapezoidal area

Either way, wall would likely be some combination of bearing on the soil and anchored to the adjacent abutments.

Thanks,
Matt

Matthew D. Baughman, P.Eng, P.E., S.E.
Senior Project Manager
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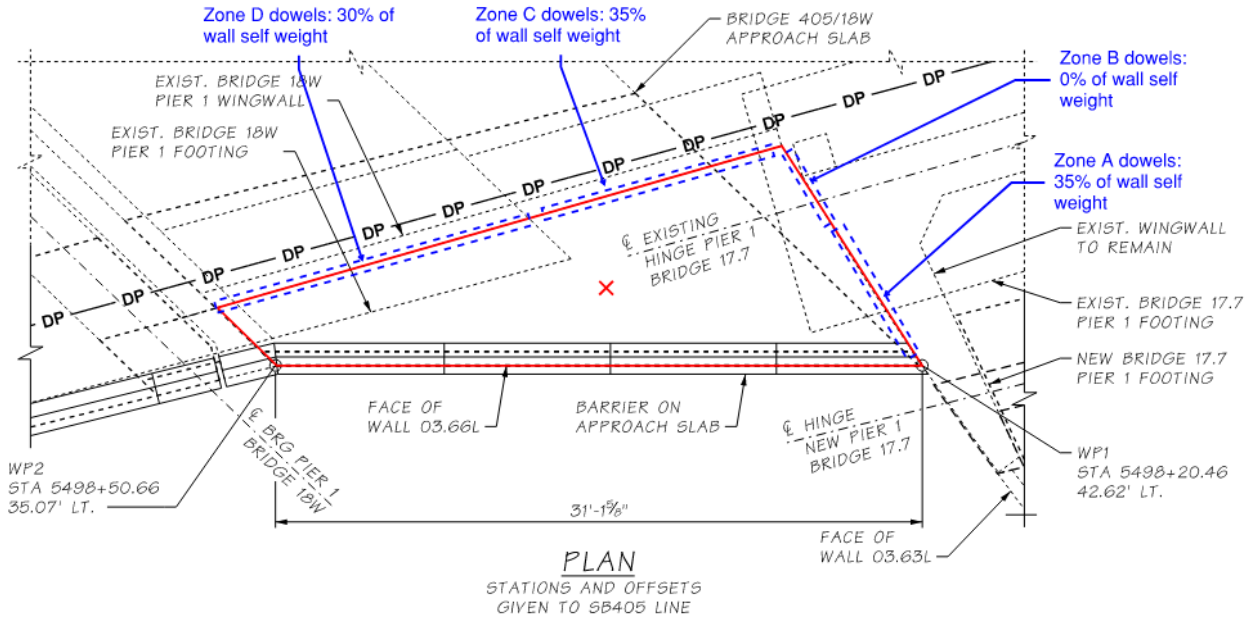
3. Design checks of 03.66L wall

3.1 Load distribution within 03.66L wall

3.1.1 Vertical load distribution

The distribution of vertical loads (i.e. the self weight of the wall) in the 03.66L wall will be considered as shown below.

The vertical loads have to be transferred to 3 surfaces, namely Bridge 18W abutment seat, Briedge 18W wingwall and Bridge 17.7 wingwall. The Bridge 18W abutment seat is only nominally connected to the 03.66L wall and hence will be not be assigned any loads. To determine the distribution between the remaining two surfaces, the ratio of the distance between the centre of gravity and the surfaces will be used.



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SUBJECT Retaining Wall 03.66L Design PAGE I10-9

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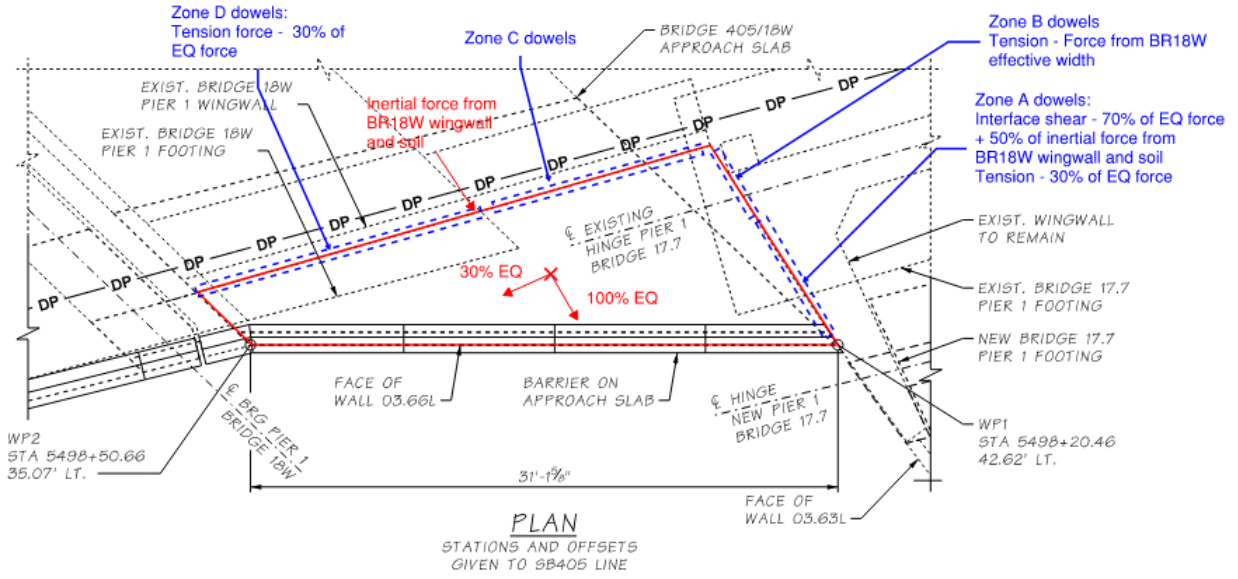
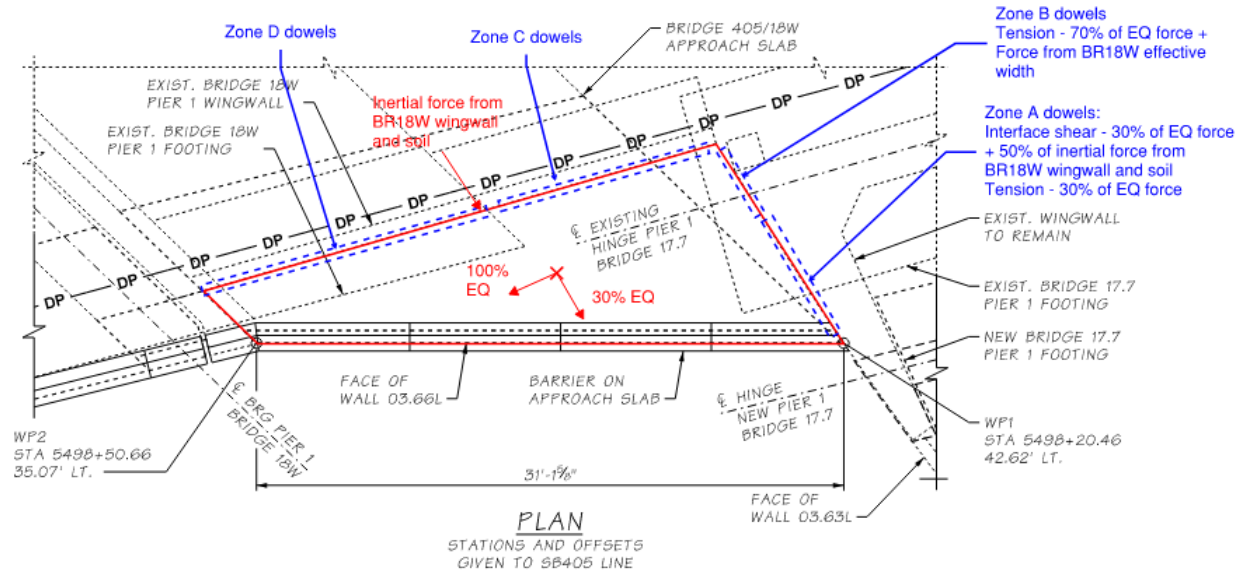
3.1.2 Lateral load distribution

Two cases will be considered for the lateral load distribution, as shown below. Under lateral loads, Bridge 17.7 is considered the more rigid system than the Bridge 18W wingwall. Hence, it is assumed than 70% of the lateral loads are distributed to the dowels along the Bridge 17.7 wingwall and the remaining 30% is transferred to the Bridge 18W wingwall dowels. It should also be noted that the Extreme Limit State is a plastic limit state and there will be significant redistribution of the force which will mobilize more dowels. The current distribution assumed is therefore conservative.





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SUBJECT Retaining Wall 03.66L Design PAGE I10-10

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3.2 Dowels into Bridge 17.7 Wingwall

3.2.1 Interface shear check

Zone A dowels

Per the load distributions in Section 3.1,it is clear that the following interface shear loads are critical for the Zone A dowels:

- 35% of self weight of the 03.66L wall
- 70% of inertial force from 03.66L wall
- 25% of the inertial force from Bridge 18W wingwall
- 50% of inertial force coming from the soil retained by Bridge 18W wingwall

It should be noted that a further factor of 0.5 has been applied to the 50% interial force from the Bridge 18W wingwall (i.e.25% of the inertial force has been considered). This in accordance with AASHTO LRFD 2017 Article 11.6.5.1 (extract shown below).

To investigate the wall stability considering the combined effect of P_{AE} and P_{IR} and considering them not to be concurrent, the following two cases should be investigated:

- combine 100 percent of the seismic earth pressure P_{AE} with 50 percent of the wall inertial force P_{IR} and
- combine 50 percent of P_{AE} but no less than the static active earth pressure force (i.e., F_1 in Figure 11.10.5.2-1), with 100 percent of the wall inertial force P_{IR}.

The most conservative result from these two analyses should be used for design of the wall.

Interface shear force from self weight: $V_{ui.sw} := 0.35 \cdot W_{wall} = 270.7335\ kip$

Interface shear force from lateral seismic load: $V_{ui.EQ} := 0.7 \cdot H_{IR.wall} + 0.25 \cdot H_{IR.wingwall} + 0.5 \cdot P_{AE} = 595.8734\ kip$

Total interface shear: $V_{ui} := \sqrt{V_{ui.sw}^2 + V_{ui.EQ}^2} = 654.494\ kip$

Conservatively, we will not rely on any cohesion component from the concrete itself.

$\mu := 0.6$

The dowels in the Bridge 17.7 wingwall will consist of #6 dowels.

Area of bar: $A_b := 0.44\ in^2$

Yield strength of reinforcement: $f_y := 60\ ksi$

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Compression on interface shear surface: $P_c := 0\ kip$

Concrete compressive strength: $f_{cp} := 4000\ psi$

Factored interface shear resistance per dowel:

$\varphi := 0.9$ $V_{ni} := \varphi \cdot \mu \cdot \left(A_b \cdot f_y + P_c \right) = 14.256\ kip$

Number of dowels required: $n_{req.shear} := \frac{V_{ui}}{V_{ni}} = 45.91$

In Zone A, we can use the wall face reinforcement dowels (#6@6"). Add an additional layer of #6 dowels @ 1' spacing vertically.

Number of dowels provided: $n_{prov} := \left(\frac{h}{6\ in} + 1 \right) + \left(\frac{h}{12\ in} + 1 \right) = 72.62$

$CoD := \frac{n_{prov}}{n_{req.shear}} = 1.5818$

3.2.2 Tensile capacity check

Zone A dowels

Per the load distributions in Section 3.1,it is clear that the following tensile load is critical for the Zone A dowels:
- 30% of inertial force from 03.66L wall

Tension force on Zone A dowels: $T_f := 0.3 \cdot H_{IR.wall} = 129.952\ kip$

Factored tensile resistance per dowel:

$\varphi = 0.9$ $T_r := \varphi \cdot A_b \cdot f_y = 23.76\ kip$

Number of dowels required: $n_{req.tens} := \frac{T_f}{T_r} = 5.469$

$CoD := \frac{n_{prov}}{n_{req.tens}} = 13.2776$

The dowels in Zone A are used simultaenously as interface shear reinforcement and tension reinforcement. Checking the combined demand on the Zone A dowels we obtain:

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$n_{req.total} := n_{req.shear} + n_{req.tens} = 51.379$

$CoD := \frac{n_{prov}}{n_{req.total}} = 1.4134$

Zone B dowels

Per the load distributions in Section 3.1,it is clear that the following loads are critical for the Zone B dowels:

- 70% of inertial force from 03.66L wall
- Tension force from effective width of Bridge 18W
- Tension force from earth pressure of retained soil by Bridge 17.7 wingwall

The 03.66L wall connectes Bridge 18W and Bridge 17.7 abutments. In resisting lateral loads, Bridge 17.7 is considered the more rigid system. Hence, the inertial force from an effective width of Bridge 18W is effectively carried by Bridge 17.7. This inertial force has to transfer from Bridge 18W through the 03.66L wall to Bridge 17.7.

Assuming that the effective width of Bridge 18W is equivalent to 3 times the thickness of Bridge 18W (i.e. the thickness of the abutment seat),

Effective width of Br18W: $b_{eff.18W} := 3 \cdot 5\ ft = 15\ ft$

BR18W backwall thickness: $t_{bwall} := 1.25\ ft$

BR18W backwall height: $h_{bwall} := 8.25\ ft$

Abutment seat thickness: $t_{seat} := 5\ ft$

Abutment seat height: $h_{seat} := 27.03\ ft$

Self weight of effective width of existing BR18W:

$$W_{exist.18W} := \left(t_{bwall} \cdot h_{bwall} + t_{seat} \cdot h_{seat} \right) \cdot b_{eff.18W} \cdot \gamma_{conc} = 338.2\ kip$$

Self weight of BR18W extension:

$W_{ext.18W} := 71.954\ kip$ (See CEKH calculations for Bridge 18W widening checks)

Inertial force from effective width of existing BR18W:

$$H_{IR.exist.18W} := k_h \cdot W_{exist.18W} = 189.39\ kip$$

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Inertial force from BR18W extension:

$$H_{IR.ext.18W} := k_h \cdot W_{ext.18W} = 40.294\ \text{kip}$$

Seismic earth pressure from backfill retained by effective width of BR18W:

$$P_{fill.18W} := 0.5 \cdot \gamma_{fill} \cdot h_{seat}^{\ 2} \cdot k_h \cdot \left(b_{eff.18W}\right) = 306.861\ \text{kip}$$

Force from isolation bearings:

$H_{brg} := 2 \cdot 22\ \text{kip} = 44\ \text{kip}$ (Maximum force per isolation bearing obtained from PYSM. 2 isolation bearings considered within effective width)

Total force from BR18W to be transferred to BR17.7:

$$H_{18W} := H_{IR.exist.18W} + H_{IR.ext.18W} + 0.5 \cdot P_{fill.18W} + H_{brg} = 427.117\ \text{kip}$$

Length of Bridge 17.7 wingwall: $l_{wwall} := 15\ \text{ft} - \left(1\ \text{ft} + 4.5\ \text{in}\right) = 13.625\ \text{ft}$

Seismic earth pressure from backfill retained by BR17.7 wingwall:

$$P_{fill.17.7} := 0.5 \cdot \gamma_{fill} \cdot h^{2} \cdot k_h \cdot \left(l_{wwall}\right) = 211.401\ \text{kip}$$

Total tensile force in 03.66L wall reinforcement:

$$T_f := H_{18W} + 0.7 \cdot H_{IR.wall} + 0.5 \cdot P_{fill.17.7} = 836.039\ \text{kip}$$

In Zone B, assume we are using a single layer of #8 bars at 1foot spacing, dowelled with 2-#6 bars into Bridge 17.7.

Tensile resistance of #8 bars:

$\varphi = 0.9$ $T_r := \varphi \cdot \left(\frac{h}{1\ \text{ft}} + 1\right) \cdot 0.79\ \text{in}^2 \cdot f_y = 1046.876\ \text{kip}$

$$CoD := \frac{T_r}{T_f} = 1.2522$$

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Check of Bridge 18W wingwall rebar

For the force from Bridge 18W to be transferred to Bridge 17.7, it has to be carried to the 03.66L wall through the Bridge 18W wingwall rebar.

The Bridge 18W wingwall rebar consists of 28 #5 bars on the near face and 28 #6 bars on the far face.

Tensile resistance of wingwall rebar:

$\varphi = 0.9$ $T_r := \varphi \cdot \left(28 \cdot 0.31\ \text{in}^2 + 28 \cdot 0.44\ \text{in}^2\right) \cdot f_y = 1134\ \text{kip}$

$CoD := \frac{T_r}{H_{18W}} = 2.655$

It should be noted that the wingwall rebar are already carrying some stress from resisting soil pressure behind the wingwall. However, from inspection, there is more than enough reserve capacity to accomodate that from the high CoD ratio above.

Check of Bridge 17.7 Wingwall Rebar

We also need to verify if the reinforcement in the Bridge 17.7 pier wall is able to carry this tension force. The reinforcement in the Bridge 17.7 pier wall consists of #7 bars in each face spaced at 1 foot (total of 47 #7 bars).

Tensile resistance of #7 bars:

$\varphi = 0.9$ $T_r := \varphi \cdot 47 \cdot 0.6\ \text{in}^2 \cdot f_y = 1522.8\ \text{kip}$

$CoD := \frac{T_r}{T_f} = 1.8214$

Check of splicing between 03.66L wall bar and Bridge 18W wingwall rebar

Since we are effecively splicing the reinforcement in the 03.66L wall (#8 bars) to the bars in the Bridge 18W wingwall (2 layers of #7 bars), we need to provide enough reinforcement to anchor the concrete struts forming over the splice length between the bars.

Class B splice length for a #8 bar: $l_{ClassB.splice} := 3\ \text{ft} + 2\ \text{in}$

Offset between bars being spliced: $s := 3\ \text{ft} + 6\ \text{in}$

Assuming a 45 degree concrete strut can form between the bars, the total splice length required is:

$l_{splice} := l_{ClassB.splice} + 2 \cdot s = 10.167\ \text{ft}$

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Tensile force being transferred:

$H_{18W} = 427.12\ \text{kip}$

The tension tie is provided by the #5 dowels into the Bridge 18W wingwall.

Tensile resistance per #5 bar:

$\varphi = 0.9$ $\quad T_r := \varphi \cdot 0.31\ \text{in}^2 \cdot f_y = 16.74\ \text{kip}$

Number of dowels required: $n_{req.splice} := \frac{H_{18W}}{T_r} = 25.515$

Over the splice length between the bars, provide layers of 5- #5 bars at a spacing of 1' vertically.

Number of dowels provided: $n_{prov} := 5 \cdot \left(\frac{h}{12\ \text{in}} + 1 \right) = 122.7$

$CoD := \frac{n_{prov}}{n_{req.splice}} = 4.809$

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3.3 Dowels into Bridge 18W Wingwall

3.3.1 Interface shear check

Zone C dowels

Per the load distribution in Section 3.1, the critical interface shear load on the Zone C dowels is 35% of self weight of the 03.66L wall

Interface shear force from self weight: $V_{ui.sw} := 0.35 \cdot W_{wall} = 270.7335\ kip$

Conservatively, we will not rely on any cohesion component from the concrete itself.

