COLLEGE OF ENGINEERING
CIVIL, ARCHITECTURAL,
& ENVIRONMENTAL
MIAMI

LRFD Design Example

(2020)

Bent Cap Analysis and Design with GFRP Reinforcement

Preface

Given the document "Bent Cap Analysis and Design with GFRP Reinforcement " developed by Florida Department of Transportation (FDOT), the work of University of Miami, Department of Civil, Architectural and Environmental Engineering (CAE), is to:

- Edit the given example to account for properties of Glass Fiber Reinforced Polymer (GFRP) bars.
- Update the given example according to AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 8th Edition, 2017
- Improve the given example with notes, comments and drawings.

Coral Gables, FL
August, 2020

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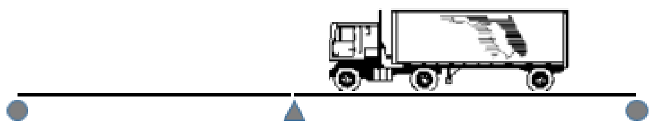
Nafiseh Kiani, PhD Candidate

Bent-Cap Analysis & Design

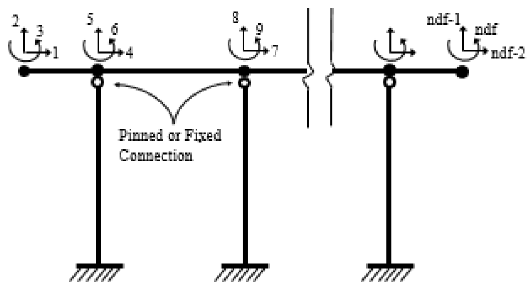
State Structures Design Office

Run the appropriate worksheets by double clicking the icons below. Modify the input data as required & execute <calculate worksheet> (Ctrl + F9) twice to save/view information. When finished, close the worksheet window <u>without saving</u> to return to this screen. Project information is stored in the Project Data File (.dat file), so Mathcad worksheets should <u>not</u> be saved, unless permanent modifications are intended.

PART 1: LOAD GENERATOR

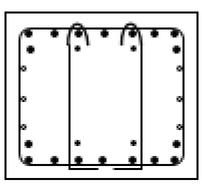


PART 2: FRAME ANALYSIS

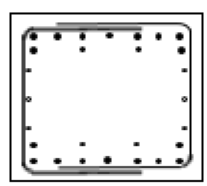


Bent Cap Analysis Model

PART 3: DESIGN & AASHTO BDS CHECKS

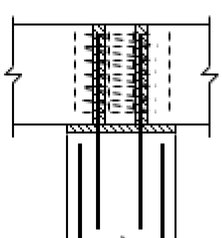


Steel Rebar

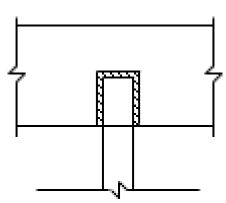


GFRP

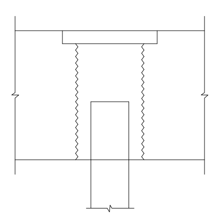
PART 4: CONNECTION DESIGN



Grouted Duct



Pile Pocket



Pile Pocket w/ CMP

For a list of assumptions/limitations of the current program, click on...... ***Program Assumptions***

For a list of recent changes to the program, click on...... ***Program Changes***

Intermediate Bent-Cap Analysis & Design
Part 1. Load Generator

FDOT
State Structures Design Office

Project =
Designed By =
Checked By =
Back Checked By =

Data Files Folder

Change Folder

D:\PhD-Nafis\2-Spring-2020\Research\FDOT Course\Bent Cap\HRB\Mathcad-01\BentCapV1.0-NK\

Open Existing Data File (optional)

Bent Cap Example - Trial 1.dat
Halls River Phase III.dat
HRB-NK.dat
testsdf.dat
US90 DS.dat
US90 Pile.dat

Refresh List

Load Data

Input Data

Superstructure (Symmetrical)

Girder type

FIB-45
FIB-54
FIB-63
FIB-72
FIB-78
FIB-84
FIB-96
CUSTOM

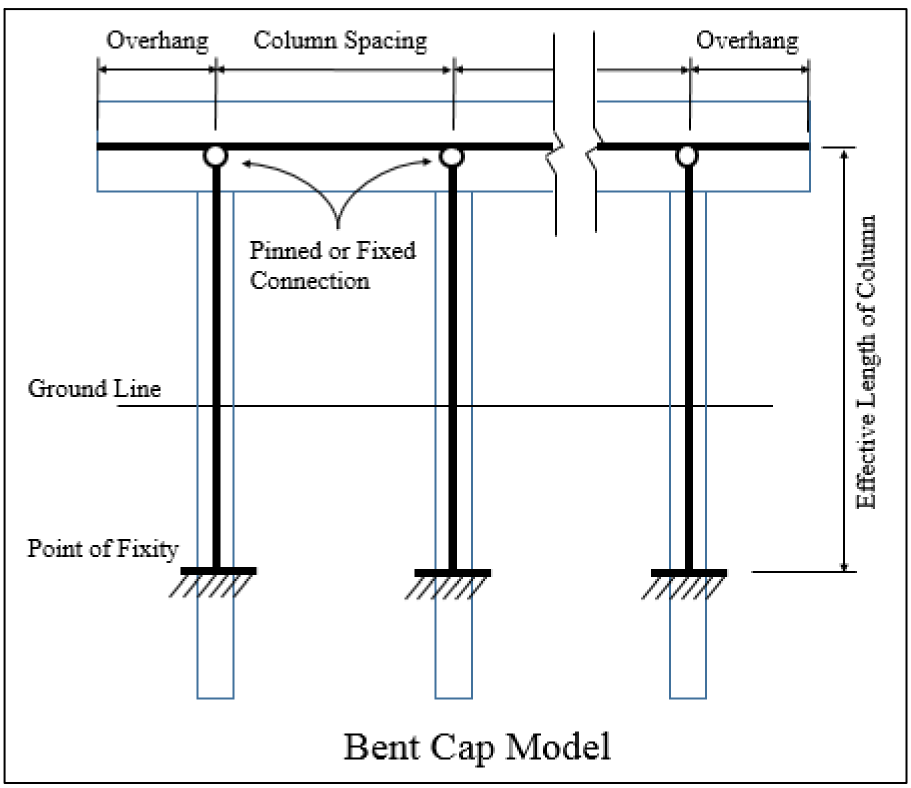
For steel or custom beams not shown under Girder type, input the beam properties under the Loads collapsed region.

Cap Skew....................................	0	*Degrees*
Average Haunch Thickness..................	1.14	*inches*
Barrier Height............................	32	*inches*
Barrier weight per SDG Table 2.2-1........	420	*lb/ft*
Slab Thickness (including sacrificial wearing surface)...........................	8.5	*inches*

Length of back station span................	37.17	*feet*		
Length of ahead station span...............	37.17	*feet*		
Total Number of Beams in Typical Section...	9			
Centerline-to-centerline beam spacing......	6.63	*feet*		
Curb-to-curb roadway width.................	40	*feet*		
Distance from coping to roadway edge.......	10.56	*feet*		

	Exterior Beam		*Interior Beam*	
Dead load of Wearing Surfaces and Utilities per beam line.............................	165	*lb/ft*	150	*lb/ft*
Additional dead load of structural components and nonstructural attachments per beam line(i.e. SIP forms).............	20	*lb/ft*	20	*lb/ft*

Substructure (Symmetrical about CL of Superstructure)



Number of columns, minimum of 2 required...	6		
Effective length of columns...............	33.7	*feet*	*See bent cap model*
Column spacing............................	10.38	*feet*	

Column type...............................	2	1 = Round 2 = Square	
Column Diameter/Width.....................	18	inches	
Cap Height................................	36	inches	
Cap Width.................................	48	inches	
Cap Length................................	59.9	feet	
Average pedestal height...................	4	inches	
Pedestal Width............................	30	inches	In direction of cap width
Pedestal Length...........................	34	inches	In direction of cap length
Beam bearing pad Length	22	inches	In direction of cap length
Min.28-day compressive strength for cap....	5.5	ksi	
Min.28-day compressive strength for cols...	6	ksi	
Correction factor for source of aggregate..	1		[SDG 1.4.1]
Concrete unit weight for calculating Ec....	0.145	kcf	
Concrete unit weight for calculating dead loads................................	0.150	kcf	[SDG Table 2.2-1]

Additional Input for Centrifugal Force (CE)

Radius of curvature of traffic lane........	0	feet	Input a Radius of 0 for bridges with no horizontal curve.

If the highway design speed is not specified, it should be conservatively taken as the maximum specified in the AASHTO publication, A Policy on Geometric Design of Highways and Streets (70 mph).

Highway design speed......................	50	mph

The total CE load shall be distributed to bent caps based on the superstructure continuity and the relative stiffness of the bent caps in the direction of CE load.

Distribution Factor for CE load to intermediate bent cap	1	Minimum 0 Maximum 1

Additional Input for Braking Force (BR)

The total BR load shall be distributed to bent caps based on the superstructure continuity and the relative stiffness of the bent caps in the direction of BR load.