$\mu := 0.6$

Area of #5 bar: $A_b := 0.31\ in^2$

Yield strength of reinforcement: $f_y := 60\ ksi$

Compression on interface shear surface: $P_c := 0\ kip$

Concrete compressive strength: $f_{cp} := 4000\ psi$

Factored interface shear resistance per dowel:

$\varphi := 0.9$ $V_{ni} := \varphi \cdot \mu \cdot \left(A_b \cdot f_y + P_c \right) = 10.044\ kip$

Number of dowels required: $n_{req.shear} := \frac{V_{ui.sw}}{V_{ni}} = 26.955$

In Zone C, we have 5- #5 bars at a spacing of 1' vertically.

Number of dowels provided: $n_{prov} := 5 \cdot \left(\frac{h}{12\ in} + 1 \right) = 122.7$

$CoD := \frac{n_{prov}}{n_{req.shear}} = 4.5521$

Note that the Zone C dowels are used simultaenously for interface shear from the self weight of the wall and to provide the tie force for splicing the #8 bars in the wall to the Bridge 18W wingwall rebar.

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Total number of dowels required: $n_{req.total} := n_{req.splice} + n_{req.shear} = 52.469$

Number of dowels provided: $n_{prov} := 5 \cdot \left(\frac{h}{12\ in} + 1 \right) = 122.7$

$CoD := \frac{n_{prov}}{n_{req.total}} = 2.3385$

Zone D dowels

Per the load distribution in Section 3.1, the critical interface shear load on the Zone D dowels is 30% of self weight of the 03.66L wall

Interface shear force from self weight: $V_{ui.sw} := 0.30 \cdot W_{wall} = 232.0573\ kip$

Conservatively, we will not rely on any cohesion component from the concrete itself.

$\mu := 0.6$

Area of #5 bar: $A_b := 0.31\ in^2$

Yield strength of reinforcement: $f_y := 60\ ksi$

Compression on interface shear surface: $P_c := 0\ kip$

Concrete compressive strength: $f_{cp} := 4000\ psi$

Factored interface shear resistance per dowel:

$\varphi := 0.9$ $V_{ni} := \varphi \cdot \mu \cdot \left(A_b \cdot f_y + P_c \right) = 10.044\ kip$

Number of dowels required: $n_{req.shear} := \frac{V_{ui.sw}}{V_{ni}} = 23.104$

In Zone D, we have 35-#5 dowels.

Number of dowels provided: $n_{prov} := 35$

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$$CoD := \frac{n_{prov}}{n_{req.shear}} = 1.5149$$

3.3.2 Tensile capacity check

Per the load distribution in Section 3.1, the critical tensile load on the Zone D dowels is 30% of the inertial force of the 03.66L wall.

Tensile force from lateral seismic load: $T_f := 0.3 \cdot H_{IR.wall} = 129.9521\ kip$

Factored tensile resistance of #5 dowel:

$\varphi := 0.9$ $T_r := \varphi \cdot 0.31\ in^2 \cdot f_y = 16.74\ kip$

Number of dowels required: $n_{req.tens} := \frac{T_f}{T_r} = 7.763$

In Zone D, we have 35-#5 dowels.

Number of dowels provided: $n_{prov} := 35$

$$CoD := \frac{n_{prov}}{n_{req.tens}} = 4.5086$$

Note that the Zone D dowels are used simultaenously for interface shear from the self weight of the wall and for the tension force from the inertial force of the 03.66L wall.

Total number of dowels Zone D dowels required: $n_{req.total} := n_{req.shear} + n_{req.tens} = 30.867$

Number of dowels provided: $n_{prov} := 35$

$$CoD := \frac{n_{prov}}{n_{req.total}} = 1.1339$$

3.4 Drying shrinkage reinforcement

We want to ensure that no cracking will occur within the wall due to shrinkage stresses. In the direction perpendicular to Bridge 18W wingwall, the wall is free to contract and hence, there will not be any shrinkage stresses in the concrete. However, in the direction along Bridge 18W wingwall, the wall is restrained from shrinking.

To determine the shrinkage strain in the concrete, we will use Clause 5.4.2.3.3 of AASHTO LRFD 2017.

Volume of wall: $V := A_{conc} \cdot h = 8.624 \cdot 10^{6}\ \text{in}^{3}$

Surface of wall exposed to atmosphere: $S := l \cdot h = 1.055 \cdot 10^{5}\ \text{in}^{2}$

Volume to surface area ratio: $\frac{V}{S} = 81.708\ \text{in}$

$$k_s := \text{Max}\left(1.45 - 0.13 \cdot \frac{\text{Min}\left(\frac{V}{S}, 6\ \text{in}\right)}{\text{in}}, 1.0\right) = 1$$

Relative humidity (%): $H := 80$ (From AASHTO LRFD 2017 Figure 5.4.2.3.3-1)

$$k_{hs} := 2.0 - 0.014 \cdot H = 0.88$$

$$k_f := \frac{5}{1 + 0.8 \cdot 4\ \frac{\text{ksi}}{\text{ksi}}} = 1.19$$

Duration for consideration of shrinkage in days: $t := 10000$

$$k_{td} := \frac{t}{12 \cdot \left(\frac{100 - 4 \cdot 0.8 \cdot 4}{0.8 \cdot 4 + 20}\right) + t} = 0.996$$

Shrinkage strain:

$$\varepsilon_{sh} := k_s \cdot k_{hs} \cdot k_f \cdot k_{td} \cdot 0.48 \cdot 10^{-3} = 0.001$$

Shrinkage stress in rebar:

$$\sigma_{sh} := \varepsilon_{sh} \cdot 29000\ \text{ksi} = 14.517\ \text{ksi}$$

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3.5 Wall front face reinforcement

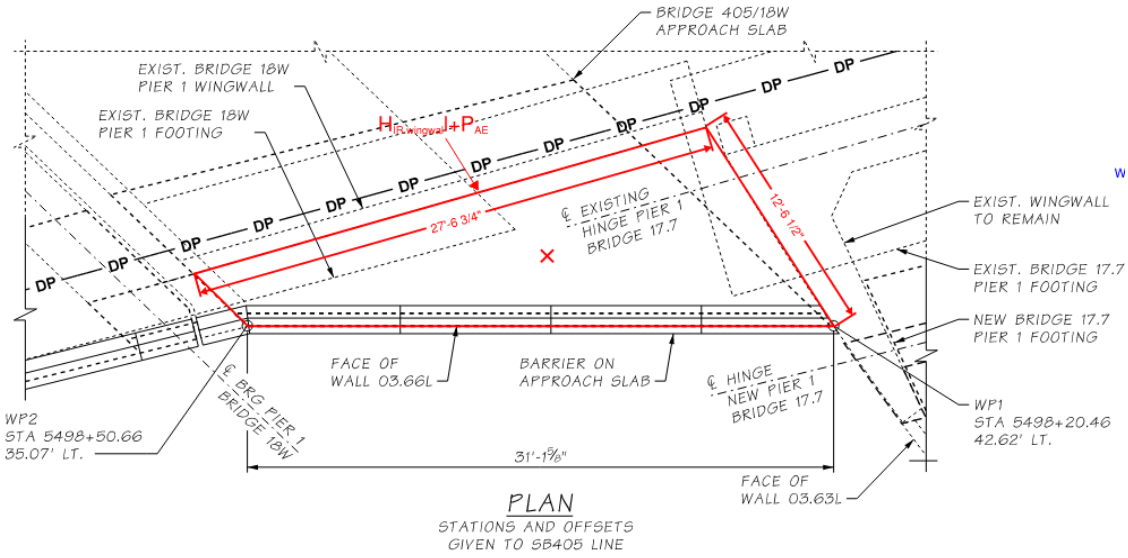
Minimum reinforcement for temp and shrinkage: (per AASHTO LRFD Article 5.10.6)

$$A_{s.min} := \frac{1.30 \cdot h \cdot 99\ in \cdot \frac{1}{2}}{2 \cdot \frac{(h + 99\ in)}{in} \cdot 60}\ \frac{in^2}{ft} = 0.7942\ \frac{in^2}{ft}$$

In each direction, provide #6 bars at 6 inches spacing on the front face of the 03.66L wall.

$A_s := 2 \cdot 0.44\ in^2 = 0.88\ in^2$

Let us consider that the wall is simply supported between Bridge 18W and Bridge 17.7. Under the action of the lateral seismic earth pressure and inertial response from the Bridge 18W wingwall, a tensile force will be generated in the wall face reinforcement.



Bending moment in wall:

$$M_f := \frac{\left(0.5 \cdot H_{IR.wingwall} + 1 \cdot P_{AE}\right) \cdot (27\ ft + 6.75\ in)}{4} = 4033.11\ kip\ ft$$

Thickness wall at critical section:

$$t_{wall} := 8\ ft + 6.25\ in$$

Tensile force:

$$T_f := \frac{M_f}{0.72 \cdot t_{wall}} = 657.39\ kip$$

Number of #6 bars:

$$n := \left(\frac{h}{6\ in}\right) = 47.08$$

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Area of #6 bar: $A_b := 0.44\ in^2$

Factored tensile resistance:

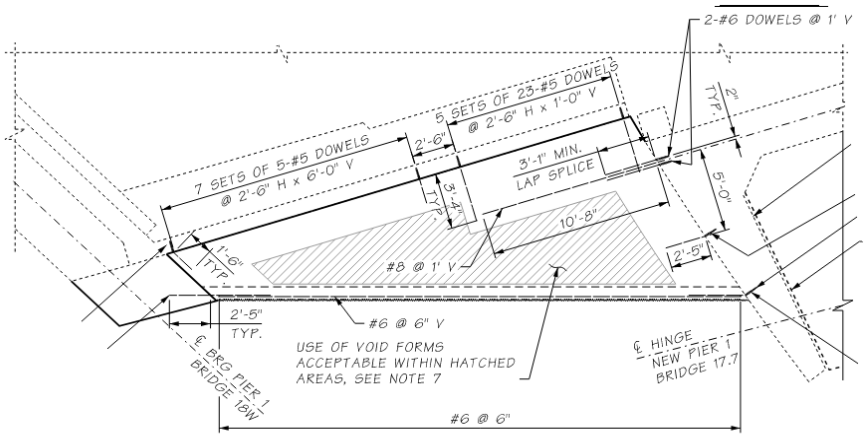
Consider that a portion of the bars are used to also carry the strain from drying shrinkage:

$\varphi := 0.9$ $T_r := \varphi \cdot n \cdot A_b \cdot \left(f_y - \sigma_{sh} \right) = 847.963\ kip$

$CoD := \frac{T_r}{T_f} = 1.2899$

3.6 Use of void forms

The use of void forms is acceptable within the 03.66L wall in order to reduce the overall weight of the wall.



7. THE USE OF VOID FORMS IS ACCEPTABLE WITHIN THE HATCHED AREAS SHOWN. VOID FORMS SHALL BE LOCATED NO CLOSER THAN 4" TO ANY REINFORCING BAR, PROVIDING MINIMUM CONCRETE THICKNESSES OF:
- 1' ALONG FRONT FACE AND AT TOP AND BOTTOM
- 2' ALONG 18W WINGWALL
- 3' ALONG 17.7 WINGWALL AND 18W ABUTMENT WALL
- 4' ALONG 18W WINGWALL NEAR ABUTMENT WALL

IF VOID FORMS ARE USED, CONTRACTOR SHALL SUBMIT MATERIAL SPECIFICATIONS AND INSTALLATION PLAN TO ENGINEER FOR APPROVAL.

The calculations before this section assume that there are no void forms installed in the wall. Hence, the demands on the dowels are conservative. However, we must consider that with the void forms installed, the 1 foot thick front face of the wall does not have a clear load path to the 18W wingwall. In that case, the front face of the wall must be able to transfer the vertical force (self weight) and lateral force (inertial force) to the Bridge 18W abutment seat and the BR17.7 wingwall. This will be checked below.

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Weight of front face of wall: $W_{wall.face} := l \cdot h \cdot 1\ ft \cdot \gamma_{conc} = 113.604\ kip$

Intertial force of front face of wall: $H_{IR.wall.face} := k_h \cdot W_{wall.face} = 63.618\ kip$

Bending moment in wall: $M_f := \frac{H_{IR.wall.face} \cdot l}{4} = 495.194\ kip\ ft$

Tensile force: $T_f := \frac{M_f}{0.72 \cdot 1\ ft} = 687.77\ kip$

Number of #6 bars: $n := \left(\frac{h}{6\ in} \right) = 47.08$

Area of #6 bar: $A_b := 0.44\ in^2$

Factored tensile resistance:

Consider that a portion of the bars are used to also carry the strain from drying shrinkage:

$\varphi := 0.9$ $T_r := \varphi \cdot n \cdot A_b \cdot \left(f_y - \sigma_{sh} \right) = 847.96\ kip$

$CoD := \frac{T_r}{T_f} = 1.2329$

The vertical force from the self weight of the front face of wall needs to be carried by the #6 dowels at 6 inches in the Bridge 18W abutment seat and in the Bridge 17.7 wingwall.

Interface shear force from self weight: $V_{ui.sw} := 0.5 \cdot W_{wall.face} = 56.8019\ kip$

Conservatively, we will not rely on any cohesion component from the concrete itself.

$\mu := 0.6$

Area of #6 bar: $A_b := 0.44\ in^2$

Yield strength of reinforcement: $f_y := 60\ ksi$

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Compression on interface shear surface: $P_c := 0\ kip$

Concrete compressive strength: $f_{cp} := 4000\ psi$

Factored interface shear resistance per dowel:

$\varphi := 0.9$ $V_{ni} := \varphi \cdot \mu \cdot \left(A_b \cdot f_y + P_c \right) = 14.256\ kip$

Number of #6 bars: $n := \left(\frac{h}{6\ in} \right) = 47.08$

Number of dowels required: $n_{req.shear} := \frac{V_{ui.sw}}{V_{ni}} = 3.984$

$CoD := \frac{n_{prov}}{n_{req.shear}} = 8.7842$

The #6 dowels carry the tension from lateral loads and shear from vertical loads. The combined capacity over demand ratio is presented below.

$CoD\left(combined\right) := \frac{1}{\frac{T_f}{T_r} + \frac{n_{req.shear}}{n_{prov}}} = 1.0812$

Therefore, it can be concluded that the 1 foot thick front face of wall has adequate capacity to transfer the forces to the adjacent structures and the use of void forms within the wall is acceptable.

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4. Check of Bridge 18W Pier 1 Footing

$L_{footing} := 47.5\ ft + 30.5\ ft = 78\ ft$

$B_{footing} := 16\ ft + 7\ in + 7\ ft + 5\ in = 24\ ft$

$H_{footing} := 3\ ft + 9\ in = 3.75\ ft$

The condition of the Pier 1 footing due to the widening of Bridge 18W has already been checked by PYSM. The calculations below are intended to capture the effect of the additional loads imposed on the footing from wall 03.66L.

The bearing resistances for the soil under Bridge 18W Pier 1 provided by the geotech engineers are as follows:

$\sigma_{r\_str} := 158.5\ ksf$

$\sigma_{r\_ext} := 316.9\ ksf$

Table 10: Bridge 18W Foundation Bearing Resistance

Scenario	Unfactored Bearing Resistance (ksf)[1]	Factored Extreme Event Resistance [2] (ksf)[1]	Factored Strength Bearing Resistance [3] (ksf)[1]
Pier 1, Abutment	352.1	316.9	158.5
Pier 2	236.3	212.7	106.3

4.1 Bearing stress about footing toe

Strength Limit States

$\gamma_{DC} := 1.25$

$W_{comb} := 12819\ kip + 0.65 \cdot \gamma_{DC} \cdot W_{wall} = 13447\ kip$

$M_{comb} := 91067\ kip\ ft + 0.65 \cdot \gamma_{DC} \cdot W_{wall} \cdot (7\ ft + 5\ in + 1\ ft + 9\ in + 1\ ft + 3\ in) = 97613.756\ kip\ ft$

$e := \frac{M_{comb}}{W_{comb}} = 7.26\ ft$

$\sigma_{max} := \frac{W_{comb}}{2 \cdot e \cdot L_{footing}} = 11.88\ ksf$

$\sigma_r := \sigma_{r\_str} = 158.5\ ksf$

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$CoD := \frac{\sigma_r}{\sigma_{max}} = 13.35$

Extreme (SEE)

$\gamma_{DC} := 1 \qquad \gamma_{EQ} := 1$

$W_{comb} := 9527\ kip + 0.65 \cdot \gamma_{DC} \cdot W_{wall} = 10030\ kip$

$M_{comb} := 37516\ kip\ ft + 0.65 \cdot \gamma_{DC} \cdot W_{wall} \cdot (7\ ft + 5\ in + 1\ ft + 9\ in + 1\ ft + 3\ in) + = 47851.8588\ kip\ ft$

$+ 1.0 \cdot \gamma_{EQ} \cdot H_{IR.wall} \cdot \frac{h}{2}$

$e := \frac{M_{comb}}{W_{comb}} = 4.77\ ft$

$\sigma_{max} := \frac{W_{comb}}{2 \cdot e \cdot L_{footing}} = 13\ ksf$

$\sigma_r := \sigma_{r\_ext} = 316.9\ ksf$

$CoD := \frac{\sigma_r}{\sigma_{max}} = 23.52$

4.2 Sliding check

Sliding Friction Coefficients			
Soil Type	φ'	Sliding Friction Coefficient	
		Cast-In-Place Concrete	Precast Concrete
ESU 2B	41	0.87	0.70
ESU 7A	42	0.90	0.72
Common Borrow	32	0.62	0.50
Select Borrow	36	0.73	0.58
Gravel Borrow	38	0.78	0.63

Notes:
Sliding coefficients calculated according to AASHTO Equation 10.6.3.4-2

Cast-In-Place Concrete Coefficient of Friction:
Coefficient of friction = 1.0*tan(Phi')

Precast Concrete Coefficient of Friction:
Coefficient of friction = 0.8*tan(Phi')

$\phi_f := 38\ deg$

$\mu_{fr} := \tan(\phi_f) = 0.78$

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Strength Limit States

The CoD ratio for the Strength Limit States remains unchanged from the calcs carried out by PYSM.