Distribution Factor for BR load to intermediate bent cap	1	Minimum 0 Maximum 1	
Length of bridge for BR load calculation (length of lane load)......................	185.83	feet	(typically length of continuous deck)

Additional Input for Wind on Structure (WS)

Low Member elevation.......................	8.10	*feet*
Elevation of low ground or water level.....	0.06	*feet*
Design Wind Speed (mph) per SDG 2.4.1......	130	*mph*
Total depth of superstructure (barrier, deck, haunch, beam and superelevation).....	5.24	*feet*

Additional Input for Water Load (WA)

NOTES:

- *Current version of Load Generator focuses on design and analysis of Bent-Cap, which is typically controlled by Strength I and Service I, III limit states with water load of 100 year Basic Flood. The flood elevation is typically below the bottom of cap. Thus, calculation of WA is omitted in current version.*
- *To consider WA load (e.g. existing bridges with flood elevation above the bottom of cap), directly input the WA load acting on the bent-cap that is under consideration.*

100 year event: parallel to the bent-cap...	0	*kip*
100 year event: perpendicular to the bent-cap	0	*kip*
500 year event: parallel to the bent-cap...	0	*kip*
500 year event: perpendicular to the bent-cap	0	*kip*

Additional Input for Force Effect due to Uniform Temperature (TU)

NOTES:

- *TU load is typically perpendicular to the plane of bent-cap and thus resisted by cantilever columns. Calculation of TU is omitted in current version.*
- *To consider TU load (e.g. bridges with big skew), directly input the TU load acting on the bent-cap that is under consideration.*

TU load in the longitudinal direction of the bridge	0	*kip*

Set Output Data

Save Data File (optional)

File Name HRB-NK.dat

Note: You can specify an output folder location by using the **Change Folder** *feature above.*

Initialize Data

Intermediate Precast Bent Cap - Part 1 - Load Generator

This worksheet computes all the loads on the bent cap from the beam reactions. Live loads are generated for each beam using the lever rule and tributary area methods.

Initialize Program Data

Superstructure:

$Beam_{Type} := inpBeamType = 9$ — *Girder type*

$Skew := str2num(inpSkew)\cdot deg = 0\cdot deg$ — *Cap skew*

$t_{haunch} := str2num\left(inpt_{haunch}\right)\cdot in = 1.14\cdot in$ — *Average Haunch Thickness*

$h_{barrier} := str2num\left(inph_{barrier}\right)\cdot in = 32\cdot in$ — *Barrier Height*

$w_{barrier} := str2num\left(inpw_{barrier}\right)\, plf = 420\cdot plf$ — *Barrier weight per SDG Table 2.2-1*

$t_{slab} := str2num\left(inpt_{slab}\right)\cdot in = 8.5\cdot in$ — *Slab Thickness (including sacrificial wearing surface)*

$w_{DW.ext} := str2num\left(inpw_{DW.ext}\right)\cdot plf = 165\cdot plf$ — *Additonal DL due to wearing surface and utilities*

$w_{DW.int} := str2num\left(inpw_{DW.int}\right)\cdot plf = 150\cdot plf$

$w_{add.ext} := str2num\left(inpw_{add.ext}\right)\cdot plf = 20\cdot plf$ — *Additional DL structural components and non-structural attachments*

$w_{add.int} := str2num\left(inpw_{add.int}\right)\cdot plf = 20\cdot plf$

$L_{back.span} := str2num\left(inpL_{back.span}\right)\, ft = 37.17\, ft$ — *Length of back station span*

$L_{ahead.span} := str2num\left(inpL_{ahead.span}\right)\, ft = 37.17\, ft$ — *Length of ahead station span*

$\#Beams := str2num(inp\#Beams) = 9$ — *Total Number of Beams in Typical Section*

$Spacing_{beam} := str2num\left(inpSpacing_{beam}\right)\cdot ft = 6.6\, ft$ — *Centerline-to-centerline beam spacing*

$W_{curb.to.curb} := str2num\left(inpW_{curb.to.curb}\right)\cdot ft = 40\, ft$ — *Curb-to-curb roadway width*

$d_{coping.to.curb} := str2num(inpd_{coping.to.curb})\cdot ft = 10.56\ ft$ — *Distance from coping to roadway edge*

Substructure:

$\#Cols := str2num(inp\#Cols) = 6$ — *Number of columns, minimum of two required*

$L_{col} := str2num(inpL_{col})\cdot ft = 33.7\ ft$ — *Column effective length*

$Spacing_{col} := str2num(inpSpacing_{col})\cdot ft = 10.38\ ft$ — *Column spacing*

$ColType := str2num(inpColType) = 2$ — *Column type: 1-round, 2-square*

$W_{col} := str2num(inpW_{col})\cdot in = 18\cdot in$ — *Column Diameter/Width*

$H_{cap} := str2num(inpH_{cap})\cdot in = 36\cdot in$ — *Cap Height*

$W_{cap} := str2num(inpW_{cap})\cdot in = 48\cdot in$ — *Cap Width*

$L_{cap} := str2num(inpL_{cap})\cdot ft = 59.9\ ft$ — *Cap Length*

$H_{pedestal} := str2num(inpH_{pedestal})\cdot in = 4\cdot in$ — *Pedestal Height*

$W_{pedestal} := str2num(inpW_{pedestal})\cdot in = 30\cdot in$ — *Pedestal Width*

$L_{pedestal} := str2num(inpL_{pedestal})\cdot in = 34\cdot in$ — *Pedestal Length*

$L_{pad} := str2num(inpL_{pad})\cdot in = 22\cdot in$ — *Beam Bearing Pad Length along length of cap*

$f_{c.cap} := str2num(inpf_{c.cap})\ ksi = 5.5\cdot ksi$ — *Min.28-day compressive strength for cap*

$f_{c.col} := str2num(inpf_{c.col})\ ksi = 6.0\cdot ksi$ — *Min.28-day compressive strength for columns*

$K_1 := str2num(inpK_1) = 1$ — *Correction factor when claculating Ec*

$w_c := str2num(inpw_c)\cdot 1000pcf = 145\cdot pcf$ — *Unit weight of concrete for calculating Ec*

$\gamma_{conc} := str2num(inp\gamma_{conc})\cdot 1000pcf = 150\cdot pcf$ — *Unit weight of concrete for calculating dead loads*

Additional Input for Centrifugal Force (CE)

$Radius := str2num(inpRadius)\cdot ft = 0\ ft$ — *Radius of curvature of traffic lane (ft)*

$V_{design.speed} := str2num(inpV_{design.speed})\cdot mph = 73.33\ \frac{ft}{s}$ — *Highway design speed (ft/s)*

$DF_{CE} := str2num(inpDF_{CE}) = 1$ — *Distribution Factor for CE load to intermediate bent cap*

Additional Input for Braking Force (BR)

$DF_{BR} := str2num(inpDF_{BR}) = 1$ — *Distribution Factor for BR load to intermediate bent cap*

$L_{bridge.BR} := str2num(inpL_{bridge.BR}) \cdot ft = 185.83\ ft$ — *Length of bridge for BR load calculation*

Additional Input for Wind on Structure (WS)

$Elev_{LM} := str2num(inpElev_{LM}) \cdot ft = 8.1\ ft$ — *Low Member Elevation*

$Elev_{ground} := str2num(inpElev_{ground}) \cdot ft = 0.06\ ft$ — *Elevation of low ground or water level*

$V_{max} := str2num(inpV_{max}) \cdot mph = 130 \cdot mph$ — *Maximum Wind Speed (mph) per SDG 2.4.1-2*

$Depth_{super} := str2num(inpDepth_{super}) \cdot ft = 5.24\ ft$ — *Total depth of superstructure*

Additional Input for Water Load (WA)

$P_{WA.x.100} := str2num(inpP_{WA.x.100}) \cdot kip = 0 \cdot kip$ — *100 year event: parallel (x-direction) to the cap*

$P_{WA.z.100} := str2num(inpP_{WA.z.100}) \cdot kip = 0 \cdot kip$ — *100 year event: perpendicular (z-direction) to the cap*

$P_{WA.x.500} := str2num(inpP_{WA.x.500}) \cdot kip = 0 \cdot kip$ — *500 year event: parallel (x-direction) to the cap*

$P_{WA.z.500} := str2num(inpP_{WA.z.500}) \cdot kip = 0 \cdot kip$ — *500 year event: perpendicular (z-direction) to the cap*

Additional Input for Force Effect due to Uniform Temperature (TU)

$P_{TU} := str2num(inpP_{TU}) \cdot kip = 0 \cdot kip$ — *TU load in the longitudinal direaction of bridge*

Initialize Data

Loads: DC&DW, LL, CE, BR, WS, WL, WA, TU

$$E_{cap} := 120000 K_1 \cdot \left(\frac{w_c}{1000pcf}\right)^2 \cdot \left(\frac{f_{c.cap}}{ksi}\right)^{0.33} \cdot ksi = 4428 \cdot ksi$$

Bent cap modulus of elasticity [LRFD 5.4.2.4]

$$E_{col} := 120000 K_1 \cdot \left(\frac{w_c}{1000pcf}\right)^2 \cdot \left(\frac{f_{c.col}}{ksi}\right)^{0.33} \cdot ksi = 4557 \cdot ksi$$

Column modulus of elasticity

$$L_{span} := \frac{L_{back.span} + L_{ahead.span}}{2} = 37.17\ ft$$

Tributary Span Length

$$W_{trans.bridge} := d_{coping.to.curb} \cdot 2 + W_{curb.to.curb} = 61.12\ ft$$

Total bridge width

$$d_{deck.oh} := \frac{W_{trans.bridge} - (\#Beams - 1) \cdot Spacing_{beam}}{2} = 4.04\ ft$$

Distance from coping to centerline of first beam

$$L_{cap.oh} := \frac{L_{cap} - (\#Cols - 1) \cdot Spacing_{col}}{2} = 4\ ft$$

Length of cap overhang - See Bent Cap Model

Beam Properties

For steel or custom beams, input beam height $h_{beam} := 21.3in$

For steel or custom beams, input beam top flange width $tf_{beam} := 24.38in$

For steel or custom beams, input beam self weight $w_{beam} := 181plf$

Properties :=

"Beam"	"Height"	"top flange"	"Self weight"
"AASHTO II"	36	12	384.4
"FIB 36"	36	48	840.2
"FIB 45"	45	48	905.8
"FIB 54"	54	48	971.4
"FIB 63"	63	48	1037.1
"FIB 72"	72	48	1102.7
"FIB 78"	78	48	1146.4
"FIB 84"	84	48	1190.2
"FIB 96"	96	48	1277.7
"Custom"	$\frac{h_{beam}}{in}$	$\frac{tf_{beam}}{in}$	$\frac{w_{beam}}{plf}$

$h_{beam} := Properties_{inpBeamType+2,2} \cdot in$ $h_{beam} = 21.3 \cdot in$

$tf_{beam} := Properties_{inpBeamType+2,3} \cdot in$ $tf_{beam} = 24.38 \cdot in$

$w_{beam} := Properties_{inpBeamType+2,4} \cdot plf$ $w_{beam} = 181 \cdot plf$

Dead Loads at Each Beam Reaction (DC) [LRFD 3.5.1]

$\gamma_{conc} = 150 \cdot pcf$ *Concrete density (SDG Table 2.2-1)*

Interior girders:

$$w_{DC.int.beam} := \left(Spacing_{beam} \cdot t_{slab} + t_{haunch} \cdot tf_{beam}\right) \cdot \gamma_{conc} + w_{beam} = 914 \cdot plf$$

Exterior girders:

$$w_{DC.ext.beam} := \left[\left(d_{deck.oh} + Spacing_{beam} \cdot 0.5\right) \cdot \left(t_{slab}\right) + t_{haunch} \cdot tf_{beam}\right] \cdot \gamma_{conc} + w_{beam} = 991 \cdot plf$$

assign DC loads into a per girder vector

$i := 1 .. \#Beams$

$$P_{DC.beam_i} := \begin{cases} L_{span}\cdot\left(w_{DC.ext.beam} + \frac{2\cdot w_{barrier}}{\#Beams} + w_{add.ext}\right) & \text{if } i = 1 \vee i = \#Beams \\ L_{span}\cdot\left(w_{DC.int.beam} + \frac{2\cdot w_{barrier}}{\#Beams} + w_{add.int}\right) & \text{otherwise} \end{cases}$$

Dead Load Reactions of structural components and non-structural attachments at each beam.

$$P_{DC.beam}^T = (41.06 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 41.06)\cdot kip$$

Dead Loads at Each Beam Reaction (DW) [LRFD 3.5.1]

$$P_{DW.beam_i} := \begin{cases} L_{span}\cdot w_{DW.ext} & \text{if } i = 1 \vee i = \#Beams \\ L_{span}\cdot w_{DW.int} & \text{otherwise} \end{cases}$$

Dead load Reaction of Wearing Surfaces and Utilities at each beam

$$P_{DW.beam}^T = (6.13 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 6.13)\cdot kip$$

Dead Load of Substructure (DC) [LRFD 3.5.1]

$Cap_{self.weight} := \gamma_{conc}\cdot H_{cap}\cdot W_{cap}\cdot L_{cap} = 107.82\cdot kip$ *Bent cap self weight*

$Pedestals_{self.weight} := \gamma_{conc}\cdot W_{pedestal}\cdot L_{pedestal}\cdot H_{pedestal}\cdot \#Beams = 3.19\cdot kip$ *Pedestals self weight*

$w_{DC.cap} := \frac{Cap_{self.weight} + Pedestals_{self.weight}}{L_{cap}} = 1.85\cdot klf$ *Total substructure dead load*

Live Loads (LL) [LRFD 3.6.1]

$Impact := 1.33$ *Dynamic Load Allowance*

$w_{lane} := 0.64\cdot klf$ *Design lane load*

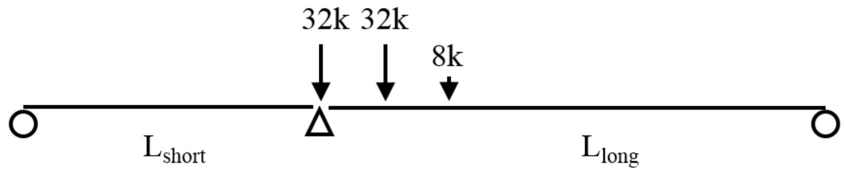
$P_{lane} := w_{lane}\cdot L_{span} = 23.79\cdot kip$ *Total lane load on bent cap*

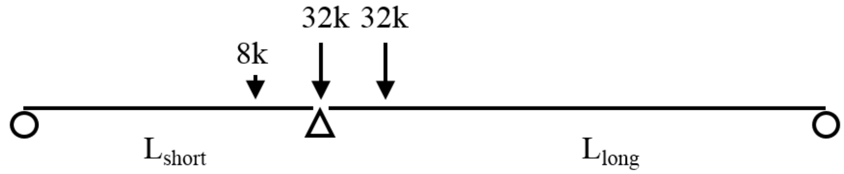
$L_{short} := \min(L_{ahead.span}, L_{back.span}) = 37.17\ ft$ *Shorter span length*

$L_{long} := \max(L_{ahead.span}, L_{back.span}) = 37.17\ ft$ *Longer span length*

$P_{tandem} := 25kip + 25kip\cdot \max\left[\left(\frac{L_{long} - 4ft}{L_{long}}\right), 0\right] = 47.31\cdot kip$ *Load on the bent cap due to one design tandem*

For maximum reaction of one design truck at an interior bent cap, place design truck at the following two locations

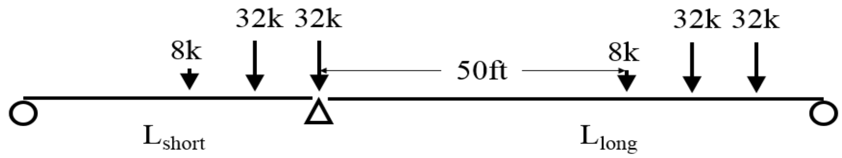




Load on the cap due to one design truck

$$P_{one.truck} := 32kip + 32kip \cdot max\left[\left(\frac{L_{long} - 14ft}{L_{long}}\right), 0\right] + 8kip \cdot max\left[\left(\frac{L_{long} - 28ft}{L_{long}}\right), \left(\frac{L_{short} - 14ft}{L_{short}}\right), 0\right] = 56.93 \cdot kip$$

For maximum reaction of truck train at an interior bent cap, place design trucks at the following location



Load on the cap due to two design trucks

$$P_{two.trucks} := 32kip + 32kip \cdot max\left[\left(\frac{L_{short} - 14ft}{L_{short}}\right), 0\right] + 8kip \cdot max\left[\left(\frac{L_{short} - 28ft}{L_{short}}\right), 0\right] \ldots = 53.9$$
$$+ 8kip \cdot max\left[\left(\frac{L_{long} - 50ft}{L_{long}}\right), 0\right] + 32kip \cdot max\left[\left(\frac{L_{long} - 64ft}{L_{long}}\right), 0\right] + 32kip \cdot max\left[\left(\frac{L_{long} - 78ft}{L_{long}}\right), 0\right]$$

Live load + impact per lane of traffic loaded. May be design tandem with lane load, one truck load with lane load or two trucks load with lane load, whichever controls. This load multiplied by the distribution and multiple presence factors will determine the live load from each beam to the bent cap..

$$P_{tandem.and.lane} := P_{lane} + Impact \cdot P_{tandem} = 86.71 \cdot kip$$

$$P_{one.truck.and.lane} := P_{lane} + Impact \cdot P_{one.truck} = 99.51 \cdot kip$$

$$P_{two.trucks.and.lane} := 0.9 \cdot \left(P_{lane} + Impact \cdot P_{two.trucks}\right) = 85.95 \cdot kip$$

$$P_{liveload.per.lane} := max\left(P_{tandem.and.lane}, P_{one.truck.and.lane}, P_{two.trucks.and.lane}\right)$$

$$P_{liveload.per.lane} = 99.51 \cdot kip$$

$$X_{beam} := \begin{array}{|l} x_1 \leftarrow d_{deck.oh} \\ \text{for } i \in 2..\#Beams \\ \quad x_i \leftarrow x_{i-1} + Spacing_{beam} \\ x \end{array}$$

$$X_{beam}^{T} = (4.04 \quad 10.67 \quad 17.3 \quad 23.93 \quad 30.56 \quad 37.19 \quad 43.82 \quad 50.45 \quad 57.08)\ ft$$

$$\#Lanes := floor\left(\frac{W_{curb.to.curb}}{12 \cdot ft}\right) = 3$$

Number of design lanes (LRFD 3.6.2)

LRFD 3.6.1.2 The loads shall be assumed to occupy 10.0-ft transversely within a design lane
LRFD 3.6.1.2.4 The design lane load shall consist of a load of 0.64-klf uniformly distributed in the longitudinal direction. Transversely, the design lane load shall be assumed to be uniformly distributed over a 10.0-ft width..
LRFD 3.6.1.3.1 The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than 2.0-ft from the edge of the design lane.