$CoD := 2.06$

Extreme (SEE)

$\gamma_{DC} := 1 \qquad \gamma_{EQ} := 1$

$P_{comp} := 12147\ kip + 1.0 \cdot \gamma_{EQ} \cdot H_{IR.wall} = 12580.174\ kip$

$Q_R := 14888\ kip + \mu_{fr} \cdot 0.65 \cdot \gamma_{DC} \cdot W_{wall} = 15280.823\ kip$

$CoD := \frac{Q_R}{P_{comp}} = 1.21$

4.4 Overturning check

Strength Limit States

$\gamma_{DC} := 0.9$

$W_{comb} := 8956\ kip + 0.65 \cdot \gamma_{DC} \cdot W_{wall} = 9408.512\ kip$

$M_{comb} := 44090\ kip\ ft + 0.65 \cdot \gamma_{DC} \cdot W_{wall} \cdot (7\ ft + 5\ in + 1\ ft + 9\ in + 1\ ft + 3\ in) = 48803.6643\ kip\ ft$

$e := \frac{M_{comb}}{W_{comb}} = 5.19\ ft$

Eccentricity limits as per AASHTO CL.11.6.3.3.:

$e_{min} := \frac{1}{2} \cdot \frac{B_{footing}}{3} = 4\ ft$

$e_{max} := B_{footing} - \frac{1}{2} \cdot \frac{B_{footing}}{3} = 20\ ft$

The eccentricity calculated above and the eccentricity calculated in are within the limits: OK

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Extreme (SEE)

$\gamma_{DC} := 1 \qquad \gamma_{EQ} := 1$

$W_{comb} := 9361\ kip + \gamma_{DC} \cdot W_{wall} = 10134.5244\ kip$

$M_{comb} := 38747\ kip\ ft + 0.65 \cdot \gamma_{DC} \cdot W_{wall} \cdot (7\ ft + 5\ in + 1\ ft + 9\ in + 1\ ft + 3\ in) + 1.0 \cdot \gamma_{EQ} \cdot H_{IR.wall} \cdot \frac{h}{2} = 49082.8588\ kip\ ft$

$e := \frac{M_{comb}}{W_{comb}} = 4.84\ ft$

Eccentricity limits as per AASHTO CL.11.6.3.3.:

$e_{min} := \frac{1}{2} \cdot \frac{B_{footing}}{3} = 4\ ft$

$e_{max} := B_{footing} - \frac{1}{2} \cdot \frac{B_{footing}}{3} = 20\ ft$

The eccentricity calculated above and the eccentricity calculated in are within the limits: OK

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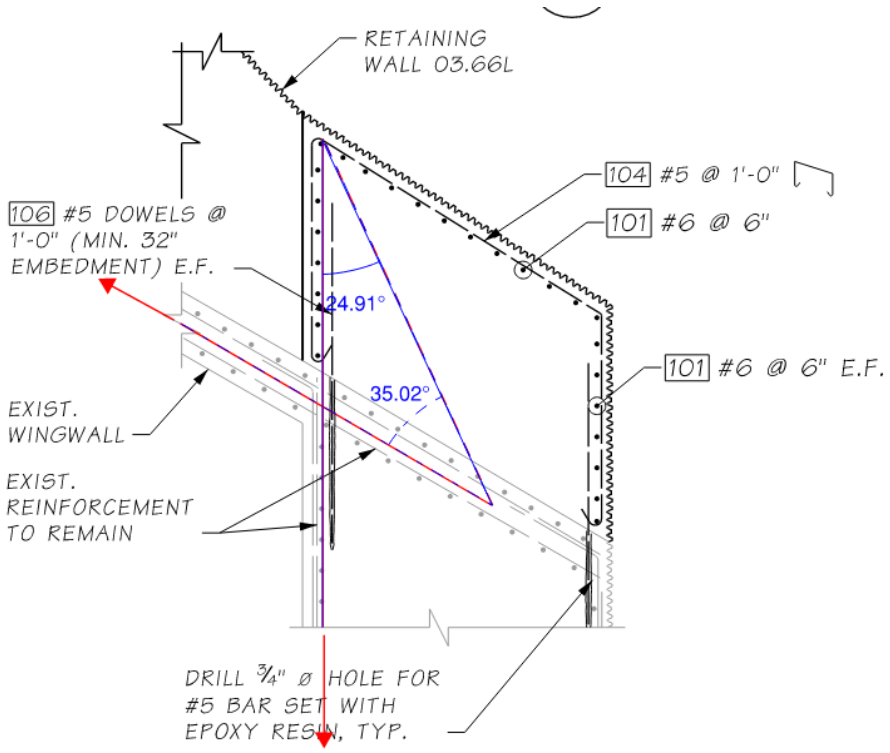
5. Check of Bridge 18W Widening

We have assumed throughout these calculations that a portion of the loads from Bridge 18W are transferred to Bridge 17.7 through the Bridge 18W wingwall rebar. At the junction of the 03.66L wall and the Bridge 18W Pier 1, the loads coming from the widened portion of Bridge 18W need to be transferred to the wingwall rebar. The strut and tie model to transfer the loads is shown below.



The tensile force in the wingwall rebar from the widened portion of 18W is:

$$T_{f.wwall} := H_{IR.ext.18W} + \frac{H_{brg}}{2} = 62.294\ \text{kip}$$

The tensile force in the the widened Bridge 18W abutment seat is:

$$T_{f.seat} := \frac{T_{f.wwall}}{\cos(35.02\ \text{deg})} \cdot \cos(25\ \text{deg}) = 68.939\ \text{kip}$$

The reinforcement in the Bridge 18W Pier 1 seat consists of 17 #5 bars at 1 foot spacing.

Factored tensile resistance:

$\varphi := 0.9$ $\qquad T_r := \varphi \cdot 17 \cdot 0.31\ \text{in}^2 \cdot f_y = 284.58\ \text{kip}$

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$$CoD := \frac{T_r}{T_{f.wwall} + T_{f.seat}} = 2.17$$

Therefore, we have adequate capacity to transfer the forces from the widened portion of Bridge 18W to the wingwall rebar.

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6. Check of Bridge 17.7 Existing Pier 1 Footing

Elevation of SB405 pavement: $EL_{SB405} := 73.05\ ft$

Elevation at top of existing Pier 1 wall: $EL_{TopWall} := EL_{SB405} - 2\ ft = 71.05\ ft$

Elevation at bottom of existing Pier 1 footing: $EL_{BottFtg} := 32\ ft + 3.57\ ft = 35.57\ ft$

$L_{ftg} := 41\ ft + 8\ in = 42\ ft$

$B_{ftg} := 12\ ft$

$H_{ftg} := 3\ ft + 6\ in = 3.5\ ft$

$L_{wall} := L_{ftg} - 2 \cdot 2\ ft = 37.667\ ft$

$B_{wall} := 2\ ft + 6\ in = 2.5\ ft$

$H_{wall} := EL_{TopWall} - EL_{BottFtg} - H_{ftg} = 31.98\ ft$

6.1 Bearing stress about center of footing (considering effective footing area)

Selfweight of Pier 1:

$$W_{17.7} := \left(L_{ftg} \cdot B_{ftg} \cdot H_{ftg} + L_{wall} \cdot B_{wall} \cdot \left(H_{wall} - 2\ ft\right)\right) \cdot \gamma_{conc} = 708.83\ kip$$

Weight of fill above Pier 1:

$$W_{fill} := \left(B_{ftg} - B_{wall}\right) \cdot L_{ftg} \cdot \left(EL_{SB405} - EL_{BottFtg} + H_{ftg}\right) \cdot \gamma_{fill} = 1622.12\ kip$$

Inertial force from Pier 1 selfweight and fill: $H_{IR.17.7} := k_h \cdot \left(W_{17.7} + W_{fill}\right) = 1305.34\ kip$

Also need to consider the transverse demands coming from the 03.66L wall and Bridge 18W. This demand has been calculated previously. This includes the portion of load coming from Bridge 18W, the inertial force from the self weight of 03.66L wall and the earth pressure from the earth retained by Bridge 17.7 wingwall.

$$H_{IR.18W} := H_{18W} + 0.7 \cdot H_{IR.wall} + 0.5 \cdot P_{fill.17.7} = 836.039\ kip$$

Finally, the new Pier 1 footing is going to be connected to the existing Pier 1 wall. From RSGR,under the extreme limit states, there are axial demands and transverse moments induced on the existing Pier 1 footing.

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Case 1 (30% transverse + 100% longitudinal):

Axial Load from new Pier 1 Ftg: $P_{Ext1} := 3710\ kip$

Transverse moment from new Pier 1 Ftg: $M_{Ext1} := 24000\ kip\ ft$

Case 2 (100% transverse + 30% longitudinal):

Axial Load from new Pier 1 Ftg: $P_{Ext2} := 4305\ kip$

Transverse moment from new Pier 1 Ftg: $M_{Ext2} := 40000\ kip\ ft$

The bearing resistance for the Strength Limit and Extreme Limit States at Bridge 17.7 Pier 1 are obtained from the geotech engineers as shown below:

Results Summary							
Scenario	Footing Width	Footing Length	Unfactored Bearing Resistance	Extreme Event Resistance[a]	Strength Bearing Resistance[b]	Service Limit State	
						1/4" Deflection	1/2" Deflection
	(ft)	(ft)	(ksf)	(ksf)	(ksf)	(ksf)	(ksf)
Pier 1 - Existing	12	42	578.3	520.5	260.3	1.6	3.1
Pier 2 - Existing	14	28	106.5	95.9	47.9	1.4	2.8

Strength Limit States

$\gamma_{DC} := 1.25$

$W_{comb} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} + \gamma_{DC} \cdot W_{17.7} + \gamma_{DC} \cdot W_{fill} = 3252.115\ kip$

$M_{comb\_long} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} \cdot \left(\frac{12\ ft + 6.5\ in}{2} - (3\ ft + 3\ in) \right) = 1022.301\ kip\ ft$

$M_{comb\_trans} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} \cdot \frac{L_{ftg}}{2} = 7050.353\ kip\ ft$

$M_{comb} := \sqrt{M_{comb\_long}^{2} + M_{comb\_trans}^{2}} = 7124\ kip\ ft$

$e := \frac{M_{comb}}{W_{comb}} = 2.19\ ft$

$\theta := \operatorname{atan}\left(\frac{\left| M_{comb\_long} \right|}{M_{comb\_trans}} \right) = 0.14\ rad$

$L'_{footing} := 2 \cdot \left(\frac{L_{footing}}{2} - e \cdot \cos(\theta) \right) = 73.66\ ft$

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$$B'_{footing} := 2 \cdot \left(\frac{B_{footing}}{2} - e \cdot \sin(\theta) \right) = 23.37 \text{ ft}$$

$$\sigma_{max} := \frac{W_{comb}}{L'_{footing} \cdot B'_{footing}} = 1.89 \text{ ksf}$$

$$\sigma_r := 260.3 \text{ ksf}$$

$$CoD := \frac{\sigma_r}{\sigma_{max}} = 137.8$$

Extreme Limit State (SEE)

Case 1: Longitudinal EQ + 30% Transverse EQ

$$\gamma_{DC} := 1.00 \qquad \gamma_{EQ} := 1.00$$

$$W_{comb} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} + \gamma_{DC} \cdot W_{17.7} + \gamma_{EQ} \cdot P_{Ext1} = 4689.5666 \text{ kip}$$

$$M_{comb\_long} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} \cdot \left(\frac{12 \text{ ft} + 6.5 \text{ in}}{2} - (3 \text{ ft} + 3 \text{ in}) \right) + \gamma_{EQ} \cdot H_{IR.17.7} \cdot \frac{h}{2} = 16181.65 \text{ kip ft}$$

$$M_{comb\_trans} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} \cdot \frac{L_{ftg}}{2} + 0.3 \cdot \gamma_{EQ} \cdot H_{IR.18W} \cdot \frac{h}{2} + \gamma_{EQ} \cdot M_{Ext1} = 32592.34 \text{ kip ft}$$

$$M_{comb} := \sqrt{M_{comb\_long}^2 + M_{comb\_trans}^2} = 36388 \text{ kip ft}$$

$$e := \frac{M_{comb}}{W_{comb}} = 7.76 \text{ ft}$$

$$\theta := \operatorname{atan}\left(\frac{\left| M_{comb\_long} \right|}{M_{comb\_trans}} \right) = 0.46 \text{ rad}$$

$$L'_{footing} := 2 \cdot \left(\frac{L_{footing}}{2} - e \cdot \cos(\theta) \right) = 64.1 \text{ ft}$$

$$B'_{footing} := 2 \cdot \left(\frac{B_{footing}}{2} - e \cdot \sin(\theta) \right) = 17.1 \text{ ft}$$

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$$\sigma_{max} := \frac{W_{comb}}{L'_{footing} \cdot B'_{footing}} = 4.28\ ksf$$

$$\sigma_r := 520.5\ ksf$$

$$CoD := \frac{\sigma_r}{\sigma_{max}} = 121.65$$

Case 2: 30% Longitudinal EQ + 100% Transverse EQ

$$\gamma_{DC} := 1.00 \qquad \gamma_{EQ} := 1.00$$

$$W_{comb} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} + \gamma_{DC} \cdot W_{17.7} + \gamma_{EQ} \cdot P_{Ext2} = 5284.57\ kip$$

$$M_{comb\_long} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} \cdot \left(\frac{12\ ft + 6.5\ in}{2} - (3\ ft + 3\ in) \right) + 0.3 \cdot \gamma_{EQ} \cdot H_{IR.17.7} \cdot \frac{h}{2} = 5426.98\ kip\ ft$$

$$M_{comb\_trans} := 0.35 \cdot \gamma_{DC} \cdot W_{wall} \cdot \frac{L_{ftg}}{2} + 1.0 \cdot \gamma_{EQ} \cdot H_{IR.18W} \cdot \frac{h}{2} + \gamma_{EQ} \cdot M_{Ext2} = 55480.46\ kip\ ft$$

$$M_{comb} := \sqrt{M_{comb\_long}^{2} + M_{comb\_trans}^{2}} = 55745\ kip\ ft$$

$$e := \frac{M_{comb}}{W_{comb}} = 10.55\ ft$$

$$\theta := \operatorname{atan}\left(\frac{\left| M_{comb\_long} \right|}{M_{comb\_trans}} \right) = 0.1\ rad$$

$$L'_{footing} := 2 \cdot \left(\frac{L_{footing}}{2} - e \cdot \cos(\theta) \right) = 57\ ft$$

$$B'_{footing} := 2 \cdot \left(\frac{B_{footing}}{2} - e \cdot \sin(\theta) \right) = 21.95\ ft$$

$$\sigma_{max} := \frac{W_{comb}}{L'_{footing} \cdot B'_{footing}} = 4.22\ ksf$$

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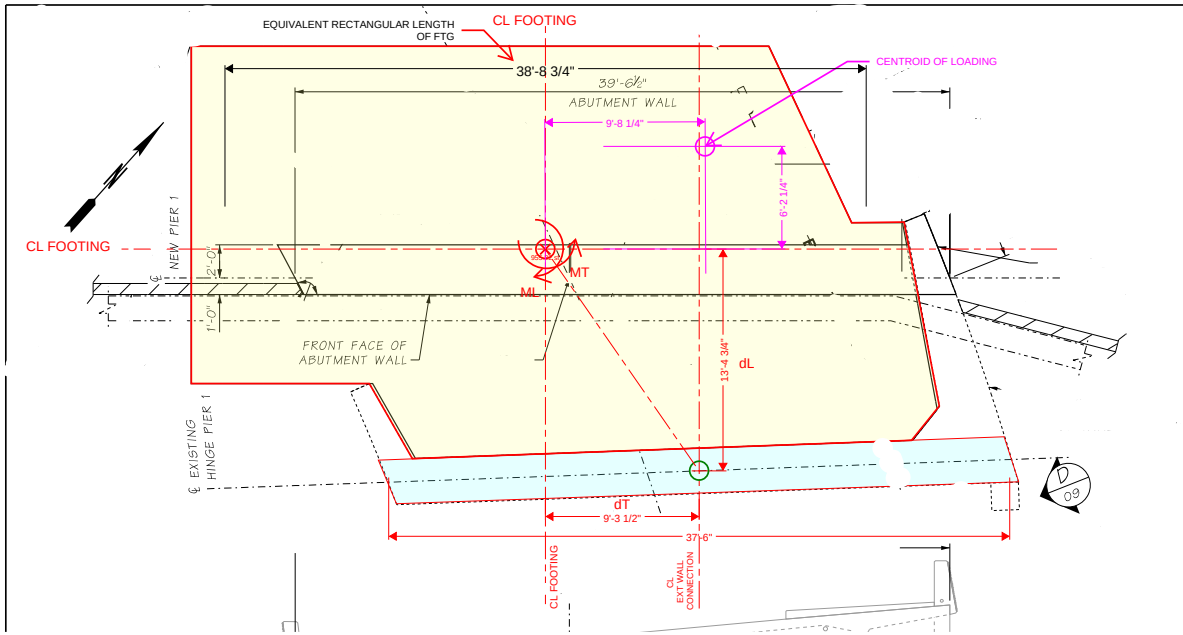
$\sigma_r := 520.5\ \text{ksf}$

$CoD := \frac{\sigma_r}{\sigma_{max}} = 123.22$



Compression Force into existing wall

100% transverse + 30% longitudinal

Case	MT	ML	dT	dL	P
	k-ft	k-ft	ft	ft	kips
EXT	40000	0	9.291667	13.39583	4305
	31000	1450	9.291667	13.39583	3445
	24000	15100	9.291667	13.39583	3710
STR	6600	-8600	9.291667	13.39583	68
SER	5400	-12000	9.291667	13.39583	-315

Use this value
STR and SER can be ignored to induce any compression forces in the wall

100%L + 30%T

Max. Compression to existing wall due to moments at new footing tied to existing wall

DATE : 2021 NOV 19
BY: RSGR

COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I11

CALC BOOK COMPLETION DATE: **2021 Nov 30**

Title: Tendon Geometry

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I11-0	-	I11-0	Calc Register (this sheet)	N/A	N/A	
2	I11-1	-	I11-1	Tendon Gemoetry	MWBM	LDLI	
		-					
		-					

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PROJECT	I-405, Renton to Bellevue Widening and Express Toll Lanes Project			CONT	A207833
SUBJECT	BR-28W- Geometric Data-Elevations			PAGE	-
BY:	MWBM	DATE	2021-11-30	CHECK BY:	LDLI	DATE	2021-11-30

PT-1		Station	Offset	Elevation	Long. Slope	Cross Slope	Top dist	Bott dist	PT WP	Northing	Easting	Length	Angle	
Location	Roadway	[ft]	[ft]	[ft]	[%]	[%]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[deg]	
New Pier 1	R-Line	15+13.68	-13.02	95.07	0.146	-0.040	1.625	2.625	93.45	178159.04	1302198.28	11.87		
Exist Pier 1	R-Line	15+24.51	-12.97	96.60	0.129	-0.040	3.375	0.875	93.23	178153.53	1302208.79	35.63	11.88	ExPr1
Deviator Diaphragm 1	R-Line	15+56.88	-12.91	100.71	0.123	-0.040	3.375	0.875	97.34	178132.35	1302237.15	30.92	10.53	DevDia1
Exist Pier 2	R-Line	15+86.44	-12.94	104.20	0.113	-0.040	1.042	3.208	103.16	178110.32	1302258.05		8.24	ExPr2
Deviator Diaphragm 2	R-Line	16+24.94	-12.94	108.31	0.100	-0.040	3.375	0.875	104.93	178082.11	1302284.25	38.54	5.17	DevDia2
Exist Pier 3	R-Line	16+64.78	-12.94	112.02	0.069	-0.040	1.625	2.625	110.39	178052.92	1302311.36	40.21		
												157.17	**35.83**	