When one lane is loaded, the distance the left wheel can travel $W_{curb.to.curb} - 4ft - 6ft = 30\ ft$

For 0.5ft station increments, the required number of increments is

$$\#Increments_{liveload} := Ceil\left(\frac{W_{curb.to.curb} - 10ft}{0.5ft}, 2\right) = 60$$

To reduce computing time for really wide bridges, limit $\#Increments_{liveload}$ *to a max of 80. And a min of 10 for really narrow bridges.*

$$\#Increments_{liveload} := min(max(\#Increments_{liveload}, 10), 80) = 60$$

Station increment $$\Delta x1 := \frac{W_{curb.to.curb} - 10ft}{\#Increments_{liveload} - 1} = 0.51\ ft$$

When two lanes are loaded, first place the second truck in the left most position of the 2nd lane (wheel load 2 ft from the lane left edge) and move the 1st truck from the left most to the right most position within the 1st lane at 0.5ft increments (4 stations at 0, 0.5, 1, 1.5ft). Then move the two trucks together across the bridge. Similarly at the right end of the bridge.

When two lanes are loaded, the distance the two trucks travel together is $W_{curb.to.curb} - 2 \cdot 12ft = 16\ ft$

Station increment $$\Delta x2 := \frac{W_{curb.to.curb} - 2 \cdot 12ft}{\#Increments_{liveload} - 8 - 1} = 0.31\ ft$$

When three lanes are loaded, place the 1st truck in the right most position of 1st lane, 2nd truck in the left most postion of 2nd lane and 3rd truck in the left position of 3rd lane while traveling across the bridge with half of the total number of increments. Then place the 2nd truck in the right most position of 2nd lane (1st truck right most and 3rd truck left most) while travelling across the bridge.
When three lanes are loaded, the distance the three trucks can travel together is $W_{curb.to.curb} - 3 \cdot 12ft = 4\ ft$

Station increment $$\Delta x3 := \frac{W_{curb.to.curb} - 3 \cdot 12ft}{0.5\#Increments_{liveload} - 1} = 0.14\ ft$$

Four or more lanes loaded generally does not control due to a lower multipresence factor. Thus NOT considered.

$\#Lanes_{LL} := \min(3, \#Lanes) = 3$

Max number of design lanes considered for LL

Function below creates a matrix of wheel load point distances (referenced from the left coping), for each case of loaded lanes (rows).

location of each truck wheel measured from the bridge coping

i = Index number = increment
j = Lanes Loaded = row number

```
fX_live.load(i) := | if #Lanes_LL ≥ 3
                   |   | I ← i  if i ≤ 0.5#Increments_liveload
                   |   | I ← i − 0.5#Increments_liveload  otherwise
                   |   | x_3,1 ← d_coping.to.curb + 4ft + Δx3·(I − 1)
                   |   | x_3,3 ← d_coping.to.curb + 12ft + 2ft + Δx3·(I − 1)  if i ≤ 0.5#Increments_liveload
                   |   | x_3,3 ← d_coping.to.curb + 12ft + 4ft + Δx3·(I − 1)  otherwise
                   |   | x_3,5 ← d_coping.to.curb + 24ft + 2ft + Δx3·(I − 1)
                   |   | for j ∈ 1..3
                   |   |   x_3,2·j ← x_3,2·j−1 + 6ft
                   | if #Lanes_LL ≥ 2
                   |   | if i ≤ 4
                   |   |   | x_2,1 ← d_coping.to.curb + 2ft + 0.5ft·(i − 1)
                   |   |   | x_2,3 ← d_coping.to.curb + 12ft + 2ft
                   |   | if 4 < i ≤ #Increments_liveload − 4
                   |   |   | x_2,1 ← d_coping.to.curb + 4·ft + Δx2·(i − 4 − 1)
                   |   |   | x_2,3 ← d_coping.to.curb + 12ft + 2ft + Δx2·(i − 4 − 1)
                   |   | otherwise
                   |   |   | x_2,1 ← d_coping.to.curb + W_curb.to.curb − 24ft + 4·ft
                   |   |   | x_2,3 ← d_coping.to.curb + W_curb.to.curb − 12ft + 2ft + 0.5ft·[i − (#Increments_liveload − 4)]
                   |   | for j ∈ 1..2
                   |   |   x_2,2·j ← x_2,2·j−1 + 6ft
                   | if #Lanes_LL ≥ 1
                   |   | x_1,1 ← d_coping.to.curb + 2ft + Δx1·(i − 1)
                   |   | x_1,2 ← x_1,1 + 6ft
                   | x
```

$$fX_{live.load}(1) = \begin{pmatrix} 12.56 & 18.56 & 0 & 0 & 0 & 0 \\ 12.56 & 18.56 & 24.56 & 30.56 & 0 & 0 \\ 14.56 & 20.56 & 24.56 & 30.56 & 36.56 & 42.56 \end{pmatrix} ft$$

$$fX_{live.load}(\#Increments_{liveload}) = \begin{pmatrix} 42.56 & 48.56 & 0 & 0 & 0 & 0 \\ 30.56 & 36.56 & 42.56 & 48.56 & 0 & 0 \\ 18.56 & 24.56 & 30.56 & 36.56 & 40.56 & 46.56 \end{pmatrix} ft$$

Function below creates a matrix of distribution factors for a given increment i given the number of lanes loaded and beam number. The size of the matrix is number of lanes loaded (rows) by number of beams (columns).

```
fDF_wheel.load(i) := | X ← fX_live.load(i)
                     | for j ∈ 1 .. #Lanes_LL
                     |   for k ∈ 1 .. #Beams
                     |     | df_(j,k) ← 0
                     |     | for m ∈ 1 .. j·2
                     |     |   | if k = 1
                     |     |   |   | df_(j,k) ← df_(j,k) + 1.0  if X_(j,m) ≤ Xbeam_k
                     |     |   |   | df_(j,k) ← df_(j,k) + (Xbeam_(k+1) − X_(j,m)) / Spacing_beam  if Xbeam_k < X_(j,m) ≤ Xbeam_(k+1)
                     |     |   | if 2 ≤ k < #Beams
                     |     |   |   | df_(j,k) ← df_(j,k) + (X_(j,m) − Xbeam_(k−1)) / Spacing_beam  if Xbeam_(k−1) ≤ X_(j,m) < Xbeam_k
                     |     |   |   | df_(j,k) ← df_(j,k) + (Xbeam_(k+1) − X_(j,m)) / Spacing_beam  if Xbeam_k ≤ X_(j,m) < Xbeam_(k+1)
                     |     |   | if k = #Beams
                     |     |   |   | df_(j,k) ← df_(j,k) + (X_(j,m) − Xbeam_(k−1)) / Spacing_beam  if Xbeam_(k−1) ≤ X_(j,m) < Xbeam_k
                     |     |   |   | df_(j,k) ← df_(j,k) + 1.0  if X_(j,m) ≥ Xbeam_k
                     | df
```

$$fDF_{wheel.load}(1) = \begin{pmatrix} 0 & 0.71 & 1.1 & 0.19 & 0 & 0 & 0 & 0 & 0 \\ 0 & 0.71 & 1.1 & 1.1 & 1.1 & 0 & 0 & 0 & 0 \\ 0 & 0.41 & 1.1 & 1.4 & 1.19 & 1.1 & 0.81 & 0 & 0 \end{pmatrix}$$

Multiple Presence Load Intensity Reduction Factor (LRFD 3.12.1

```
fMP_factor(i) := | 1.2   if i = 1
                 | 1.0   if i = 2
                 | 0.85  if i = 3
                 | 0.65  if i ≥ 4
```