PT-2		Station	Offset	Elevation	Long. Slope	Cross Slope	Top dist	Bott dist	PT WP	Northing	Easting	Length	Angle	
Location	Roadway	[ft]	[ft]	[ft]	[%]	[%]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[deg]	
New Pier 1	R-Line	15+12.28	-9.45	94.73	0.146	-0.040	1.625	2.625	93.10	178156.45	1302195.40	12.05		
Exist Pier 1	R-Line	15+23.54	-9.40	96.33	0.129	-0.040	3.375	0.875	92.96	178150.94	1302206.12	35.57	12.80	ExPr1
Deviator Diaphragm 1	R-Line	15+56.72	-8.65	100.52	0.123	-0.040	3.375	0.875	97.15	178129.37	1302234.08	30.82	10.20	DevDia1
Exist Pier 2	R-Line	15+86.44	-8.35	104.02	0.113	-0.040	1.042	3.208	102.98	178107.21	1302254.69		8.27	ExPr2
Deviator Diaphragm 2	R-Line	16+24.94	-8.35	108.12	0.100	-0.040	3.375	0.875	104.75	178078.99	1302280.89	38.54	5.17	DevDia2
Exist Pier 3	R-Line	16+64.78	-8.35	111.83	0.069	-0.040	1.625	2.625	110.21	178049.80	1302308.00	40.21		
												157.19	**36.45**	

PT-3		Station	Offset	Elevation	Long. Slope	Cross Slope	Top dist	Bott dist	PT WP	Northing	Easting	Length	Angle	
Location	Roadway	[ft]	[ft]	[ft]	[%]	[%]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[deg]	
New Pier 1	R-Line	15+07.23	1.90	93.54	0.146	-0.040	1.625	2.625	91.91	178148.11	1302186.12	12.66		
Exist Pier 1	R-Line	15+20.08	1.99	95.40	0.146	-0.040	3.375	0.875	92.03	178142.64	1302197.53	35.85	12.23	ExPr1
Deviator Diaphragm 1	R-Line	15+56.28	2.07	100.04	0.123	-0.040	3.375	0.875	96.67	178121.90	1302226.40	30.59	11.26	DevDia1
Exist Pier 2	R-Line	15+86.44	2.04	103.60	0.113	-0.040	1.042	3.208	102.56	178100.13	1302247.07		8.50	ExPr2
Deviator Diaphragm 2	R-Line	16+24.94	2.04	107.71	0.100	-0.040	3.375	0.875	104.33	178071.92	1302273.27	38.54	5.17	DevDia2
Exist Pier 3	R-Line	16+64.78	2.04	111.42	0.069	-0.040	1.625	2.625	109.79	178042.73	1302300.38	40.21		
												157.85	**37.16**	

PT-4		Station	Offset	Elevation	Long. Slope	Cross Slope	Top dist	Bott dist	PT WP	Northing	Easting	Length	Angle	
Location	Roadway	[ft]	[ft]	[ft]	[%]	[%]	[ft]	[ft]	[ft]	[ft]	[ft]	[ft]	[deg]	
New Pier 1	R-Line	15+04.85	6.60	93.00	0.146	-0.040	1.625	2.625	91.38	178144.61	1302182.23	12.94		
Exist Pier 1	R-Line	15+18.47	6.66	94.98	0.146	-0.040	3.375	0.875	91.61	178139.23	1302194.00	35.88	13.09	ExPr1
Deviator Diaphragm 1	R-Line	15+56.04	7.10	99.81	0.123	-0.040	3.375	0.875	96.43	178118.39	1302222.80	30.47	11.62	DevDia1
Exist Pier 2	R-Line	15+86.44	7.41	103.39	0.113	-0.040	1.042	3.208	102.35	178096.48	1302243.14		8.55	ExPr2
Deviator Diaphragm 2	R-Line	16+24.94	7.41	107.49	0.100	-0.040	3.375	0.875	104.12	178068.27	1302269.34	38.54	5.17	DevDia2
Exist Pier 3	R-Line	16+64.78	7.41	111.20	0.069	-0.040	1.625	2.625	109.58	178039.08	1302296.45	40.21		
												158.05	**38.44**	

COWI

PROJECT: **WSDOT I-405 R2B - BR 17.7**

CLIENT: **Wood PLC**

CONT: **A207833**

Book: I12 CALC BOOK COMPLETION DATE: **2021 Nov 26**

Title: Shear Key Checks for Non-Seismic Loading

Item	Page #'s	to	Page #'s	Subject / Description	Designer's Initials	Checker's Initials	Comments
1	I12-0	-	I12-0	Calc Register (this sheet)	N/A	N/A	
2	I12-1	-	I12-5	Pier 1 (Dowel and Box Capacities)	LDLI	RSGR	
3	I12-6	-	I12-9	Pier 1 (Dowel and Wall Capacities)	LDLI	RSGR	
4	I12-10	-	I12-14	Pier 2 (Dowel and Box Capacities)	LDLI	RSGR	
4	I12-15	-	I12-18	Pier 2 (Dowel and Wall Capacities)	LDLI	RSGR	
6	I12-19	-	I12-24	Pier 3 (Dowel and Box Capacities)	LDLI	RSGR	
7	I12-25	-	I12-30	Pier 3 (Dowel and Cap Beam Capacities)	LDLI	RSGR	
		-					
		-					

1. Purpose and Assumptions

1.The purpose is to calculate and check the dowel and surrounding concrete box capacities of the shear key at new Pier 1
2. The calculation is based on AASHTO 8th Edition, 2017. and ACI 318-14

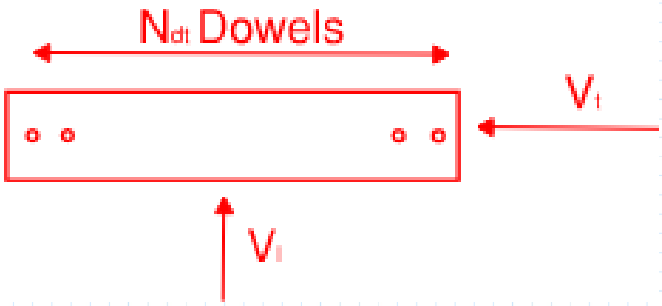
2. Material Properties

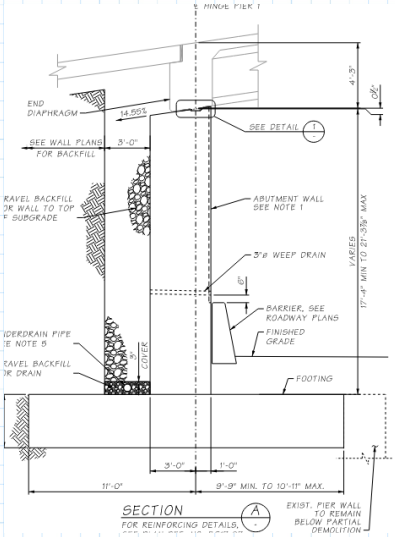
Concrete Compressive Strength	$f_c := 5 \cdot ksi$
Reinforcement Yield Strength	$f_y := 60 \cdot ksi$
Reinforcement Ultimate Tensile Strength	$f_{uta} := 90 \cdot ksi$

3. Structural Properties

Width of diaphragm	$b := 37.5 \cdot ft$
Thickness of diaphragm	$t := 32.5\ in$
Shear key top width	$b_{aa} := 10\ in$
Shear key depth	$d := 2\ in$
Shear key angle	$\alpha := 45\ deg$
Shear key bottom width	$b_{bb} := b_{aa} - 2\ d \cdot \tan(\alpha) = 6\ in$

For dowels





Dowel Size	#9	
Dowel Area	$A_d := 1 \cdot in^2$	
Dowel diameter	$d_d := 1.128 \cdot in$	
Dowel Qty	$N_{dt} := 72$	
Total Dowel Area	$N_{dt} \cdot A_d = 72\ in^2$	
Dowel spacing	$s_t := \frac{b - 2 \cdot 2\ ft}{N_{dt} - 1} = 5.66\ in$	Note: assume the first dowel starts at 2' from concrete face
Dowel distributed width	$B := b - 2\ ft = 35.5\ ft$	

4. Demands

Longitudinal Shear Force	$V_l := 999\ kip$	Note: these are the maximum absolute longitudinal and transverse shear from the strength limit states envelope.
Transverse Shear Force	$V_t := 75\ kip$	

5. Anchor Rod Shear Resistance

Anchor rod shear resistance factor (ACI 318 17.3.3)	$\phi_{shear.dowel} := 0.65$
Factored anchor rod shear resistance	$V_{sa} := 0.6\ \phi_{shear.dowel} \cdot f_{uta} \cdot (N_{dt}) \cdot A_d = (2.53 \cdot 10^3)\ kip$

6. Anchor Reinforcement Shear Breakout Resistance

Height of box $h_a := 4.25\ ft$

Embedment length of dowels $h_{ef} := 3.5\ ft$

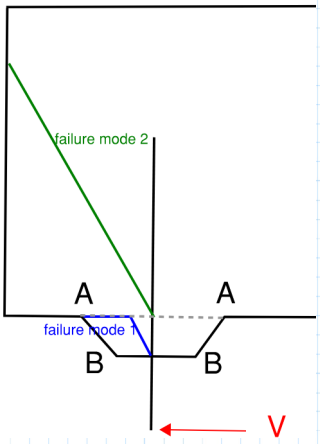
Resistance factor (ACI 318-14 17.3.3) $\phi_{concrete} := 0.75$

Load bearing length of the anchor for shear (ACI 17.5.2.2) $l_e := min\left(h_{ef}, 8 \cdot d_d\right) = 9.02\ in$

Note: assume the anchor strength is governed by concrete breakout and it's assumed to have supplementary reinforcement

For longitudinal shear

The concrete around the dowel may potentially fail in shear breakout by cracking from the start of the dowel embedment to point A (mode 1), or by cracking from the intersection between the dowel and line A-A to the face of the end diaphragm, (mode 2), as shown in the sketch on the right. We can calculate the interface shear along A-A by assuming the shear is uniformly transferred from the dowel to the surrounding concrete along the depth of the dowel embedment.



Mode 1

Interface shear demand along A-A

Depth of shear key $d = 2\ in$

Factored shear force for 1st segment $V_1 := V_l \cdot \frac{d}{h_{ef}} = 47.57\ kip$

Interface shear resistance along A-A

Cohesion factor for monolithically placed concrete(5.7.4.3) $c := 0.4 \cdot ksi$

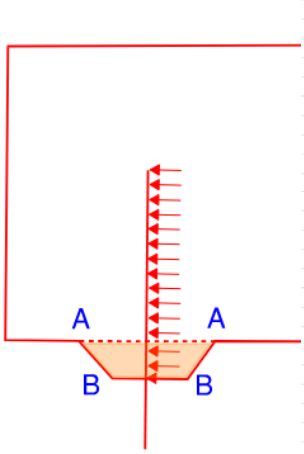
Friction factor(5.7.4.3) $\mu := 1.4$

Fraction of concrete strength available to resist interface shear (5.7.4.3) $K_1 := 0.25$

Limiting interface shear resistance (5.7.4.3) $K_2 := 1.5 \cdot ksi$

Area of concrete considered to be engaged in interface shear transfer (5.7.4.2) $A_{cv} := b \cdot \frac{b_{aa}}{2} = 15.63\ ft^2$

Nominal shear resistance of interface plane (5.7.4.3) $V_{ni} := c \cdot A_{cv} = 900\ kip$



Note: assume no compression to be conservative

The nominal shear resistance used in the design shall not exceed either of the following:

$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = \left(2.81 \cdot 10^3\right)\ kip$

$V_{ni.2} := K_2 \cdot A_{cv} = \left(3.38 \cdot 10^3\right)\ kip$

$V_{ni.limit} := min\left(V_{ni.1}, V_{ni.2}\right) = \left(2.81 \cdot 10^3\right)\ kip$

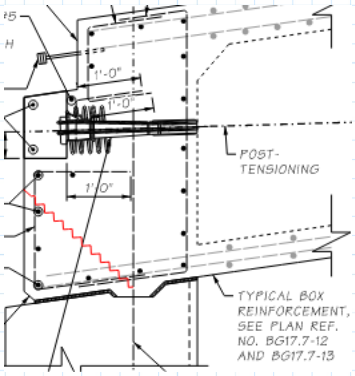
Check for shear force resistance

$$V_{ni} := \left\| \begin{array}{l} \text{if } V_{ni} \le V_{ni.limit} \\ \quad V_{ni} \\ \text{if } V_{ni} > V_{ni.limit} \\ \quad V_{ni.limit} \end{array} \right\| = 900\ kip$$

Shear force resistance factor (AASHTO 5.5.4.2) $\phi_{interfaceshear} := 0.9$

Factored Interface shear resistance along A-A $V_{ri} := V_{ni} \cdot \phi_{interfaceshear} = 810\ kip$

D/C ratio $\frac{V_l}{V_{ri}} = 0.06$



Mode 2

Shear breakout resistance

Thickness of diaphragm $t = 32.5\ in$

Distance from the center of an anchor shaft to the edge of concrete in one direction $c_{a1} := t - 12\ in = 20.5\ in$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1 $c_{a2} := 2\ ft$

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b)) $A_{vc} := min\left(b, \left(2 \cdot 1.5 \cdot c_{a1} + b - 2 \cdot 2\ ft\right)\right) \cdot min\left(1.5 \cdot c_{a1}, h_a\right) = \left(1.38 \cdot 10^4\right)\ in^2$

Projected area for a single anchor $A_{vc0} := 4.5 \cdot {c_{a1}}^2 = \left(1.89 \cdot 10^3\right)\ in^2$

Modification factor for anchor groups loaded eccentrically (ACI 17.5.2.5) $\psi_{ecv} := 1$

Modification factor for cracked concrete (ACI 17.5.2.7) $\psi_{cv} := 1.4$ Note: for cast-in dowel

Modification factor for edge effect (ACI 17.5.2.6) $\psi_{edv} := min\left(1, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) = 0.93$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 73.96\ kip$$

The shear breakout will also be resisted by the longitudinal reinforcements in the bottom slab

Shear reinforcement sizes #8

Shear reinforcement area $A_{sr} := 0.79\ in^2$

Number of within ca1 and $N_s := 30 \cdot 2 = 60$

Resistance factor (ACI 17.5.2.9) $\phi_{sr} := 0.75$ *Since ld<ca1, the long. reinforcement can develop up to the yield strength*

Development length $l_d := 13.6\ in$

Reinforcement resistance $V_{sr} := \phi_{sr} \cdot A_{sr} \cdot f_y \cdot N_s = \left(2.13 \cdot 10^3\right)\ kip$

Shear force resistance (ACI 17.5.2.1) $V_{cbgl} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b + V_{sr} = \left(2.66 \cdot 10^3\right)\ kip$

D/C ratio $\frac{V_l}{V_{cbgl}} = 0.38$ *Therefore mode 2 governs*

For transverse shear

Check for edge dowel

Assuming the demand on the edge dowel is V_t/N_{dt}, where V_t is the total transverse shear and N_{dt} is the number of the dowels

Distance from the center of an anchor shaft to the edge of concrete in one direction

$c_{a1} := 24\ in$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$c_{a2} := 12\ in$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 93.69\ kip$$

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b))

$$A_{vc} := min\left(t, \left(2 \cdot 1.5 \cdot c_{a1}\right)\right) \cdot \left(1.5 \cdot c_{a1}\right) = \left(1.17 \cdot 10^3\right)\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot \left(c_{a1}\right)^2 = \left(2.59 \cdot 10^3\right)\ in^2$$

Shear force perpendicular to the edge (ACI 17.5.2.1b)

$$V_{cbt} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 41.48\ kip$$

Demand on the edge dowel

$$V_{t.edgedowel} := \frac{V_t}{N_{dt}} = 1.04\ kip$$

D/C Ratio:

$$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.03$$

7. Concrete Pryout Strength of Anchor in Shear

Embedment Depth

$h_{ef} := 3.5\ ft$

Edge distance (ACI Fig R 17.4.2.1)

$c_1 := 12\ in$

Edge distance (ACI Fig R 17.4.2.1)

$c_2 := 2\ ft$

Adjusted embedment depth

$$h_{ef't} := \max\left(\frac{s_t}{3}, \frac{\left(c_1\right)}{1.5}\right) = 0.67\ ft$$

Anchorage projection area of a single anchor (ACI 17.4.2)

$$A_{Nco} := 9 \cdot {h_{ef't}}^2 = 4\ ft^2$$

Anchorage projection area of a group anchor

$$A_{Nct} := min\left(t, 2 \cdot \left(min\left(c_2, 1.5 \cdot h_{ef't}\right)\right)\right) \cdot min\left(b, \left(\left(N_{dt} - 1\right) \cdot s_t + c_1 + 1.5 \cdot h_{ef't}\right)\right) = 71\ ft^2$$

Adjusted anchorage projection area of a group anchor

$$A_{Nct} := min\left(A_{Nct}, N_{dt} \cdot A_{Nco}\right) = 71\ ft^2$$

Min edge distance

$$c_{min} := min\left(c_1, 1.5 \cdot h_{ef't}\right) = 12\ in$$

Eccentricity effects (ACI 17.4.2.4)

$\psi_{ec.N} := 1$

Edge effects (ACI 17.4.2.5)

$$\psi_{ed.N} := min\left(0.7 + 0.3 \cdot \frac{c_{min}}{1.5 \cdot h_{ef't}}\right) = 1$$

Concrete cracking (ACI 318 17.4.2.6)

$\psi_{c.N} := 1.25$

Concrete splitting (ACI 318 17.4.2.7)

$\psi_{cp.N} := 1$

Note: for cast in anchor

For normal density concrete

$\lambda := 1$

Basic tensile resistance (ACI 17.4.2.2)

$$N_b := 24 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot \frac{\left(\frac{h_{ef't}}{in}\right)^{1.5} \cdot kip}{1000} = 38.4\ kip$$

Concrete breakout group resistance (ACI 17.4.2) $N_{cbgt} := \frac{A_{Nct}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 852$ *kip*

Concrete breakout single rod resistance $N_{cbt} := \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 48$ *kip*

Coefficient for pryout strength (ACI 17.5.3.1) $k_{cp} := 2$ Note: hef>2.5in

Coefficient for basic concrete breakout strength in tension (ACI 17.5.3.1) $k_c := 24$ Note: kc is 24 for cast-in place anchors

Factored concrete pryout strength (ACI 17.5.3.1) $V_{cpg} := \phi_{concrete} \cdot k_{cp} \cdot (N_{cbgt}) = (1.28 \cdot 10^3)$ *kip*

D/C ratio $\frac{V_l}{V_{cpg}} = 0.78$ $\frac{V_t}{V_{cpg}} = 0.06$

8. Anchor Rod on Concrete Bearing

Concrete resistance $\phi_c := 0.75$

Bearing area $A_{br} := d_d \cdot min(8 \cdot d_d, h_{ef}) = 10.18$ *in*^{2}

Factored Concrete bearing resistance $B_r := (N_{dt}) \cdot \phi_c \cdot A_{br} \cdot f_c = (2.75 \cdot 10^3)$ *kip*