Function below computes the service load for each beam (columns) and number of lanes loaded (row) for a given live *load increment i.*

$$fP_{ServiceI}(i) := \begin{array}{|l} DF \leftarrow fDF_{wheel.load}(i) \\ \text{for } j \in 1..\#Lanes_{LL} \\ \quad \text{for } k \in 1..\#Beams \\ \qquad p_{j,k} \leftarrow 1.0 \cdot fMP_{factor}(j) \cdot P_{liveload.per.lane} \cdot \dfrac{DF_{j,k}}{2} \\ p \end{array}$$

$$fP_{ServiceI}(1) = \begin{pmatrix} 0 & 42.69 & 65.38 & 11.35 & 0 & 0 & 0 & 0 & 0 \\ 0 & 35.57 & 54.48 & 54.48 & 54.48 & 0 & 0 & 0 & 0 \\ 0 & 17.48 & 46.31 & 59.07 & 50.33 & 46.31 & 34.25 & 0 & 0 \end{pmatrix} \cdot kip$$

Function below finds the maximum service loads per beam with the corresponding increment and number of lanes loaded

$$Max_{liveload.beam} := \begin{array}{|l} \text{for } k \in 1..\#Beams \\ \quad M_{1,k} \leftarrow k \\ \quad maxP \leftarrow 1 \cdot kip \\ \quad \text{for } i \in 1..\#Increments_{liveload} \\ \qquad P \leftarrow fP_{ServiceI}(i) \\ \qquad \text{for } j \in 1..\#Lanes_{LL} \\ \qquad \quad \text{if } P_{j,k} > maxP \\ \qquad \qquad maxP \leftarrow P_{j,k} \\ \qquad \qquad M_{2,k} \leftarrow \dfrac{P_{j,k}}{kip} \\ \qquad \qquad M_{3,k} \leftarrow i \\ \qquad \qquad M_{4,k} \leftarrow j \\ M \end{array}$$

$$Max_{liveload.beam} = \begin{pmatrix} 1 & 2 & 3 & 4 & 5 & 6 & 7 & 8 & 9 \\ 0 & 42.7 & 65.4 & 73.6 & 73.9 & 73.6 & 65.4 & 42.7 & 0 \\ 0 & 1 & 1 & 16 & 37 & 45 & 51 & 60 & 0 \\ 0 & 1 & 1 & 2 & 2 & 2 & 1 & 1 & 0 \end{pmatrix} \qquad \begin{pmatrix} \text{"Beam number"} \\ \text{"Max beam LL (kip)"} \\ \text{"Increment"} \\ \text{"number of lanes loaded"} \end{pmatrix}$$

Matrix of Live Loads at each girder

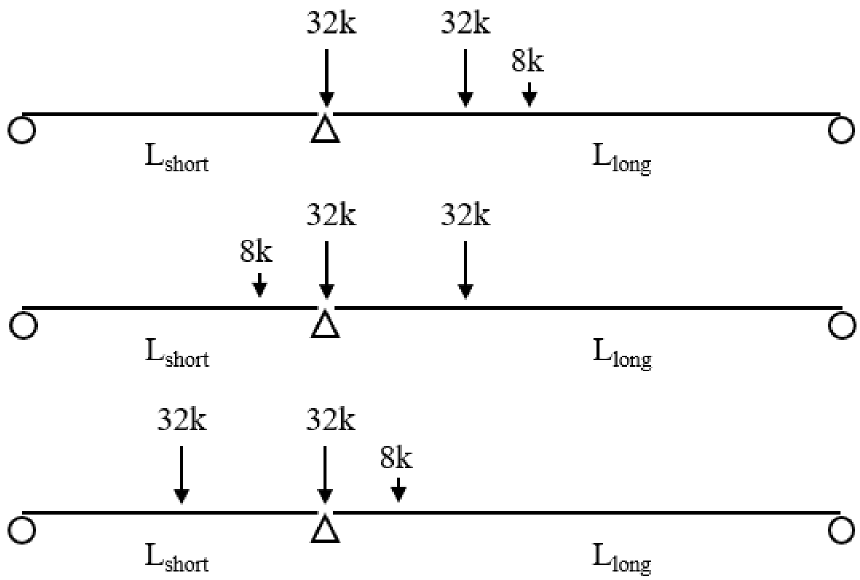
$$mP_{LL.beam} := \begin{array}{|l} p_{service} \leftarrow fP_{ServiceI}(1) \\ \text{for } i \in 2..\#Increments_{liveload} \\ \quad p_{service} \leftarrow stack\left(p_{service}, fP_{ServiceI}(i)\right) \\ p_{service} \end{array}$$

$mP_{LL.beam} = \text{BigMatrix}\{180,9\}\ \text{lbf}$

Fatigue Loads (FatLL) [LRFD 3.6.1]

$ImpactFat := 1.15$ *Dynamic Load Allowance*

For maximum reaction of one fatigue truck at an interior bent cap, place truck at the following three locations



Load on the cap due to one fatigue truck

$$P_{fat.truck1} := 32kip + 32kip \cdot \max\left[\left(\frac{L_{long} - 30ft}{L_{long}}\right), 0\right] + 8kip \cdot \max\left[\left(\frac{L_{long} - 44ft}{L_{long}}\right), 0\right] = 38.17 \cdot kip$$

$$P_{fat.truck2} := 32kip + 32kip \cdot \max\left[\left(\frac{L_{long} - 30ft}{L_{long}}\right), 0\right] + 8kip \cdot \max\left[\left(\frac{L_{short} - 14ft}{L_{short}}\right), 0\right] = 43.16 \cdot kip$$

$$P_{fat.truck3} := 32kip + 32kip \cdot \max\left[\left(\frac{L_{short} - 30ft}{L_{short}}\right), 0\right] + 8kip \cdot \max\left[\left(\frac{L_{long} - 14ft}{L_{long}}\right), 0\right] = 43.16 \cdot kip$$

$$P_{fat.truck} := \max\left(P_{fat.truck1}, P_{fat.truck2}, P_{fat.truck3}\right) = 43.16 \cdot kip$$

$$P_{liveload.fat} := P_{fat.truck} \cdot ImpactFat = 49.63 \cdot kip$$

Location of each truck wheel measured from the bridge coping

$$fX_{live.load.fat}(i) := \left| \begin{array}{l} x_1 \leftarrow d_{coping.to.curb} + 2ft + \Delta x1 \cdot (i - 1) \\ x_2 \leftarrow x_1 + 6ft \\ x \end{array} \right.$$

$$fX_{live.load.fat}(1) = \begin{pmatrix} 12.56 \\ 18.56 \end{pmatrix} ft \qquad fX_{live.load.fat}(\#Increments_{liveload}) = \begin{pmatrix} 42.56 \\ 48.56 \end{pmatrix} ft$$

Function below creates a matrix of distribution factors for a given increment i given the number of lanes loaded and beam number. The size of the matrix is number of lanes loaded (rows) by number of beams (columns).

$$fDF_{wheel.load.fat}(i) := \left| \begin{array}{l} X \leftarrow fX_{live.load.fat}(i) \\ \text{for } k \in 1..\#Beams \\ \quad \left| \begin{array}{l} df_k \leftarrow 0 \\ \text{for } m \in 1..2 \\ \quad \left| \begin{array}{l} \text{if } k = 1 \\ \quad \left| \begin{array}{l} df_k \leftarrow df_k + 1.0 \ \text{ if } X_m \le X_{beam_k} \\ df_k \leftarrow df_k + \dfrac{X_{beam_{k+1}} - X_m}{Spacing_{beam}} \ \text{ if } X_{beam_k} < X_m \le X_{beam_{k+1}} \end{array} \right. \\ \text{if } 2 \le k < \#Beams \\ \quad \left| \begin{array}{l} df_k \leftarrow df_k + \dfrac{X_m - X_{beam_{k-1}}}{Spacing_{beam}} \ \text{ if } X_{beam_{k-1}} \le X_m < X_{beam_k} \\ df_k \leftarrow df_k + \dfrac{X_{beam_{k+1}} - X_m}{Spacing_{beam}} \ \text{ if } X_{beam_k} \le X_m < X_{beam_{k+1}} \end{array} \right. \\ \text{if } k = \#Beams \\ \quad \left| \begin{array}{l} df_k \leftarrow df_k + \dfrac{X_m - X_{beam_{k-1}}}{Spacing_{beam}} \ \text{ if } X_{beam_{k-1}} \le X_m < X_{beam_k} \\ df_k \leftarrow df_k + 1.0 \ \text{ if } X_m \ge X_{beam_k} \end{array} \right. \end{array} \right. \end{array} \right. \\ df \end{array} \right.$$

Function below computes the fatigue load on each beam for a given live load increment i.

$$fP_{Fatigue}(i) := \left| \begin{array}{l} DF \leftarrow fDF_{wheel.load.fat}(i) \\ \text{for } k \in 1..\#Beams \\ \quad p_k \leftarrow 1.0 \cdot P_{liveload.fat} \cdot \dfrac{DF_k}{2} \\ p \end{array} \right.$$

Matrix of Fatigue Live Loads at each girder

$$mP_{Fat.LL.beam} := \left| \begin{array}{l} p_{service} \leftarrow fP_{Fatigue}(1)^T \\ \text{for } i \in 2..\#Increments_{liveload} \\ \quad p_{service} \leftarrow stack\left(p_{service}, fP_{Fatigue}(i)^T\right) \\ p_{service} \end{array} \right.$$

Centrifugal Forces (CE) [LRFD 3.6.3]

The centrifugal force acts in the transverse (X) direction.

$$f_{constant} := \frac{4}{3}$$ *Factor per LRFD 3.6.3*

$$C_{factor} := if\left(Radius = 0 \cdot ft, 0, \frac{f_{constant} \cdot V_{design.speed}^{2}}{g \cdot Radius}\right) = 0$$ *Centrifugal factor*

$$P_{CE} := C_{factor} \cdot (8kip + 32kip + 32kip) \cdot \#Lanes \cdot fMP_{factor}(\#Lanes) = 0 \cdot kip$$ *Total Centrifugal force*

$$P_{CE.z} := P_{CE} \cdot DF_{CE} \cdot \sin(Skew) = 0 \cdot kip$$ *Centrifugal force perpendicular (z-direction) to the cap length*

$$P_{CE.x} := P_{CE} \cdot DF_{CE} \cdot \cos(Skew) = 0 \cdot kip$$ *Centrifugal force parallel (x-direction) to the cap length*

Lateral loads on the superstructure create moments in the substructure due to moment arms. Moments caused by longitudinal forces along the bridge (perpendicular to the cap for small skews) are resolved into beam vertical reactions that are relatively small for long spans. Additionally, the beam reactions from ahead span and back span on intermediate bent caps are in opposite directions and may cancel out for continuouse loading resulting in a net torsional effect on the cap. Thus, vertical reactions due to moments caused by lateral force components that are perpendicular to the cap are ignored for cap design.

$$arm_{CE} := 6 \cdot ft + t_{slab} + t_{haunch} + h_{beam} = 8.58\,ft$$ *Moment Arm*

$$M_{CE.x} := P_{CE.x} \cdot arm_{CE} = 0 \cdot kip \cdot ft$$ *Moment parallel to cap*

The following function computes beam vertical reactions due to moments caused by lateral force components that are *parallel to the cap length (x-direction), assuming a rigid deck.*

$$fP_{beam}(M) := \left| \begin{array}{l} \text{for } k \in 1..\#Beams \\ \quad d_k \leftarrow X_{beam_k} - 0.5\left(X_{beam_1} + X_{beam_{\#Beams}}\right) \\ \text{for } k \in 1..\#Beams \\ \quad P_{beam_k} \leftarrow \dfrac{M}{\sum d^2} \cdot d_k \\ P_{beam} \end{array} \right.$$

$P_{CE.beam} := fP_{beam}(M_{CE.x})$

$P_{CE.beam}^{T} = (0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0) \cdot kip$

Beam reactions due to moment of CE force component that is parallel to the cap length (x-direction)

Braking Force (BR) [LRFD 3.6.4]

Force per Lane:

$P_{BR.truck} := .25 \cdot (32kip + 32kip + 8kip) = 18 \cdot kip$

$P_{BR.truck.and.lane} := .05 \cdot \left[(32kip + 32kip + 8kip) + w_{lane} \cdot L_{bridge.BR}\right] = 9.55 \cdot kip$

$P_{BR.tandem} := .25(25kip + 25kip) = 12.5 \cdot kip$

$P_{BR.tandem.and.lane} := .05 \cdot \left[(25kip + 25kip) + 2w_{lane} \cdot L_{span}\right] = 4.88 \cdot kip$

Total Braking force

$P_{BR} := \max\left(P_{BR.truck}, P_{BR.truck.and.lane}, P_{BR.tandem}, P_{BR.tandem.and.lane}\right) \cdot \#Lanes \cdot fMP_{factor}(\#Lanes) = 45.9 \cdot kip$

The braking force is transferred to the pier by the bearing pads. The braking forces need to be resolved along the direction of the skew for design of the cap.

$P_{BR.z} := P_{BR} \cdot DF_{BR} \cdot \cos(Skew) = 45.9 \cdot kip$

Total Braking force perpendicular (z-direction) to the cap

$P_{BR.x} := P_{BR} \cdot DF_{BR} \cdot \sin(Skew) = 0 \cdot kip$

Total Braking force parallel (x-direction) to the cap

$arm_{BR} := 6 \cdot ft + t_{slab} + t_{haunch} + h_{beam} = 8.58\,ft$

Moment Arm

$M_{BR.x} := P_{BR.x} \cdot arm_{BR} = 0 \cdot kip \cdot ft$

Moment parallel to cap

$P_{BR.beam} := fP_{beam}(M_{BR.x})$

$P_{BR.beam}^{T} = (0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0 \;\; 0) \cdot kip$

Beam reactions due to moment of CE force component that is parallel to the cap

Wind Load on Structure (WS) LRFD 3.8

Structure must be checked for aeroelastic stability per LRFD 3.8.3:

$$CheckAeroelasticStability := if\left[max\left[\frac{L_{long}}{W_{trans.bridge}}, \frac{L_{long}}{(h_{beam} + t_{haunch} + t_{slab})}\right] < 30, "Okay", "Re\text{-}analyze"\right]$$

$$CheckAeroelasticStability = "Okay"$$

$Depth_{super} = 5.24\ ft$ *Depth of superstructure*

$Area_{wind} := L_{span} \cdot Depth_{super} = 194.77\ ft^2$ *Area of the superstructure exposed to wind*

Design 3-second gust wind speed for different load combinations [LRFD Table 3.8.1.1.2-1]

$$V := \begin{pmatrix} V_{max} \\ 80mph \\ 70mph \\ 0.75V_{max} \end{pmatrix} = \begin{pmatrix} 130 \\ 80 \\ 70 \\ 97.5 \end{pmatrix} \cdot mph \qquad \begin{pmatrix} "Strength\ III" \\ "Strength\ V" \\ "Service\ I" \\ "Service\ IV" \end{pmatrix}$$

$h := Elev_{LM} - Elev_{ground} = 8.04\ ft$ *Height to bottom of girder*

$z := h + .5 \cdot (Depth_{super}) = 10.66\ ft$ *Height to centroid of area*

assumes wind above Flood Plain

$$K_z := \frac{\left(2.5\ln\left(\frac{max\left(\frac{z}{ft}, 33\right)}{0.0984}\right) + 7.35\right)^2}{478.4} = 1$$

Velocity Pressure Exposure Coefficient - Strength III and Service IV

$$K_z := \begin{pmatrix} K_z \\ 1.0 \\ 1.0 \\ K_z \end{pmatrix} = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \end{pmatrix} \qquad \begin{pmatrix} "Strength\ III" \\ "Strength\ V" \\ "Service\ I" \\ "Service\ IV" \end{pmatrix}$$

$z_{sub} := z = 10.66\ ft$ *Height for substructure Kz*

$$K_{z.sub} := \frac{\left(2.5\ln\left(\frac{max\left(\frac{z_{sub}}{ft}, 33\right)}{0.0984}\right) + 7.35\right)^2}{478.4} = 1$$

Velocity Pressure Exposure Coefficient for substructure - Strength III and Service IV

$$K_{z.sub} := \begin{pmatrix} K_{z.sub} \\ 1.0 \\ 1.0 \\ K_{z.sub} \end{pmatrix} = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \end{pmatrix} \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$$

$G := 0.85$ — *Gust Effect factor*

$C_{p.sup} := 1.3$ — *Pressure Coefficient, superstructure*

$C_{p.sub} := 1.6$ — *Pressure Coefficient, substructure*

WS Loads from Superstructure:

$w := 1 .. 4$

$$P_{z.sup_w} := 0.00256 \cdot K_{z_w} \cdot \left(\frac{V_w}{mph}\right)^2 \cdot G \cdot C_{p.sup} \cdot psf$$

Wind pressure

$$P_{z.sup} = \begin{pmatrix} 47.88 \\ 18.1 \\ 13.86 \\ 26.93 \end{pmatrix} \cdot psf \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$$

$$WS_{sup_w} := P_{z.sup_w} \cdot Depth_{super}$$

Wind load

$$WS_{sup} = \begin{pmatrix} 0.25 \\ 0.09 \\ 0.07 \\ 0.14 \end{pmatrix} \cdot klf \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$$

The wind pressure on the superstructure consists of transverse (x-direction) and longitudinal (z-direction) components.

$$Wind_{skew} := \begin{pmatrix} 0 \\ 15 \\ 30 \\ 45 \\ 60 \end{pmatrix} \qquad Wind_{Factor} := \begin{pmatrix} 1.0 & 0.0 \\ 0.88 & 0.12 \\ 0.82 & 0.24 \\ 0.66 & 0.32 \\ 0.34 & 0.38 \end{pmatrix}$$

For prestressed beam bridges, the following wind pressures factors are given to account for the angle of attack [SDG Table 2.4.1-3]

Conservatively apply the max transverse and longitudinal components concurrently.

$$WS_{sup.trans} := WS_{sup} \cdot \max\left(Wind_{Factor}^{\langle 1 \rangle}\right) = \begin{pmatrix} 0.25 \\ 0.09 \\ 0.07 \\ 0.14 \end{pmatrix} \cdot klf$$