D/C Ratio $\frac{V_l}{B_r} = 0.36$ $\frac{V_t}{B_r} = 0.03$

Note: calculating the bearing resistance is not required in AASHTO or ACI, but similar calculation was found in CSA S6-19 8.16.7.3. The embedment depth is 5*diameter of the anchor in S6 and is changed to 8*diameter of the anchor based on ACI

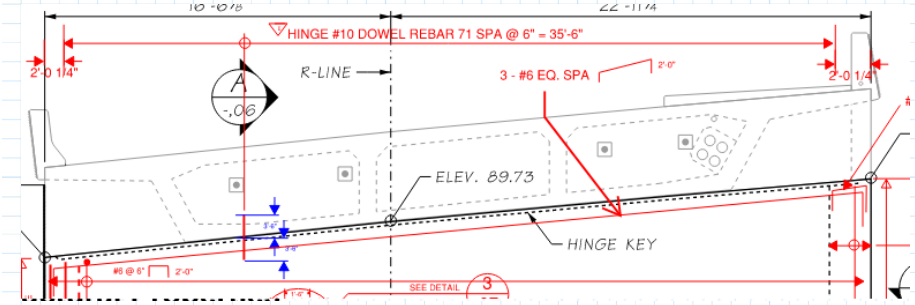
9. Summary

Governed longitudinal shear resistance for anchorage group $V_{rl} := min(V_{sa}, V_{cbgl}, V_{cpg}) = (1.28 \cdot 10^3)$ *kip*

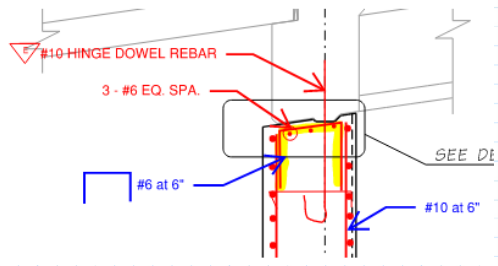
Governed transverse shear resistance for anchorage group $V_{rt} := min(V_{sa}, V_{cpg}) = (1.28 \cdot 10^3)$ *kip*

Total number of dowels needed $N_{dt} = 72$

	Demands	Resistance	D/C Ratio
Longitudinal Shear on Dowel Group	$V_l = 999$ *kip*	$V_{rl} = (1.28 \cdot 10^3)$ *kip*	$\frac{V_l}{V_{rl}} = 0.78$
Transverse Shear on Dowel Group	$V_t = 75$ *kip*	$V_{rt} = (1.28 \cdot 10^3)$ *kip*	$\frac{V_t}{V_{rt}} = 0.06$
Transverse Shear on Edge Dowel	$V_{t.edgedowel} = 1.04$ *kip*	$V_{cbt} = 41.48$ *kip*	$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.03$
Bearing	$V_l = 999$ *kip* $V_t = 75$ *kip*	$B_r = (2.75 \cdot 10^3)$ *kip*	$\frac{V_l}{B_r} = 0.36$ $\frac{V_t}{B_r} = 0.03$



Elevation View



Section View

1. Purpose and Assumptions

1. The purpose is to calculate and check the dowel and surrounding concrete wall capacities of the shear key at new Pier 1
2. The calculation is based on AASHTO 8th Edition, 2017. and ACI 318-14

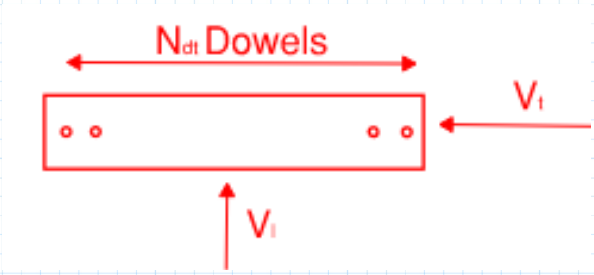
2. Material Properties

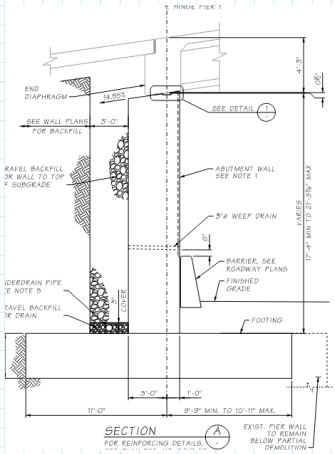
Concrete Compressive Strength	$f_c := 4 \cdot ksi$
Reinforcement Yield Strength	$f_y := 60 \cdot ksi$
Reinforcement Ultimate Tensile Strength	$f_{uta} := 90 \cdot ksi$

3. Structural Properties

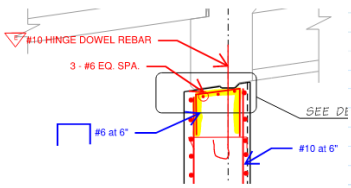
Width of wall	$b := 41.73 \cdot ft$
Thickness of wall	$t := 3\ ft$

For dowels





Dowel Size	#9
Dowel Area	$A_d := 1 \cdot in^2$
Dowel diameter	$d_d := 1.128 \cdot in$
Dowel Qty	$N_{dt} := 72$
Total Dowel Area	$N_{dt} \cdot A_d = 72\ in^2$
Dowel spacing	$s_t := \frac{b - 2 \cdot 2\ ft}{N_{dt} - 1} = 6.38\ in$
Dowel distributed width	$B := b - 2\ ft = 39.73\ ft$



Note: assume the first dowel starts at 2' from concrete face

4. Demands

Longitudinal Shear Force	$V_l := 999\ kip$
Transverse Shear Force	$V_t := 75\ kip$

Note: these are the maximum absolute longitudinal and transverse shear from the strength limit states envelope.

5. Anchor Rod Shear Resistance

Anchor rod shear resistance factor (ACI 318 17.3.3)	$\phi_{shear.dowel} := 0.65$
Factored anchor rod shear resistance	$V_{sa} := 0.6\ \phi_{shear.dowel} \cdot f_{uta} \cdot (N_{dt}) \cdot A_d = (2.53 \cdot 10^3)\ kip$

6. Anchor Reinforcement Shear Breakout Resistance

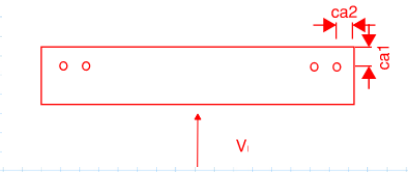
For longitudinal shear

Thickness of member in which an anchor is located: $h_a := 19.5\ ft$

Embedment length of dowels: $h_{ef} := 3.5\ ft$ — Note: designed value

Resistance factor (ACI 17.3.3): $\phi_{concrete} := 0.75$ — Note: assume the anchor strength is governed by concrete breakout and it's assumed to have supplementary reinforcement

Load bearing length of the anchor for shear (ACI 17.5.2.2): $l_e := min\left(h_{ef}, 8 \cdot d_d\right) = 9.02\ in$



Distance from the center of an anchor shaft to the edge of concrete in one direction: $c_{a1} := 1\ ft$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1: $c_{a2} := 2\ ft$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2):

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 29.63\ kip$$

Modification factor for edge effect (ACI 17.5.2.6):

$$\psi_{edv} := min\left(1, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) = 1$$

Modification factor for cracked concrete (ACI 17.5.2.7): $\psi_{cv} := 1.4$ — Note: cast-in anchors

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b)):

$$A_{vc} := min\left(b, \left(2 \cdot 1.5 \cdot c_{a1} + b - 2 \cdot 6\ in\right)\right) \cdot min\left(1.5 \cdot c_{a1}, h_a\right) = \left(9.01 \cdot 10^3\right)\ in^2$$

Projected area for a single anchor: $A_{vc0} := 4.5 \cdot {c_{a1}}^2 = 648\ in^2$

Modification factor for anchor groups loaded eccentrically (ACI 17.5.2.5): $\psi_{ecv} := 1$

Shear force perpendicular to the edge (ACI 17.5.2.1):

$$V_{cbgl} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 432.7\ kip$$

Shear reinforcement sizes: #10

Shear reinforcement area: $A_{sr} := 1.27\ in^2$

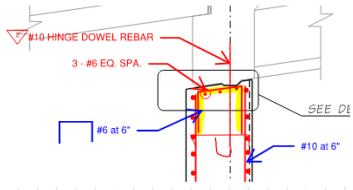
Resistance factor (ACI 17.5.2.9): $\phi_{sr} := 0.75$

Number of #10 available: $N_s := 72$

Reinforcement resistance: $V_{sr} := \phi_{sr} \cdot A_{sr} \cdot f_y \cdot N_s$

Total shear force resistance: $V_{cbgl'} := V_{cbgl} + V_{sr} = \left(4.55 \cdot 10^3\right)\ kip$

D/C Ratio: $\frac{V_l}{V_{cbgl'}} = 0.22$

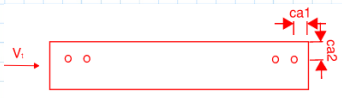


For transverse shear

Check for edge dowel

Assuming the demand on the edge dowel is V_t / N_{dt}, where V_t is the total transverse shear and N_{dt} is the number of the dowels

Distance from the center of an anchor shaft to the edge of concrete in one direction: $c_{a1} := 24\ in$



Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1 $c_{a2} := 1\ ft$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2) $V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 83.79\ kip$

Total projected area for a group of anchors (ACI Figure R17.5.2.1(b)) $A_{vc} := min\left(t, \left(2 \cdot 1.5 \cdot c_{a1}\right)\right) \cdot \left(1.5 \cdot c_{a1}\right) = \left(1.3 \cdot 10^3\right)\ in^2$

Projected area for a single anchor $A_{vc0} := 4.5 \cdot \left(c_{a1}\right)^2 = \left(2.59 \cdot 10^3\right)\ in^2$

Shear force perpendicular to the edge (ACI 17.5.2.1b) $V_{cbt} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 43.99\ kip$

Demand on the edge dowel $V_{t.edgedowel} := \frac{V_t}{N_{dt}} = 1.04\ kip$

D/C Ratio: $\frac{V_{t.edgedowel}}{V_{cbt}} = 0.02$

7. Concrete Pryout Strength of Anchor in Shear

Embedment Depth $h_{ef} := 3.5\ ft$

Edge distance (ACI Fig R 17.4.2.1) $c_1 := 12\ in$

Edge distance (ACI Fig R 17.4.2.1) $c_2 := 24\ in$

Adjusted embedment depth $h_{ef't} := \max\left(\frac{s_t}{3}, \frac{\left(c_1\right)}{1.5}\right) = 0.67\ ft$

Anchorage projection area of a single anchor (ACI 17.4.2) $A_{Nco} := 9 \cdot {h_{ef't}}^2 = 4\ ft^2$

Anchorage projection area of a group anchor $A_{Nct} := min\left(t, 2 \cdot \left(min\left(c_2, 1.5 \cdot h_{ef't}\right)\right)\right) \cdot min\left(b, \left(\left(N_{dt} - 1\right) \cdot s_t + c_1 + 1.5 \cdot h_{ef't}\right)\right) = 79.46\ ft^2$

Adjusted anchorage projection area of a group anchor $A_{Nct} := min\left(A_{Nct}, N_{dt} \cdot A_{Nco}\right) = 79.46\ ft^2$

Min edge distance $c_{min} := min\left(c_1, 1.5 \cdot h_{ef't}\right) = 12\ in$

Eccentricity effects (ACI 17.4.2.4) $\psi_{ec.N} := 1$

Edge effects (ACI 17.4.2.5) $\psi_{ed.N} := min\left(0.7 + 0.3 \cdot \frac{c_{min}}{1.5 \cdot h_{ef't}}\right) = 1$

Concrete cracking (ACI 318 17.4.2.6) $\psi_{c.N} := 1.25$

Concrete splitting (ACI 318 17.4.2.7) $\psi_{cp.N} := 1$ Note: for cast in anchor

For normal density concrete $\lambda := 1$

Basic tensile resistance (ACI 17.4.2.2) $N_b := 24 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot \frac{\left(\frac{h_{ef't}}{in}\right)^{1.5} \cdot kip}{1000} = 34.35\ kip$

Concrete breakout group resistance (ACI 17.4.2) $N_{cbgt} := \frac{A_{Nct}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 852.85\ kip$

Concrete breakout single rod resistance $N_{cbt} := \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 42.93\ kip$

Coefficient for pryout strength (ACI 17.5.3.1) $k_{cp} := 2$ Note: hef>2.5in

Coefficient for basic concrete breakout strength in tension (ACI 17.5.3.1) $k_c := 24$ Note: kc is 24 for cast-in place anchors

Factored concrete pryout strength (ACI 17.5.3.1) $V_{cpg} := \phi_{concrete} \cdot k_{cp} \cdot \left(N_{cbgt}\right) = \left(1.28 \cdot 10^3\right)\ kip$

D/C Ratio $\frac{V_l}{V_{cpg}} = 0.78$ $\frac{V_t}{V_{cpg}} = 0.06$

8. Anchor Rod on Concrete Bearing

Concrete resistance $\phi_c := 0.75$

Bearing area $A_{br} := d_d \cdot min\left(8 \cdot d_d, h_{ef}\right) = 10.18\ in^2$

Factored Concrete bearing resistance $B_r := \left(N_{dt}\right) \cdot \phi_c \cdot A_{br} \cdot f_c = \left(2.2 \cdot 10^3\right)\ kip$

D/C Ratio $\frac{V_l}{B_r} = 0.45$ $\frac{V_t}{B_r} = 0.03$

9. Summary

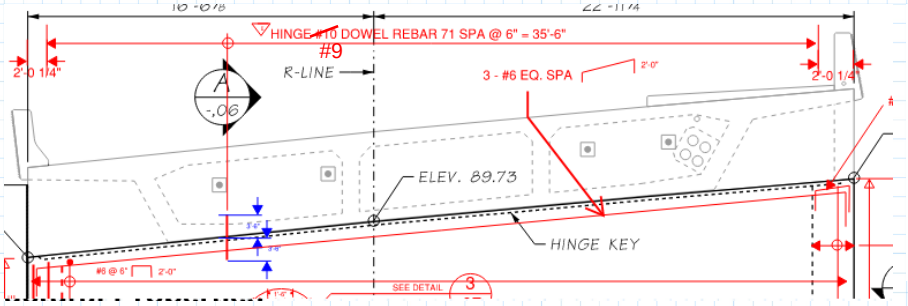
Governed longitudinal shear resistance for anchorage group $V_{rl} := min\left(V_{sa}, V_{cbgl'}, V_{cpg}\right) = \left(1.28 \cdot 10^3\right)\ kip$

Governed transverse shear resistance for anchorage group $V_{rt} := min\left(V_{sa}, V_{cpg}\right) = \left(1.28 \cdot 10^3\right)\ kip$

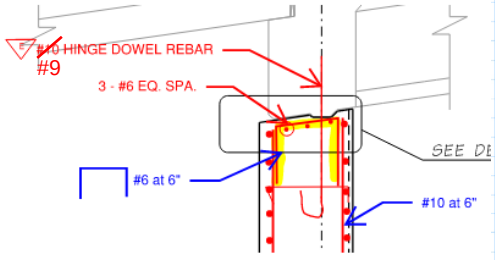
Total number of dowels needed $N_{dt} = 72$

Total number of vertical reinforcement needed within ca1 $N_s = 72$

	Demands	Resistance	D/C Ratio
Longitudinal Shear on Dowel Group	$V_l = 999\ kip$	$V_{rl} = \left(1.28 \cdot 10^3\right)\ kip$	$\frac{V_l}{V_{rl}} = 0.78$
Transverse Shear on Dowel Group	$V_t = 75\ kip$	$V_{rt} = \left(1.28 \cdot 10^3\right)\ kip$	$\frac{V_t}{V_{rt}} = 0.06$
Transverse Shear on Edge Dowel	$V_{t.edgedowel} = 1.04\ kip$	$V_{cbt} = 43.99\ kip$	$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.02$
Bearing	$V_l = 999\ kip$ $V_t = 75\ kip$	$B_r = \left(2.199 \cdot 10^3\right)\ kip$	$\frac{V_l}{B_r} = 0.45$ $\frac{V_t}{B_r} = 0.03$



Elevation View



Section View

1. Purpose and Assumptions

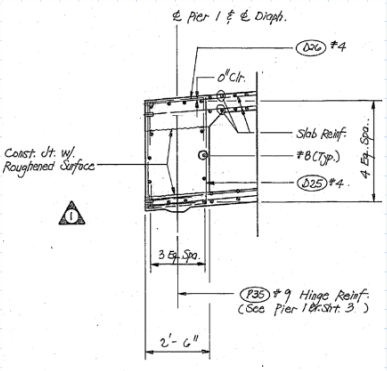
1. The purpose is to calculate and check the dowel and surrounding concrete box capacities of the shear key at Pier 2
2. The calculation is based on AASHTO 8th Edition, 2017. and ACI 318-14

2. Material Properties

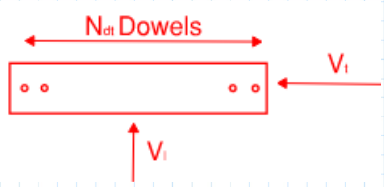
Concrete Compressive Strength	$f_c := 5 \cdot ksi$
Reinforcement Yield Strength	$f_y := 60 \cdot ksi$
Reinforcement Ultimate Tensile Strength	$f_{uta} := 90 \cdot ksi$

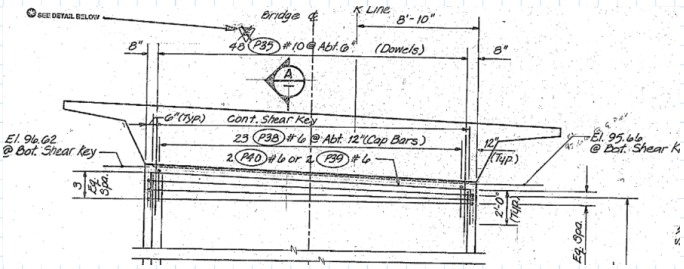
3. Structural Properties

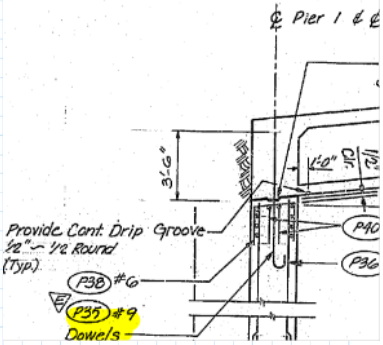
Width of diaphragm	$b := 24 \cdot ft$
Thickness of diaphragm	$t := 2.5\ ft$
Shear key top width	$b_{aa} := 9\ in$
Shear key depth	$d := 2\ in$
Shear key angle	$\alpha := 45\ deg$
Shear key bottom width	$b_{bb} := b_{aa} - 2\ d \cdot \tan(\alpha) = 5\ in$



For dowels







Dowel Size	#9
Dowel Area	$A_d := 1 \cdot in^2$
Dowel diameter	$d_d := 1.124 \cdot in$
Dowel Qty	$N_{dt} := 48$
Total Dowel Area	$N_{dt} \cdot A_d = 48\ in^2$
Dowel spacing	$s_t := \frac{b - 2 \cdot 8\ in}{N_{dt} - 1} = 5.79\ in$
Dowel distributed width	$B := b - 2 \cdot 8\ in = 22.67\ ft$

4. Demands

Longitudinal Shear Force	$V_l := 65\ kip$
Transverse Shear Force	$V_t := 72\ kip$

5. Anchor Rod Shear Resistance

Anchor rod shear resistance factor (AASHTO 5.13.2.2)	$\phi_{shear.dowel} := 0.65$
Factored anchor rod shear resistance	$V_{sa} := 0.6\ \phi_{shear.dowel} \cdot f_{uta} \cdot (N_{dt}) \cdot A_d = (1.68 \cdot 10^3)\ kip$

6. Anchor Reinforcement Shear Breakout Resistance

Thickness of member in which an anchor is located	$h_a := 4.25\ ft$	
Embedment length of dowels	$h_{ef} := 3.5\ ft$	Note: Measured in bluebeam in dwg "embedment length"

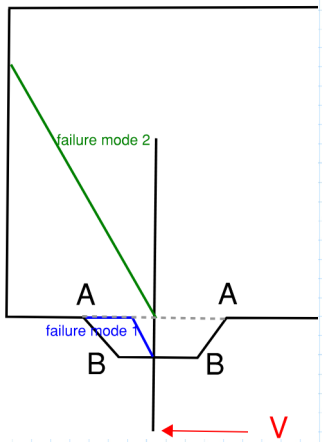
Resistance factor (ACI 17.3.3) $\phi_{concrete} := 0.75$

Load bearing length of the anchor for shear (ACI 17.5.2.2) $l_e := min\left(h_{ef}, 8 \cdot d_d\right) = 8.99\ in$

Note: assume the anchor strength is governed by concrete breakout and it's assumed to have supplementary reinforcement

For longitudinal shear

The concrete around the dowel may potentially fail in shear breakout by cracking from the start of the dowel embedment to point A (mode 1), or by cracking from the intersection between the dowel and line A-A to the face of the end diaphragm, (mode 2), as shown in the sketch on the right. We can calculate the interface shear along A-A by assuming the shear is uniformly transferred from the dowel to the surrounding concrete along the depth of the dowel embedment.