Transverse wind load on superstructure

$$WS_{sup.long} := WS_{sup} \cdot \max\left(Wind_{Factor}^{\langle 2 \rangle}\right) = \begin{pmatrix} 0.1 \\ 0.04 \\ 0.03 \\ 0.05 \end{pmatrix} \cdot klf$$

Longitudinal wind load on superstructure

$$P_{WS.sup.trans} := WS_{sup.trans} \cdot L_{span} = \begin{pmatrix} 9.32 \\ 3.53 \\ 2.7 \\ 5.25 \end{pmatrix} \cdot kip$$

Total transverse superstructure WS load at int. cap

$$P_{WS.sup.long} := WS_{sup.long} \cdot L_{span} = \begin{pmatrix} 3.54 \\ 1.34 \\ 1.03 \\ 1.99 \end{pmatrix} \cdot kip$$

Total longitunidal superstructure WS load at int. cap

$$P_{WS.sup.x} := P_{WS.sup.trans} \cdot \cos(Skew) + P_{WS.sup.long} \cdot \sin(Skew) = \begin{pmatrix} 9.32 \\ 3.53 \\ 2.7 \\ 5.25 \end{pmatrix} \cdot kip$$

$$P_{WS.sup.z} := P_{WS.sup.long} \cdot \cos(Skew) + P_{WS.sup.trans} \cdot \sin(Skew) = \begin{pmatrix} 3.54 \\ 1.34 \\ 1.03 \\ 1.99 \end{pmatrix} \cdot kip$$

Vertical wind load

$$P_{vert} := \begin{pmatrix} 20 \\ 0 \\ 0 \\ 10 \end{pmatrix} \cdot psf \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$$

$$P_{WS.vert} := P_{vert} \cdot L_{span} \cdot W_{trans.bridge} = \begin{pmatrix} 45.44 \\ 0 \\ 0 \\ 22.72 \end{pmatrix} \cdot kip$$

Total vertical wind load at int. cap

$$M_{WS.sup.z} := P_{WS.sup.x} \cdot \frac{Depth_{super}}{2} + P_{WS.vert} \cdot \frac{W_{trans.bridge}}{4} \cdot \cos(Skew)$$

Moment of WS force component that is parallel to the cap

$$M_{WS.sup.z} = \begin{pmatrix} 718.7 \\ 9.24 \\ 7.07 \\ 360.88 \end{pmatrix} ft \cdot kip$$

$P_{WS.beam_w} := fP_{beam}\left(M_{WS.sup.z_w}\right)$ *Beam reactions due to moment of WS force component that is parallel to the cap*

$P_{WS.beam_1}{}^T = \begin{pmatrix} -7.23 & -5.42 & -3.61 & -1.81 & -1.59\times 10^{-15} & 1.81 & 3.61 & 5.42 & 7.23 \end{pmatrix}\cdot kip$

$P_{WS.beam_2}{}^T = \begin{pmatrix} -0.09 & -0.07 & -0.05 & -0.02 & 0 & 0.02 & 0.05 & 0.07 & 0.09 \end{pmatrix}\cdot kip$

$P_{WS.beam_3}{}^T = \begin{pmatrix} -0.07 & -0.05 & -0.04 & -0.02 & 0 & 0.02 & 0.04 & 0.05 & 0.07 \end{pmatrix}\cdot kip$

$P_{WS.beam_4}{}^T = \begin{pmatrix} -3.63 & -2.72 & -1.81 & -0.91 & 0 & 0.91 & 1.81 & 2.72 & 3.63 \end{pmatrix}\cdot kip$

Loads on Substructure:

$P_{z.sub_w} := 0.00256\, K_{z.sub_w}\cdot\left(\frac{V_w}{mph}\right)^2\cdot G\cdot C_{p.sub}\cdot psf$ *Wind pressure on substructure*

$P_{z.sub} = \begin{pmatrix} 58.92 \\ 22.28 \\ 17.06 \\ 33.14 \end{pmatrix}\cdot psf \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$

Conservatively apply the max transverse and longitudinal components concurrently.

$p_{z.sub.trans} := P_{z.sub}\cdot max\left(Wind_{Factor}{}^{\langle 1\rangle}\right) = \begin{pmatrix} 58.92 \\ 22.28 \\ 17.06 \\ 33.14 \end{pmatrix}\cdot psf$ *Transverse wind presure on substructure*

$p_{z.sub.long} := P_{z.sub}\cdot max\left(Wind_{Factor}{}^{\langle 2\rangle}\right) = \begin{pmatrix} 22.39 \\ 8.47 \\ 6.48 \\ 12.59 \end{pmatrix}\cdot psf$ *Longitudinal wind presure on substructure*

$p_{z.sub.x} := p_{z.sub.trans}\cdot cos(Skew) + p_{z.sub.long}\cdot sin(Skew) = \begin{pmatrix} 58.92 \\ 22.28 \\ 17.06 \\ 33.14 \end{pmatrix}\cdot psf$ *Max wind presure parallel to cap*

$p_{z.sub.z} := p_{z.sub.long}\cdot cos(Skew) + p_{z.sub.trans}\cdot sin(Skew) = \begin{pmatrix} 22.39 \\ 8.47 \\ 6.48 \\ 12.59 \end{pmatrix}\cdot psf$ *Max wind presure perpendicular to cap*

$$P_{WS.sub.x.cap} := p_{z.sub.x} \cdot H_{cap} \cdot W_{cap} = \begin{pmatrix} 0.71 \\ 0.27 \\ 0.2 \\ 0.4 \end{pmatrix} \cdot kip$$

WS force on cap: parallel (x-direction) to the cap

$$P_{WS.sub.z.cap} := p_{z.sub.z} \cdot H_{cap} \cdot L_{cap} = \begin{pmatrix} 4.02 \\ 1.52 \\ 1.16 \\ 2.26 \end{pmatrix} \cdot kip$$

WS force on cap: perpendicular (z-direction) to the cap

$$P_{WS.sub.x.col} := p_{z.sub.x} \cdot W_{col} \cdot \left(Elev_{LM} - H_{cap} - Elev_{ground}\right) = \begin{pmatrix} 0.45 \\ 0.17 \\ 0.13 \\ 0.25 \end{pmatrix} \cdot kip$$

WS force on one column: parallel (x-direction) to the cap

$$P_{WS.sub.z.col} := p_{z.sub.z} \cdot W_{col} \cdot \left(Elev_{LM} - H_{cap} - Elev_{ground}\right) = \begin{pmatrix} 0.17 \\ 0.06 \\ 0.05 \\ 0.1 \end{pmatrix} \cdot kip$$

WS force on one column: perpendicular (z-direction) to the cap

Total substructure WS force parallel (x-direction) to the cap (Half of the WS on columns is to be applied at cap)

$$P_{WS.sub.x} := P_{WS.sub.x.cap} + 0.5 P_{WS.sub.x.col} \cdot \#Cols = \begin{pmatrix} 2.04 \\ 0.77 \\ 0.59 \\ 1.15 \end{pmatrix} \cdot kip$$

Total substructure WS force perpendicular (z-direction) to the cap (Half of the WS on columns is to be applied at cap)

$$P_{WS.sub.z} := P_{WS.sub.z.cap} + 0.5 P_{WS.sub.z.col} \cdot \#Cols = \begin{pmatrix} 4.53 \\ 1.71 \\ 1.31 \\ 2.55 \end{pmatrix} \cdot kip$$

Summary of WS Loads

Total WS force parallel (x-direction) to the cap

$$P_{WS.x} := P_{WS.sup.x} + P_{WS.sub.x} = \begin{pmatrix} 11.37 \\ 4.3 \\ 3.29 \\ 6.39 \end{pmatrix} \cdot kip \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$$

Total WS force perpendicular (z-direction) to the cap

$$P_{WS.z} := P_{WS.sup.z} + P_{WS.sub.z} = \begin{pmatrix} 8.07 \\ 3.05 \\ 2.34 \\ 4.54 \end{pmatrix} \cdot kip \qquad \begin{pmatrix} \text{"Strength III"} \\ \text{"Strength V"} \\ \text{"Service I"} \\ \text{"Service IV"} \end{pmatrix}$$

Wind on Live Load (WL) [LRFD 3.8.1.3]

The LRFD specifies that wind load should be applied to vehicles on the bridge..

$$Skew_{wind} := \begin{pmatrix} 0 \\ 15 \\ 30 \\ 45 \\ 60 \end{pmatrix} \qquad Wind_{LRFD} := \begin{pmatrix} .100 & 0 \\ .088 & .012 \\ .082 & .024 \\ .066 & .032 \\ .034 & .038 \end{pmatrix} \cdot \frac{kip}{ft}$$

Conservatively apply the max transverse and longitudinal components concurrently.