Mode 1

Interface shear demand along A-A

Depth of shear key $d = 2\ in$

Factored shear force for 1st segment $V_1 := V_l \cdot \frac{d}{h_{ef}} = 3.1\ kip$

Interface shear resistance along A-A

Cohesion factor for monolithically placed concrete(5.7.4.3) $c := 0.4 \cdot ksi$

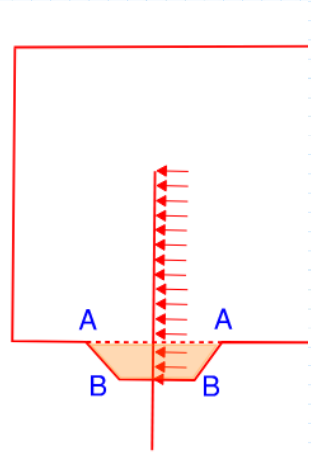
Friction factor(5.7.4.3) $\mu := 1.4$

Fraction of concrete strength available to resist interface shear (5.7.4.3) $K_1 := 0.25$

Limiting interface shear resistance (5.7.4.3) $K_2 := 1.5 \cdot ksi$

Area of concrete considered to be engaged in interface shear transfer (5.7.4.2) $A_{cv} := b \cdot \frac{b_{aa}}{2} = 9\ ft^2$

Nominal shear resistance of interface plane (5.7.4.3) $V_{ni} := c \cdot A_{cv} = 518.4\ kip$



Note: assume no compression to be conservative

The nominal shear resistance used in the design shall not exceed either of the following:

$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = \left(1.62 \cdot 10^3\right)\ kip$

$V_{ni.2} := K_2 \cdot A_{cv} = \left(1.94 \cdot 10^3\right)\ kip$

$V_{ni.limit} := min\left(V_{ni.1}, V_{ni.2}\right) = \left(1.62 \cdot 10^3\right)\ kip$

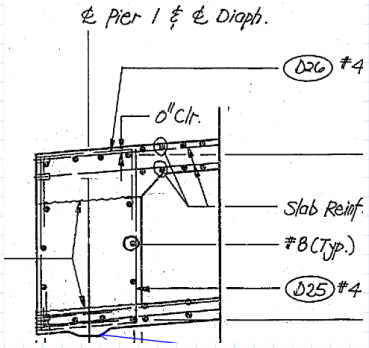
Check for shear force resistance

$$V_{ni} := \left\| \begin{array}{l} \text{if } V_{ni} \le V_{ni.limit} \\ \quad \left\| V_{ni} \right. \\ \text{if } V_{ni} > V_{ni.limit} \\ \quad \left\| V_{ni.limit} \right. \end{array} \right. = 518.4\ kip$$

Shear force resistance factor (AASHTO 5.5.4.2) $\phi_{interfaceshear} := 0.9$

Factored Interface shear resistance along A-A $V_{ri} := V_{ni} \cdot \phi_{interfaceshear} = 466.56\ kip$

D/C ratio $\frac{V_l}{V_{ri}} = 6.63 \cdot 10^{-3}$



Mode 2

Shear breakout resistance

Distance from the center of an anchor shaft to the edge of concrete in one direction $c_{a1} := \frac{t}{2} = 15\ in$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1 $c_{a2} := 8\ in$

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b)) $A_{vc} := min\left(b, \left(2 \cdot 1.5 \cdot c_{a1} + b - 2 \cdot 8\ in\right)\right) \cdot min\left(1.5 \cdot c_{a1}, h_a\right) = \left(6.48 \cdot 10^3\right)\ in^2$

Projected area for a single anchor $A_{vc0} := 4.5 \cdot {c_{a1}}^2 = \left(1.01 \cdot 10^3\right)\ in^2$

Modification factor for anchor groups loaded eccentrically (ACI 17.5.2.5) $\psi_{ecv} := 1$

Modification factor for cracked concrete (ACI 17.5.2.7) $\psi_{cv} := 1.4$ Note: for cast-in dowel

Modification factor for edge effect (ACI 17.5.2.6) $\psi_{edv} := min\left(1, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) = 0.81$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2) $V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 46.21\ kip$

The shear breakout will also be resisted by the longitudinal reinforcements in the bottom slab

Shear reinforcement sizes #8

Shear reinforcement area $A_{sr} := 0.79\ in^2$

Number of within ca1 and $N_s := 36 \cdot 2$

Resistance factor (AASHTO 5.5.4.2) $\phi_{sr} := 0.75$

Reinforcement resistance $V_{sr} := \phi_{sr} \cdot A_{sr} \cdot f_y \cdot N_s = \left(2.56 \cdot 10^3\right)\ kip$

Shear force resistance $V_{cbgl} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b + V_{sr} = \left(2.81 \cdot 10^3\right)\ kip$

D/C ratio $\frac{V_l}{V_{cbgl}} = 0.02$ *Therefore mode 2 governs*

For transverse shear

Check for edge dowel

Assuming the demand on the edge dowel is V_t / N_{dt}, where V_t is the total transverse shear and N_{dt} is the number of the dowels

Distance from the center of an anchor shaft to the edge of concrete in one direction $c_{a1} := 8\ in$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1 $c_{a2} := \frac{t}{2} = 15\ in$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2) $V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 18\ kip$

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b))

$$A_{vc} := min\left(t, \left(2 \cdot 1.5 \cdot c_{a1}\right)\right) \cdot \left(1.5 \cdot c_{a1}\right) = 288\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot \left(c_{a1}\right)^2 = 288\ in^2$$

Shear force perpendicular to the edge (ACI 17.5.2.1b)

$$V_{cbt} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 15.24\ kip$$

Demand on the edge dowel

$$V_{t.edgedowel} := \frac{V_t}{N_{dt}} = 1.5\ kip$$

D/C Ratio:

$$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.1$$

7. Concrete Pryout Strength of Anchor in Shear

Embedment Depth

$$h_{ef} := 3.5\ ft$$

Edge distance (ACI Fig R 17.4.2.1)

$$c_1 := 8\ in$$

Edge distance (ACI Fig R 17.4.2.1)

$$c_2 := \frac{t}{2} = 15\ in$$

Adjusted embedment depth

$$h_{ef't} := \max\left(\frac{s_t}{3}, \frac{\left(c_1\right)}{1.5}\right) = 0.44\ ft$$

Anchorage projection area of a single anchor (ACI 17.4.2)

$$A_{Nco} := 9 \cdot {h_{ef't}}^2 = 1.78\ ft^2$$

Anchorage projection area of a group anchor

$$A_{Nct} := min\left(t, 2 \cdot \left(min\left(c_2, 1.5 \cdot h_{ef't}\right)\right)\right) \cdot min\left(b, \left(\left(N_{dt} - 1\right) \cdot s_t + c_1 + 1.5 \cdot h_{ef't}\right)\right) = 32\ ft^2$$

Adjusted anchorage projection area of a group anchor

$$A_{Nct} := min\left(A_{Nct}, N_{dt} \cdot A_{Nco}\right) = 32\ ft^2$$

Min edge distance

$$c_{min} := min\left(c_1, 1.5 \cdot h_{ef't}\right) = 8\ in$$

Eccentricity effects (ACI 17.4.2.4)

$$\psi_{ec.N} := 1$$

Edge effects (ACI 17.4.2.5)

$$\psi_{ed.N} := min\left(0.7 + 0.3 \cdot \frac{c_{min}}{1.5 \cdot h_{ef't}}, 1\right) = 1$$

Concrete cracking (ACI 318 17.4.2.6)

$$\psi_{c.N} := 1.25$$

Concrete splitting (ACI 318 17.4.2.7)

$$\psi_{cp.N} := 1$$

Note: for cast in anchor

For normal density concrete

$$\lambda := 1$$

Basic tensile resistance (ACI 17.4.2.2)

$$N_b := 24 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot \frac{\left(\frac{h_{ef't}}{in}\right)^{1.5} \cdot kip}{1000} = 20.9\ kip$$

Concrete breakout group resistance (ACI 17.4.2)

$$N_{cbgt} := \frac{A_{Nct}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 470.3\ kip$$

Concrete breakout single rod resistance

$$N_{cbt} := \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 26.13\ kip$$

Coefficient for pryout strength (ACI 17.5.3.1)

$$k_{cp} := 2$$

Note: hef>2.5in

Coefficient for basic concrete breakout strength in tension (ACI 17.5.3.1)

$$k_c := 24$$

Note: kc is 24 for cast-in place anchors

Factored concrete pryout strength (ACI 17.5.3.1)

$$V_{cpg} := \phi_{concrete} \cdot k_{cp} \cdot \left(N_{cbgt}\right) = 705.45\ kip$$

D/C ratio $\frac{V_l}{V_{cpg}} = 0.09$ $\frac{V_t}{V_{cpg}} = 0.1$

8. Anchor Rod on Concrete Bearing

Note: calculating the bearing resistance is not required in AASHTO or ACI, but similar calculation was found in CSA S6-19 8.16.7.3. The embedment depth is 5*diameter of the anchor in S6 and is changed to 8*diameter of the anchor based on ACI

Concrete resistance (AASHTO D6.2.2) $\phi_c := 0.75$

Bearing area $A_{br} := d_d \cdot min\left(8 \cdot d_d, h_{ef}\right) = 10.11\ in^2$

Factored Concrete bearing resistance $B_r := \left(N_{dt}\right) \cdot \phi_c \cdot A_{br} \cdot f_c = \left(1.82 \cdot 10^3\right)\ kip$

D/C Ratio $\frac{V_l}{B_r} = 0.04$ $\frac{V_t}{B_r} = 0.04$

9. Summary

Governed longitudinal shear resistance for anchorage group $V_{rl} := min\left(V_{sa}, V_{cbgl}, V_{cpg}\right) = 705.45\ kip$

Governed transverse shear resistance for anchorage group $V_{rt} := min\left(V_{sa}, V_{cpg}\right) = 705.45\ kip$

Total number of dowels needed $N_{dt} = 48$

	Demands	Resistance	D/C Ratio
Longitudinal Shear on Dowel Group	$V_l = 65\ kip$	$V_{rl} = 705.45\ kip$	$\frac{V_l}{V_{ri}} = 6.63 \cdot 10^{-3}$
Transverse Shear on Dowel Group	$V_t = 72\ kip$	$V_{rt} = 705.45\ kip$	$\frac{V_t}{V_{rt}} = 0.1$
Transverse Shear on Edge Dowel	$V_{t.edgedowel} = 1.5\ kip$	$V_{cbt} = 15.24\ kip$	$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.1$
Bearing	$V_l = 65\ kip$ $V_t = 72\ kip$	$B_r = \left(1.82 \cdot 10^3\right)\ kip$	$\frac{V_l}{B_r} = 0.04$ $\frac{V_t}{B_r} = 0.04$

1. Purpose and Assumptions

1. The purpose is to calculate and check the dowel and surrounding concrete wall capacities of the shear key at Pier 2
2. The calculation is based on AASHTO 8th Edition, 2017. and ACI 318-14

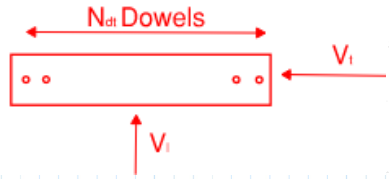
2. Material Properties

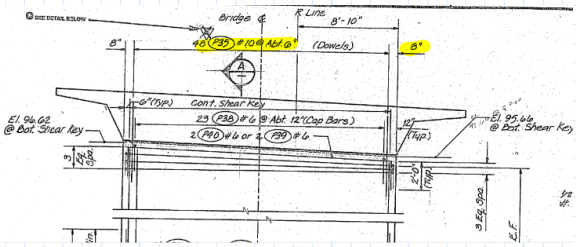
Concrete Compressive Strength	$f_c := 4 \cdot ksi$
Reinforcement Yield Strength	$f_y := 60 \cdot ksi$
Reinforcement Ultimate Tensile Strength	$f_{uta} := 90 \cdot ksi$

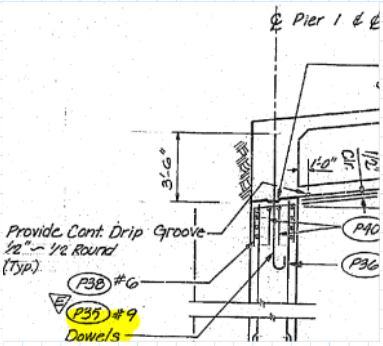
3. Structural Properties

Width of wall	$b := 24 \cdot ft$
Thickness of wall	$t := 2.5\ ft$

For dowels







Dowel Size	#9
Dowel Area	$A_d := 1 \cdot in^2$
Dowel diameter	$d_d := 1.128 \cdot in$
Dowel Qty	$N_{dt} := 48$
Total Dowel Area	$N_{dt} \cdot A_d = 48\ in^2$
Dowel spacing	$s_t := \frac{b - 2 \cdot 8\ in}{N_{dt} - 1} = 5.79\ in$
Dowel distributed width	$B := b - 2 \cdot 8\ in = 22.67\ ft$

Note: the first dowel starts at 8" from concrete face as shown in the as-built dwg. above

4. Demands

Longitudinal Shear Force	$V_l := 65\ kip$
Transverse Shear Force	$V_t := 72\ kip$

5. Anchor Rod Shear Resistance

Anchor rod shear resistance factor (AASHTO 5.13.2.2)	$\phi_{shear.dowel} := 0.65$
Factored anchor rod shear resistance	$V_{sa} := 0.6\ \phi_{shear.dowel} \cdot f_{uta} \cdot (N_{dt}) \cdot A_d = (1.68 \cdot 10^3)\ kip$

6. Anchor Reinforcement Shear Breakout Resistance

For longitudinal shear

Depth of member in which an anchor is located	$h_a := 31.66\ ft$
Embedment length of dowels	$h_{ef} := 3.5\ ft$
Resistance factor (ACI 17.3.3)	$\phi_{concrete} := 0.75$
Load bearing length of the anchor for shear (ACI 17.5.2.2)	$l_e := min(h_{ef}, 8 \cdot d_d) = 9.02\ in$

Note: Measured in bluebeam in dwg "embedment length"

Note: assume the anchor strength is governed by concrete breakout and it's assumed to have supplementary reinforcement

Distance from the center of an anchor shaft to the edge of concrete in one direction

$$c_{a1} := \frac{t}{2} = 1.25\ ft$$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$$c_{a2} := 8\ in$$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 41.4\ kip$$

Modification factor for edge effect (ACI 17.5.2.6)

$$\psi_{edv} := min\left(1, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) = 0.81$$

Modification factor for cracked concrete (ACI 17.5.2.7)

$$\psi_{cv} := 1.4$$

Note: cast-in anchors

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b))

$$A_{vc} := min\left(b, \left(2 \cdot 1.5 \cdot c_{a1} + b - 2 \cdot 8\ in\right)\right) \cdot min\left(1.5 \cdot c_{a1}, h_a\right) = \left(6.48 \cdot 10^3\right)\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot {c_{a1}}^2 = \left(1.01 \cdot 10^3\right)\ in^2$$

Modification factor for anchor groups loaded eccentrically (ACI 17.5.2.5)

$$\psi_{ecv} := 1$$

Shear force perpendicular to the edge (ACI 17.5.2.1)

$$V_{cbgl} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 224.44\ kip$$

Shear reinforcement sizes

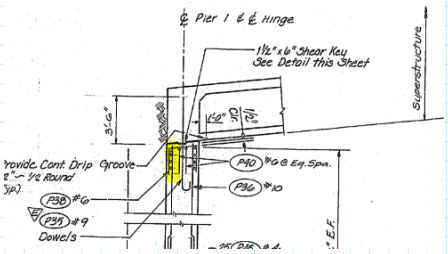
#6

Shear reinforcement area

$$A_{sr} := 0.44\ in^2$$

Number of within ca1 and

$$N_s := 2 \cdot 23 = 46$$



Resistance factor (AASHTO 5.5.4.2)

$$\phi_{sr} := 0.75$$

Reinforcement resistance

$$V_{sr} := \phi_{sr} \cdot A_{sr} \cdot f_y \cdot N_s = 910.8\ kip$$

Total shear force resistance

$$V_{cbgl'} := V_{cbgl} + V_{sr} = \left(1.14 \cdot 10^3\right)\ kip$$

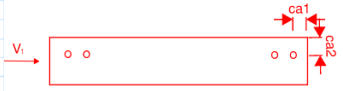
D/C Ratio

$$\frac{V_l}{V_{cbgl'}} = 0.06$$

For transverse shear

Check for edge dowel

Assuming the demand on the edge dowel is V_t / N_{dt}, where V_t is the total transverse shear and N_{dt} is the number of the dowels



Distance from the center of an anchor shaft to the edge of concrete in one direction

$$c_{a1} := 8\ in$$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$$c_{a2} := \frac{t}{2} = 15\ in$$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 16.13\ kip$$

Total projected area for a group of anchors (ACI Figure R17.5.2.1(b))

$$A_{vc} := min\left(t, \left(2 \cdot 1.5 \cdot c_{a1}\right)\right) \cdot \left(1.5 \cdot c_{a1}\right) = 288\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot \left(c_{a1}\right)^2 = 288\ in^2$$

Shear force perpendicular to the edge (ACI 17.5.2.1b)

$$V_{cbt} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 13.66\ kip$$

Demand on the edge dowel

$$V_{t.edgedowel} := \frac{V_t}{N_{dt}} = 1.5 \ kip$$

D/C Ratio:

$$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.11$$

7. Concrete Pryout Strength of Anchor in Shear

Embedment Depth

$$h_{ef} := 3.5 \ ft$$

Edge distance (ACI Fig R 17.4.2.1)

$$c_1 := 8 \ in$$

Edge distance (ACI Fig R 17.4.2.1)

$$c_2 := \frac{t}{2} = 15 \ in$$

Adjusted embedment depth

$$h_{ef't} := \max\left(\frac{s_t}{3}, \frac{\langle c_1 \rangle}{1.5}\right) = 0.44 \ ft$$

Anchorage projection area of a single anchor (ACI 17.4.2)

$$A_{Nco} := 9 \cdot {h_{ef't}}^2 = 1.78 \ ft^2$$

Anchorage projection area of a group anchor

$$A_{Nct} := min\left(t, 2 \cdot \left(min\left(c_2, 1.5 \cdot h_{ef't}\right)\right)\right) \cdot min\left(b, \left(\left(N_{dt} - 1\right) \cdot s_t + c_1 + 1.5 \cdot h_{ef't}\right)\right) = 32 \ ft^2$$

Adjusted anchorage projection area of a group anchor

$$A_{Nct} := min\left(A_{Nct}, N_{dt} \cdot A_{Nco}\right) = 32 \ ft^2$$

Min edge distance

$$c_{min} := min\left(c_1, 1.5 \cdot h_{ef't}\right) = 8 \ in$$

Eccentricity effects (ACI 17.4.2.4)

$$\psi_{ec.N} := 1$$

Edge effects (ACI 17.4.2.5)

$$\psi_{ed.N} := min\left(0.7 + 0.3 \cdot \frac{c_{min}}{1.5 \cdot h_{ef't}}, 1\right) = 1$$

Concrete cracking (ACI 318 17.4.2.6)

$$\psi_{c.N} := 1.25$$

Note: for uncracked concrete

Concrete splitting (ACI 318 17.4.2.7)

$$\psi_{cp.N} := 1$$

Note: for cast in anchor

For normal density concrete

$$\lambda := 1$$

Basic tensile resistance (ACI 17.4.2.2)

$$N_b := 24 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot \frac{\left(\frac{h_{ef't}}{in}\right)^{1.5} \cdot kip}{1000} = 18.7 \ kip$$

Concrete breakout group resistance (ACI 17.4.2)

$$N_{cbgt} := \frac{A_{Nct}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 420.65 \ kip$$

Concrete breakout single rod resistance

$$N_{cbt} := \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 23.37 \ kip$$

Coefficient for pryout strength (ACI 17.5.3.1)

$$k_{cp} := 2$$

Note: hef>2.5in

Coefficient for basic concrete breakout strength in tension (ACI 17.5.3.1)

$$k_c := 24$$

Note: kc is 24 for cast-in place anchors

Factored concrete pryout strength (ACI 17.5.3.1)

$$V_{cpg} := \phi_{concrete} \cdot k_{cp} \cdot \left(N_{cbgt}\right) = 630.98 \ kip$$

D/C Ratio

$$\frac{V_l}{V_{cpg}} = 0.1$$

$$\frac{V_t}{V_{cpg}} = 0.11$$

8. Anchor Rod on Concrete Bearing

Note: calculating the bearing resistance is not required in AASHTO or ACI, but similar calculation was found in CSA S6-19 8.16.7.3. The embedment depth is 5*diameter of the anchor in S6 and is changed to 8*diameter of the anchor based on ACI

Concrete resistance (AASHTO D6.2.2)	$\phi_c := 0.75$
Bearing area	$A_{br} := d_d \cdot min\left(8 \cdot d_d, h_{ef}\right) = 10.18\ in^2$
Factored Concrete bearing resistance	$B_r := \left(N_{dt}\right) \cdot \phi_c \cdot A_{br} \cdot f_c = \left(1.47 \cdot 10^3\right)\ kip$
D/C Ratio	$\frac{V_l}{B_r} = 0.04$ $\frac{V_t}{B_r} = 0.05$

9. Summary

Governed longitudinal shear resistance for anchorage group	$V_{rl} := min\left(V_{sa}, V_{cbgl'}, V_{cpg}\right) = 630.98\ kip$
Governed transverse shear resistance for anchorage group	$V_{rt} := min\left(V_{sa}, V_{cpg}\right) = 630.98\ kip$
Total number of dowels needed	$N_{dt} = 48$
Total number of vertical reinforcement needed within ca1	$N_s = 46$

	Demands	Resistance	D/C Ratio
Longitudinal Shear on Dowel Group	$V_l = 65\ kip$	$V_{rl} = 630.98\ kip$	$\frac{V_l}{V_{rl}} = 0.1$
Transverse Shear on Dowel Group	$V_t = 72\ kip$	$V_{rt} = 630.98\ kip$	$\frac{V_t}{V_{rt}} = 0.11$
Transverse Shear on Edge Dowel	$V_{t.edgedowel} = 1.5\ kip$	$V_{cbt} = 13.66\ kip$	$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.11$

Bearing	$V_l = 65\ kip$	$V_t = 72\ kip$	$B_r = \left(1.466 \cdot 10^3\right)\ kip$	$\frac{V_l}{B_r} = 0.04$	$\frac{V_t}{B_r} = 0.05$

1. Purpose and Assumptions

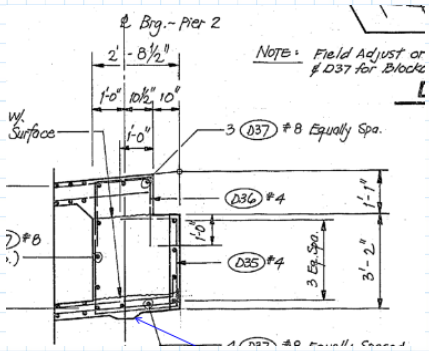
1. The purpose is to calculate and check the dowel and surrounding concrete box capacities of the shear key at Pier 3
2. The calculation is based on AASHTO 8th Edition, 2017. and ACI 318-14

2. Material Properties

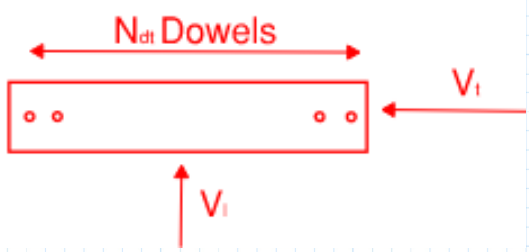
Concrete Compressive Strength	$f_c := 5 \cdot ksi$
Reinforcement Yield Strength	$f_y := 60 \cdot ksi$
Reinforcement Ultimate Tensile Strength	$f_{uta} := 90 \cdot ksi$

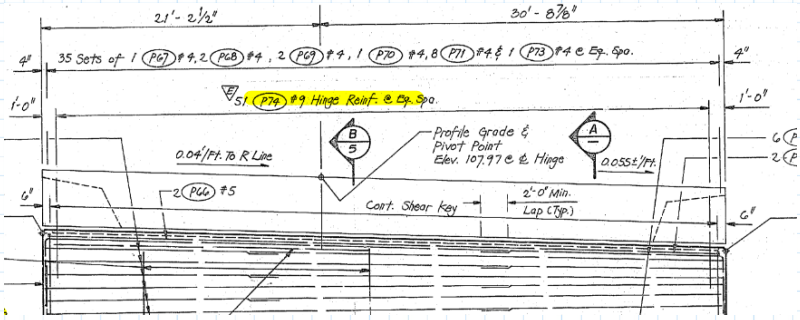
3. Structural Properties

Width of diaphragm	$b := 21\ ft + 2.5\ in + 30\ ft + 8.875\ in = 51.95\ ft$
Thickness of diaphragm	$t := 2\ ft + 8.5\ in = 2.71\ ft$
Shear key top width	$b_{aa} := 9\ in$
Shear key depth	$d := 2\ in$
Shear key angle	$\alpha := 45\ deg$
Shear key bottom width	$b_{bb} := b_{aa} - 2\ d \cdot \tan(\alpha) = 5\ in$



For dowels





Dowel Size	#9
Dowel Area	$A_d := 1 \cdot in^2$
Dowel diameter	$d_d := 1.128 \cdot in$
Dowel Qty	$N_{dt} := 51$
Total Dowel Area	$N_{dt} \cdot A_d = 51\ in^2$
Dowel spacing	$s_t := \frac{b - 2 \cdot 12\ in}{N_{dt} - 1} = 11.99\ in$
Dowel distributed width	$B := b - 2 \cdot 12\ in = 49.95\ ft$

4. Demands

Longitudinal Shear Force	$V_l := 1020\ kip$
Transverse Shear Force	$V_t := 66\ kip$

5. Anchor Rod Shear Resistance

Anchor rod shear resistance factor (AASHTO 5.13.2.2)	$\phi_{shear.dowel} := 0.65$
Factored anchor rod shear resistance	$V_{sa} := 0.6\ \phi_{shear.dowel} \cdot f_{uta} \cdot (N_{dt}) \cdot A_d = (1.79 \cdot 10^3)\ kip$
D/C ratio	$\frac{V_l}{V_{sa}} = 0.57$

6. Anchor Reinforcement Shear Breakout Resistance

Thickness of member in which an anchor is located $h_a := 4.25\ ft$

Embedment length of dowels $h_{ef} := 3.5\ ft$

Note: Measured in bluebeam in dwg "embedment length"

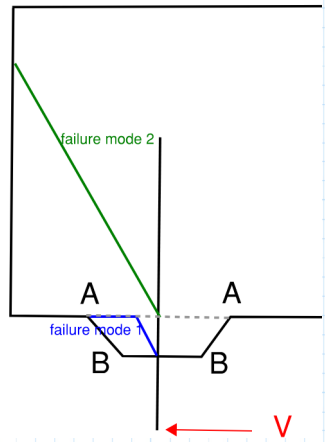
Resistance factor (ACI 17.3.3) $\phi_{concrete} := 0.75$

Load bearing length of the anchor for shear (ACI 17.5.2.2) $l_e := min\left(h_{ef}, 8 \cdot d_d\right) = 9.02\ in$

Note: assume the anchor strength is governed by concrete breakout and it's assumed to have supplementary reinforcement

For longitudinal shear

The concrete around the dowel may potentially fail in shear breakout by cracking from the start of the dowel embedment to point A (mode 1), or by cracking from the intersection between the dowel and line A-A to the face of the end diaphragm, (mode 2), as shown in the sketch on the right. We can calculate the interface shear along A-A by assuming the shear is uniformly transferred from the dowel to the surrounding concrete along the depth of the dowel embedment.



Mode 1

Interface shear demand along A-A

Depth of shear key $d = 2\ in$

Factored shear force for 1st segment $V_1 := V_l \cdot \frac{d}{h_{ef}} = 48.57\ kip$

Interface shear resistance along A-A

Cohesion factor for monolithically placed concrete(5.7.4.3) $c := 0.4 \cdot ksi$

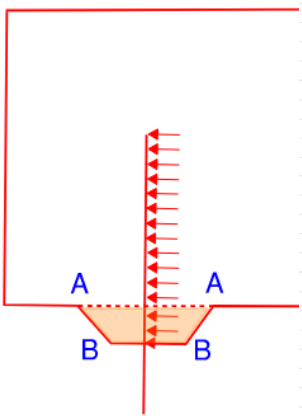
Friction factor(5.7.4.3) $\mu := 1.4$

Fraction of concrete strength available to resist interface shear (5.7.4.3) $K_1 := 0.25$

Limiting interface shear resistance (5.7.4.3) $K_2 := 1.5 \cdot ksi$

Area of concrete considered to be engaged in interface shear transfer (5.7.4.2) $A_{cv} := b \cdot \frac{b_{aa}}{2} = 19.48\ ft^2$

Nominal shear resistance of interface plane (5.7.4.3) $V_{ni} := c \cdot A_{cv} = \left(1.12 \cdot 10^3\right)\ kip$



Note: assume R=0

The nominal shear resistance used in the design shall not exceed either of the following:

$V_{ni.1} := K_1 \cdot f_c \cdot A_{cv} = \left(3.51 \cdot 10^3\right)\ kip$

$V_{ni.2} := K_2 \cdot A_{cv} = \left(4.21 \cdot 10^3\right)\ kip$

$V_{ni.limit} := min\left(V_{ni.1}, V_{ni.2}\right) = \left(3.51 \cdot 10^3\right)\ kip$

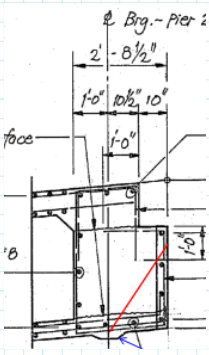
Check for shear force resistance

$$V_{ni} := \left\| \begin{array}{l} \text{if } V_{ni} \le V_{ni.limit} \\ \quad \left\| V_{ni} \right. \\ \text{if } V_{ni} > V_{ni.limit} \\ \quad \left\| V_{ni.limit} \right. \end{array} \right\| = \left(1.12 \cdot 10^3\right)\ \textit{kip}$$

Shear force resistance factor (AASHTO 5.5.4.2) $\phi_{interfaceshear} := 0.9$

Factored Interface shear resistance along A-A $V_{ri} := V_{ni} \cdot \phi_{interfaceshear} = \left(1.01 \cdot 10^3\right)\ \textit{kip}$

D/C ratio $\dfrac{V_l}{V_{ri}} = 0.05$



Failure plane

Mode 2
Shear breakout resistance

Distance from the center of an anchor shaft to the edge of concrete in one direction $c_{a1} := 10.5\ \textit{in} + 10\ \textit{in} = 20.5\ \textit{in}$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1 $c_{a2} := 12\ \textit{in}$

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b)) $A_{vc} := min\left(b\ ,\left(2 \cdot 1.5 \cdot c_{a1} + b - 2 \cdot 12\ \textit{in}\right)\right) \cdot min\left(1.5 \cdot c_{a1}\ , h_a\right) = \left(1.92 \cdot 10^4\right)\ \textit{in}^2$

Projected area for a single anchor $A_{vc0} := 4.5 \cdot {c_{a1}}^2 = \left(1.89 \cdot 10^3\right)\ \textit{in}^2$

Modification factor for anchor groups loaded eccentrically (ACI 17.5.2.5) $\psi_{ecv} := 1$

Modification factor for cracked concrete (ACI 17.5.2.7) $\psi_{cv} := 1.4$ Note: for cast-in dowel

Modification factor for edge effect (ACI 17.5.2.6) $\psi_{edv} := min\left(1\ , 0.7 + 0.3 \cdot \dfrac{c_{a2}}{1.5 \cdot c_{a1}}\right) = 0.82$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{\textit{in}} \cdot \frac{f_c}{\textit{psi}}} \cdot \frac{\left(\dfrac{c_{a1}}{\textit{in}}\right)^{1.5}}{1000} \cdot \textit{kip} = 73.96\ \textit{kip}$$

The shear breakout will also be resisted by the longitudinal reinforcement in the bottom slab

Shear reinforcement sizes *#10*

Shear reinforcement area $A_{sr} := 1.27\ \textit{in}^2$

Number of within ca1 and $N_s := 40 \cdot 2 = 80$

Resistance factor (AASHTO 5.5.4.2) $\phi_{sr} := 0.75$

Development length $l_d := 17.5\ \textit{in}$

Since ld<distance from the dowel to the face of the back of the diaphragm (10.5"+10"=20.5"), the long. reinforcement can develop up to the yield strength

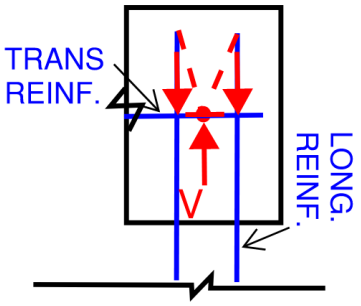
Reinforcement resistance $V_{sr} := \phi_{sr} \cdot A_{sr} \cdot f_y \cdot N_s = \left(4.57 \cdot 10^3\right)\ \textit{kip}$

Shear force resistance (ACI 17.5.2.1) $V_{cbgl} := \dfrac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b + V_{sr} = \left(5.22 \cdot 10^3\right)\ \textit{kip}$

D/C ratio $\dfrac{V_l}{V_{cbgl}} = 0.2$

Therefore mode 2 governs

The longitudinal shear force will be transferred into longitudinal reinforcement as shown in the sketch below:



load path of longitudinal shear in plan view

For transverse shear

Check for edge dowel

Assuming the demand on the edge dowel is V_t/N_{dt}, where V_t is the total transverse shear and N_{dt} is the number of the dowels

Distance from the center of an anchor shaft to the edge of concrete in one direction

$$c_{a1} := 12\ in$$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$$c_{a2} := 20.5\ in$$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 33.12\ kip$$

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b))

$$A_{vc} := min\left(t, \left(2 \cdot 1.5 \cdot c_{a1}\right)\right) \cdot \left(1.5 \cdot c_{a1}\right) = 585\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot \left(c_{a1}\right)^2 = 648\ in^2$$

Shear force perpendicular to the edge (ACI 17.5.2.1b)

$$V_{cbt} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 25.65\ kip$$

Demand on the edge dowel

$$V_{t.edgedowel} := \frac{V_t}{N_{dt}} = 1.29\ kip$$

D/C Ratio:

$$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.05$$

7. Concrete Pryout Strength of Anchor in Shear

Embedment Depth

$$h_{ef} := 3.5\ ft$$

Edge distance (ACI Fig R 17.4.2.1)

$$c_1 := 12\ in$$

Edge distance (ACI Fig R 17.4.2.1)

$$c_2 := 20.5\ in = 20.5\ in$$

Adjusted embedment depth

$$h_{ef't} := \max\left(\frac{s_t}{3}, \frac{\left(c_1\right)}{1.5}\right) = 0.67\ ft$$

Anchorage projection area of a single anchor (ACI 17.4.2)

$$A_{Nco} := 9 \cdot {h_{ef't}}^2 = 4\ ft^2$$

Anchorage projection area of a group anchor	$A_{Nct} := min\left(t, 2\cdot\left(min\left(c_2, 1.5\cdot h_{ef't}\right)\right)\right)\cdot min\left(b, \left(\left(N_{dt}-1\right)\cdot s_t + c_1 + 1.5\cdot h_{ef't}\right)\right) = 103.9\ ft^2$	
Adjusted anchorage projection area of a group anchor	$A_{Nct} := min\left(A_{Nct}, N_{dt}\cdot A_{Nco}\right) = 103.9\ ft^2$	
Min edge distance	$c_{min} := min\left(c_1, 1.5\cdot h_{ef't}\right) = 12\ in$	
Eccentricity effects (ACI 17.4.2.4)	$\psi_{ec.N} := 1$	
Edge effects (ACI 17.4.2.5)	$\psi_{ed.N} := min\left(0.7 + 0.3\cdot\frac{c_{min}}{1.5\cdot h_{ef't}}\right) = 1$	
Concrete cracking (ACI 318 17.4.2.6)	$\psi_{c.N} := 1.25$	
Concrete splitting (ACI 318 17.4.2.7)	$\psi_{cp.N} := 1$	Note: for cast in anchor
For normal density concrete	$\lambda := 1$	
Basic tensile resistance (ACI 17.4.2.2)	$N_b := 24\cdot\lambda\cdot\sqrt{\frac{f_c}{psi}}\cdot\frac{\left(\frac{h_{ef't}}{in}\right)^{1.5}\cdot kip}{1000} = 38.4\ kip$	
Concrete breakout group resistance (ACI 17.4.2)	$N_{cbgt} := \frac{A_{Nct}}{A_{Nco}}\cdot\psi_{ec.N}\cdot\psi_{ed.N}\cdot\psi_{c.N}\cdot\psi_{cp.N}\cdot N_b = \left(1.25\cdot 10^3\right)\ kip$	
Concrete breakout single rod resistance	$N_{cbt} := \psi_{ed.N}\cdot\psi_{c.N}\cdot\psi_{cp.N}\cdot N_b = 48\ kip$	
Coefficient for pryout strength (ACI 17.5.3.1)	$k_{cp} := 2$	Note: hef>2.5in
Coefficient for basic concrete breakout strength in tension (ACI 17.5.3.1)	$k_c := 24$	Note: kc is 24 for cast-in place anchors
Factored concrete pryout strength (ACI 17.5.3.1)	$V_{cpg} := \phi_{concrete}\cdot k_{cp}\cdot\left(N_{cbgt}\right) = \left(1.87\cdot 10^3\right)\ kip$	
D/C ratio	$\frac{V_l}{V_{cpg}} = 0.55$	$\frac{V_t}{V_{cpg}} = 0.04$

8. Anchor Rod on Concrete Bearing

Concrete resistance	$\phi_c := 0.75$	
Bearing area	$A_{br} := d_d\cdot min\left(8\cdot d_d, h_{ef}\right) = 10.18\ in^2$	
Factored Concrete bearing resistance	$B_r := \left(N_{dt}\right)\cdot\phi_c\cdot A_{br}\cdot f_c = \left(1.95\cdot 10^3\right)\ kip$	
D/C ratio	$\frac{V_l}{B_r} = 0.52$	$\frac{V_t}{B_r} = 0.03$

Note: calculating the bearing resistance is not required in AASHTO or ACI, but similar calculation was found in CSA S6-19 8.16.7.3. The embedment depth is 5*diameter of the anchor in S6 and is changed to 8*diameter of the anchor based on ACI

9. Summary

Governed longitudinal shear resistance for anchorage group	$V_{rl} := min\left(V_{sa}, V_{ri}, V_{cpg}\right) = \left(1.01\cdot 10^3\right)\ kip$
Governed transverse shear resistance for anchorage group	$V_{rt} := min\left(V_{sa}, V_{cpg}\right) = \left(1.79\cdot 10^3\right)\ kip$
Total number of dowels needed	$N_{dt} = 51$

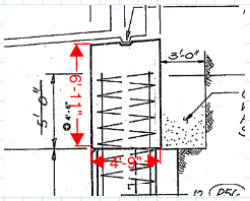
	Demands	Resistance	D/C Ratio
Longitudinal Shear on Dowel Group	$V_l = \langle 1.02 \cdot 10^3 \rangle$ *kip*	$V_{rl} = \langle 1.01 \cdot 10^3 \rangle$ *kip*	$\frac{V_l}{V_{rl}} = 1.01$
Transverse Shear on Dowel Group	$V_t = 66$ *kip*	$V_{rt} = \langle 1.79 \cdot 10^3 \rangle$ *kip*	$\frac{V_t}{V_{rt}} = 0.04$
Transverse Shear on Edge Dowel	$V_{t.edgedowel} = 1.29$ *kip*	$V_{cbt} = 25.65$ *kip*	$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.05$
Bearing	$V_l = \langle 1.02 \cdot 10^3 \rangle$ *kip* $V_t = 66$ *kip*	$B_r = \langle 1.95 \cdot 10^3 \rangle$ *kip*	$\frac{V_l}{B_r} = 0.52$ $\frac{V_t}{B_r} = 0.03$

1. Purpose and Assumptions

1. The purpose is to calculate and check the dowel and surrounding concrete beam capacities of the shear key at Pier 3
2. The calculation is based on AASHTO 8th Edition, 2017. and ACI 318-14

2. Material Properties

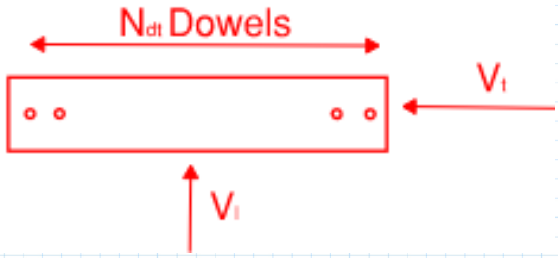
Concrete Compressive Strength	$f_c := 4 \cdot ksi$
Reinforcement Yield Strength	$f_y := 60 \cdot ksi$
Reinforcement Ultimate Tensile Strength	$f_{uta} := 90 \cdot ksi$

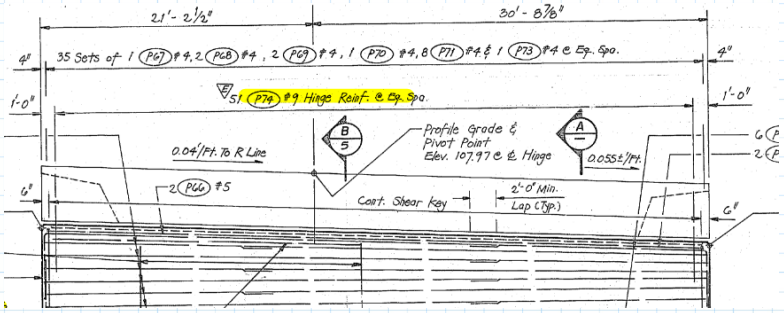


3. Structural Properties

Width of cap beam	$b := 21\ ft + 2.5\ in + 30\ ft + 8.875\ in = 51.95\ ft$
Thickness of cap beam (measured using bluebeam)	$t := 4\ ft + 9\ in = 4.75\ ft$

For dowels





Dowel Size	#9
Dowel Area	$A_d := 1 \cdot in^2$
Dowel diameter	$d_d := 1.128 \cdot in$
Dowel Qty	$N_{dt} := 51$
Total Dowel Area	$N_{dt} \cdot A_d = 51\ in^2$
Dowel spacing	$s_t := \frac{b - 2 \cdot 12\ in}{N_{dt} - 1} = 11.99\ in$
Dowel distributed width	$B := b - 2 \cdot 12\ in = 49.95\ ft$

Note: the first dowel starts at 12" from concrete face as shown in the as-built dwg. above

4. Demands

Longitudinal Shear Force	$V_l := 1020\ kip$
Transverse Shear Force	$V_t := 66\ kip$

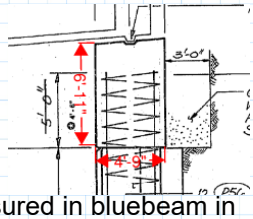
5. Anchor Rod Shear Resistance

Anchor rod shear resistance factor (AASHTO 5.13.2.2)	$\phi_{shear.dowel} := 0.65$
Factored anchor rod shear resistance	$V_{sa} := 0.6\ \phi_{shear.dowel} \cdot f_{uta} \cdot (N_{dt}) \cdot A_d = (1.79 \cdot 10^3)\ kip$

6. Anchor Reinforcement Shear Breakout Resistance

For longitudinal shear

Depth of cap beam (measured using bluebeam)	$h_a := 6\ ft + 11\ in = 6.92\ ft$
Embedment length of dowels	$h_{ef} := 3.5\ ft$
Resistance factor (ACI 17.3.3)	$\phi_{concrete} := 0.75$
Load bearing length of the anchor for shear (ACI 17.5.2.2)	$l_e := min(h_{ef}, 8 \cdot d_d) = 9.02\ in$

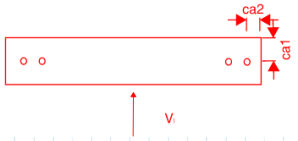


Note: Measured in bluebeam in dwg "embedment length"

Note: assume the anchor strength is governed by concrete breakout and it's assumed to have supplementary reinforcement

Distance from the center of an anchor shaft to the edge of concrete in one direction

$$c_{a1} := \frac{t}{2} = 28.5\ in$$



Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$$c_{a2} := 1\ ft$$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 108.43\ kip$$

Modification factor for edge effect (ACI 17.5.2.6)

$$\psi_{edv} := min\left(1, 0.7 + 0.3 \cdot \frac{c_{a2}}{1.5 \cdot c_{a1}}\right) = 0.78$$

Modification factor for cracked concrete (ACI 17.5.2.7)

$$\psi_{cv} := 1.4$$

Note: cast-in anchors

Total projected area for a group of anchors (ACI Figure 17.5.2.1(b))

$$A_{vc} := min\left(b, \left(2 \cdot 1.5 \cdot c_{a1} + b - 2 \cdot 8\ in\right)\right) \cdot min\left(1.5 \cdot c_{a1}, h_a\right) = \left(2.66 \cdot 10^4\right)\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot c_{a1}^{\ 2} = \left(3.66 \cdot 10^3\right)\ in^2$$

Modification factor for anchor groups loaded eccentrically (ACI 17.5.2.5)

$$\psi_{ecv} := 1$$

Shear force perpendicular to the edge (ACI 17.5.2.1)

$$V_{cbgl} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 650.99\ kip$$

Shear reinforcement sizes

#4

Shear reinforcement area

$$A_{sr} := 0.2\ in^2$$

Resistance factor (AASHTO 5.5.4.2)

$$\phi_{sr} := 0.75$$

Number of #4 rebar available

$$N_s := 3 \cdot 35 = 105$$

Reinforcement resistance

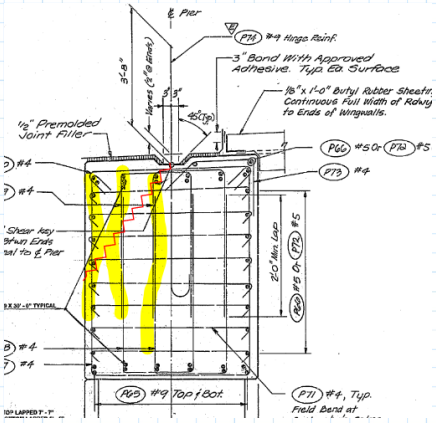
$$V_{sr} := \phi_{sr} \cdot A_{sr} \cdot f_y \cdot N_s = 945\ kip$$

Total shear force resistance

$$V_{cbgl'} := V_{cbgl} + V_{sr} = \left(1.6 \cdot 10^3\right)\ kip$$

D/C Ratio

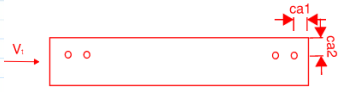
$$\frac{V_l}{V_{cbgl'}} = 0.64$$



For transverse shear

Check for edge dowel

Assuming the demand on the edge dowel is V_t / N_{dt}, where V_t is the total transverse shear and N_{dt} is the number of the dowels



Distance from the center of an anchor shaft to the edge of concrete in one direction

$$c_{a1} := 12\ in$$

Distance from the center of an anchor shaft to the edge of concrete in the direction perpendicular to ca1

$$c_{a2} := \frac{t}{2} = 28.5\ in$$

Basic concrete breakout strength in shear of a single anchor in cracked concrete (ACI 17.5.2.2)

$$V_b := 7 \cdot \left(\frac{l_e}{d_d}\right)^{0.2} \cdot \sqrt{\frac{d_d}{in} \cdot \frac{f_c}{psi}} \cdot \frac{\left(\frac{c_{a1}}{in}\right)^{1.5}}{1000} \cdot kip = 29.63\ kip$$

Total projected area for a group of anchors (ACI Figure R17.5.2.1(b))

$$A_{vc} := min\left(t, \left(2 \cdot 1.5 \cdot c_{a1}\right)\right) \cdot \left(1.5 \cdot c_{a1}\right) = 648\ in^2$$

Projected area for a single anchor

$$A_{vc0} := 4.5 \cdot \left(c_{a1}\right)^2 = 648\ in^2$$

Shear force perpendicular to the edge (ACI 17.5.2.1b)

$$V_{cbt} := \frac{A_{vc}}{A_{vc0}} \cdot \phi_{concrete} \cdot \psi_{ecv} \cdot \psi_{edv} \cdot \psi_{cv} \cdot V_b = 24.39\ kip$$

Demand on the edge dowel

$$V_{t.edgedowel} := \frac{V_t}{N_{dt}} = 1.29\ kip$$

D/C Ratio: $\frac{V_{t.edgedowel}}{V_{cbt}} = 0.05$

7. Concrete Pryout Strength of Anchor in Shear

Embedment Depth $h_{ef} := 3.5\ ft$

Edge distance (ACI 318 Figure RD5.2.1(a)) $c_1 := 12\ in$

Edge distance (ACI 318 Figure RD5.2.1(a)) $c_2 := \frac{t}{2} = 28.5\ in$

Adjusted embedment depth $h_{ef't} := \max\left(\frac{s_t}{3}, \frac{\langle c_1 \rangle}{1.5}\right) = 0.67\ ft$

Anchorage projection area of a single anchor (ACI 318 D5.2.1) $A_{Nco} := 9 \cdot {h_{ef't}}^2 = 4\ ft^2$

Anchorage projection area of a group anchor (ACI 318 D5.2.1) $A_{Nct} := min\left(t, 2 \cdot \left(min\left(c_2, 1.5 \cdot h_{ef't}\right)\right)\right) \cdot min\left(b, \left(\left(N_{dt} - 1\right) \cdot s_t + c_1 + 1.5 \cdot h_{ef't}\right)\right) = 103.9\ ft^2$

Adjusted anchorage projection area of a group anchor (ACI 318 D5.2.1) $A_{Nct} := min\left(A_{Nct}, N_{dt} \cdot A_{Nco}\right) = 103.9\ ft^2$

Min edge distance $c_{min} := min\left(c_1, 1.5 \cdot h_{ef't}\right) = 12\ in$

Eccentricity effects (ACI 17.4.2.4) $\psi_{ec.N} := 1$

Edge effects (ACI 17.4.2.5) $\psi_{ed.N} := min\left(0.7 + 0.3 \cdot \frac{c_{min}}{1.5 \cdot h_{ef't}}\right) = 1$

Concrete cracking (ACI 318 17.4.2.6) $\psi_{c.N} := 1.25$

Concrete splitting (ACI 318 17.4.2.7) $\psi_{cp.N} := 1$ Note: for cast in anchor

For normal density concrete $\lambda := 1$

Basic tensile resistance (ACI 17.4.2.2) $N_b := 24 \cdot \lambda \cdot \sqrt{\frac{f_c}{psi}} \cdot \frac{\left(\frac{h_{ef't}}{in}\right)^{1.5} \cdot kip}{1000} = 34.35\ kip$

Concrete breakout group resistance (ACI 17.4.2) $N_{cbgt} := \frac{A_{Nct}}{A_{Nco}} \cdot \psi_{ec.N} \cdot \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = \left(1.12 \cdot 10^3\right)\ kip$

Concrete breakout single rod resistance $N_{cbt} := \psi_{ed.N} \cdot \psi_{c.N} \cdot \psi_{cp.N} \cdot N_b = 42.93\ kip$

Coefficient for pryout strength (ACI 17.5.3.1) $k_{cp} := 2$ Note: hef>2.5in

Coefficient for basic concrete breakout strength in tension (ACI 17.5.3.1) $k_c := 24$ Note: kc is 24 for cast-in place anchors

Factored concrete pryout strength (ACI 17.5.3.1) $V_{cpg} := \phi_{concrete} \cdot k_{cp} \cdot \left(N_{cbgt}\right) = \left(1.67 \cdot 10^3\right)\ kip$

D/C Ratio $\frac{V_l}{V_{cpg}} = 0.61$ $\frac{V_t}{V_{cpg}} = 0.04$

8. Anchor Rod on Concrete Bearing

Concrete resistance $\phi_c := 0.75$

Bearing area $A_{br} := d_d \cdot min\left(8 \cdot d_d, h_{ef}\right) = 10.18\ in^2$

Factored Concrete bearing resistance $B_r := \left(N_{dt}\right) \cdot \phi_c \cdot A_{br} \cdot f_c = \left(1.56 \cdot 10^3\right)\ kip$

D/C Ratio $\frac{V_l}{B_r} = 0.65$ $\frac{V_t}{B_r} = 0.04$

Note: calculating the bearing resistance is not required in AASHTO or ACI, but similar calculation was found in CSA S6-19 8.16.7.3. The embedment depth is 5*diameter of the anchor in S6 and is changed to 8*diameter of the anchor based on ACI

9. Summary

Governed longitudinal shear resistance for anchorage group	$V_{rl} := min\left(V_{sa}, V_{cbgl'}, V_{cpg}\right) = \left(1.6 \cdot 10^3\right)$ ***kip***
Governed transverse shear resistance for anchorage group	$V_{rt} := min\left(V_{sa}, V_{cpg}\right) = \left(1.67 \cdot 10^3\right)$ ***kip***
Total number of dowels needed	$N_{dt} = 51$
Total number of vertical reinforcement needed within ca1	$N_s = 105$

	Demands	Resistance	D/C Ratio
Longitudinal Shear on Dowel Group	$V_l = \left(1.02 \cdot 10^3\right)$ ***kip***	$V_{rl} = \left(1.6 \cdot 10^3\right)$ ***kip***	$\frac{V_l}{V_{rl}} = 0.64$
Transverse Shear on Dowel Group	$V_t = 66$ ***kip***	$V_{rt} = \left(1.67 \cdot 10^3\right)$ ***kip***	$\frac{V_t}{V_{rt}} = 0.04$
Transverse Shear on Edge Dowel	$V_{t.edgedowel} = 1.29$ ***kip***	$V_{cbt} = 24.39$ ***kip***	$\frac{V_{t.edgedowel}}{V_{cbt}} = 0.05$

Bearing	$V_l = \left(1.02 \cdot 10^3\right)$ ***kip***	$V_t = 66$ ***kip***	$B_r = \left(1.56 \cdot 10^3\right)$ ***kip***	$\frac{V_l}{B_r} = 0.65$	$\frac{V_t}{B_r} = 0.04$