$WL_{trans} := max\left(Wind_{LRFD}^{\langle 1 \rangle}\right) = 0.1 \cdot klf$ — *Transverse WL*

$WL_{long} := max\left(Wind_{LRFD}^{\langle 2 \rangle}\right) = 0.04 \cdot klf$ — *Longitudinal WL*

$P_{WL.trans} := WL_{trans} \cdot L_{span} = 3.72 \cdot kip$ — *Total transverse WL load at int. cap*

$P_{WL.long} := WL_{long} \cdot L_{span} = 1.41 \cdot kip$ — *Total longitunidal WL load at int. cap*

$P_{WL.x} := P_{WL.trans} \cdot cos(Skew) + P_{WL.long} \cdot sin(Skew) = 3.72 \cdot kip$ — *Total WL force parallel (x-direction) to the cap*

$P_{WL.z} := P_{WL.long} \cdot cos(Skew) + P_{WL.trans} \cdot sin(Skew) = 1.41 \cdot kip$ — *Total WL force perpendicular (z-direction) to the cap*

$arm_{WL} := 6 \cdot ft + t_{slab} + t_{haunch} + h_{beam} = 8.58\ ft$ — *Moment Arm*

$M_{WL.z} := P_{WL.x} \cdot arm_{WL} = 31.89 \cdot kip \cdot ft$ — *Moment f WL force component that is parallel to the cap*

$P_{WL.beam} := fP_{beam}\left(M_{WL.z}\right)$ — *Beam reactions due to moment of WL force component that is parallel to the cap*

$$P_{WL.beam}^{T} = \begin{pmatrix} -0.32 & -0.24 & -0.16 & -0.08 & 0 & 0.08 & 0.16 & 0.24 & 0.32 \end{pmatrix} \cdot kip$$

Water Load on Structure (WA) [LRFD 3.7]

$P_{WA.x.100} = 0 \cdot kip$ — *Total WA force on cap, 100 year event: parallel (x-direction) to the cap*

$P_{WA.z.100} = 0 \cdot kip$ — *Total WA force on cap, 100 year event: perpendicular (z-direction) to the cap*

$P_{WA.x.500} = 0 \cdot kip$ — *Total WA force on cap, 500 year event: parallel (x-direction) to the cap*

$P_{WA.z.500} = 0 \cdot kip$ — *Total WA force on cap, 500 year event: perpendicular (z-direction) to the cap*

Force Effect due to Uniform Temperature (TU) [LRFD 3.12]

$P_{TU} = 0 \cdot kip$ *Total TU force on cap*

$P_{TU.x} := P_{TU} \cdot \sin(Skew) = 0 \cdot kip$ *Total TU force on cap: parallel (x-direction) to the cap*

$P_{TU.z} := P_{TU} \cdot \cos(Skew) = 0 \cdot kip$ *Total TU force on cap: perpendicular (z-direction) to the cap*

Loads: DC&DW, LL, CE, BR, WS, WL, WA, TU

Strength, Service and Extreme Event Limit States

Load factors per LRFD Table 3.4.1-1 and SDG Table 2.4.1-1:

$\gamma s :=$

DC	DW	LL	CE	BR	WA	WS	WL	TU	
1.25	1.5	1.75	1.75	1.75	1.0	0.0	0.0	1.2	"Strength I max vertical"
1.25	1.5	0.0	0.0	0.0	1.0	1.0	0.0	1.2	"Strength III max vertical"
1.25	1.5	1.35	1.35	1.35	1.0	1.0	1.0	1.2	"Strength V max vertical"
1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.2	"Service I"
1.0	1.0	0.8	0.8	0.8	1.0	0.0	0.0	1.2	"Service III"
1.0	1.0	0.2	0	0	0	0	0	0	"Sustained load: DL+0.2LL"
0.9	1.5	1.75	1.75	1.75	1.0	0.0	0.0	1.2	"Strength I min vertical"
0.9	1.5	0.0	0.0	0.0	1.0	1.0	0.0	1.2	"Strength III min vertical"
0.9	1.5	1.35	1.35	1.35	1.0	1.0	1.0	1.2	"Strength V min vertical"
1.0	1.0	1.5	0	0	0	0	0	0	"DL+Fatigue I"

$\#LimitStates := rows(\gamma s) = 10$

$LS := 1 .. \#LimitStates$

Factored total load: parallel to cap

$$
\begin{aligned}
P_{cap.x} := \ & \text{for } LS \in 1 .. \#LimitStates \\
& \quad P_{WS} \leftarrow 0 \\
& \quad P_{WS} \leftarrow P_{WS.x_1} \ \text{ if } LS = 2 \vee LS = 8 \\
& \quad P_{WS} \leftarrow P_{WS.x_2} \ \text{ if } LS = 3 \vee LS = 9 \\
& \quad P_{WS} \leftarrow P_{WS.x_3} \ \text{ if } LS = 4 \\
& \quad P_{LS} \leftarrow \gamma s_{LS,4} \cdot P_{CE.x} + \gamma s_{LS,5} \cdot P_{BR.x} + \gamma s_{LS,6} \cdot P_{WA.x.100} + \gamma s_{LS,7} \cdot P_{WS} \cdots \\
& \quad\quad + \gamma s_{LS,8} \cdot P_{WL.x} + \gamma s_{LS,9} \cdot P_{TU.x} \\
& \quad P_{LS} \leftarrow Round(P_{LS}, 0.01kip) \\
& P
\end{aligned}
$$

$P_{cap.x}{}^{T} = (0 \;\; 11.37 \;\; 8.02 \;\; 7.01 \;\; 0 \;\; 0 \;\; 0 \;\; 11.37 \;\; 8.02 \;\; 0)\cdot kip$

Factored total load: perpendicular to cap

$$
\begin{aligned}
P_{cap.z} := \; & \text{for } LS \in 1 .. \#LimitStates \\
& \quad P_{WS} \leftarrow 0 \\
& \quad P_{WS} \leftarrow P_{WS.z_1} \;\; \text{if } LS = 2 \vee LS = 8 \\
& \quad P_{WS} \leftarrow P_{WS.z_2} \;\; \text{if } LS = 3 \vee LS = 9 \\
& \quad P_{WS} \leftarrow P_{WS.z_3} \;\; \text{if } LS = 4 \\
& \quad P_{LS} \leftarrow \gamma s_{LS,4}\cdot P_{CE.z} + \gamma s_{LS,5}\cdot P_{BR.z} + \gamma s_{LS,6}\cdot P_{WA.z.100} + \gamma s_{LS,7}\cdot P_{WS} \cdots \\
& \qquad + \gamma s_{LS,8}\cdot P_{WL.z} + \gamma s_{LS,9}\cdot P_{TU.z} \\
& \quad P_{LS} \leftarrow Round(P_{LS}, 0.01kip) \\
& P
\end{aligned}
$$

$P_{cap.z}{}^{T} = (80.32 \;\; 8.07 \;\; 66.43 \;\; 49.65 \;\; 36.72 \;\; 0 \;\; 80.32 \;\; 8.07 \;\; 66.43 \;\; 0)\cdot kip$

Factored cap uniform self weight

$w_{cap.y_{LS}} := \gamma s_{LS,1}\cdot w_{DC.cap}$

$w_{cap.y} := Round(w_{cap.y}, 0.001klf)$

$w_{cap.y}{}^{T} = (2.32 \;\; 2.32 \;\; 2.32 \;\; 1.85 \;\; 1.85 \;\; 1.85 \;\; 1.67 \;\; 1.67 \;\; 1.67 \;\; 1.85)\cdot klf$

Function for Limit State Vertical Superstructure Loads per Beam Line (Column)

$$
fP_{LimitState}(LS) := \left|
\begin{array}{l}
\gamma \leftarrow \left(\gamma s^T\right)^{\langle LS \rangle} \\
\#rows \leftarrow 1 \quad \text{if } LS = 2 \vee LS = 8 \\
\#rows \leftarrow rows\left(mP_{Fat.LL.beam}\right) \quad \text{if } LS = 10 \\
\#rows \leftarrow rows\left(mP_{LL.beam}\right) \quad \text{otherwise} \\
mP_{LL} \leftarrow mP_{Fat.LL.beam} \quad \text{if } LS = 10 \\
mP_{LL} \leftarrow mP_{LL.beam} \quad \text{otherwise} \\
P_{WS.bm} \leftarrow P_{WS.beam_1} \quad \text{if } LS = 2 \vee LS = 8 \\
P_{WS.bm} \leftarrow P_{WS.beam_2} \quad \text{if } LS = 3 \vee LS = 9 \\
P_{WS.bm} \leftarrow P_{WS.beam_3} \quad \text{if } LS = 4 \\
\text{for } j \in 1..\#Beams \quad \text{otherwise} \\
\quad P_{WS.bm_j} \leftarrow 0 \\
\text{for } i \in 1..\#rows \\
\quad \left| \begin{array}{l}
\text{for } j \in 1..\#Beams \\
\quad \left| \begin{array}{l}
P_{beam_{i,j}} \leftarrow \gamma_1 \cdot \left(P_{DC.beam_j}\right) + \gamma_2 \cdot \left(P_{DW.beam_j}\right) + \gamma_3 \cdot \left(mP_{LL_{i,j}}\right) + \gamma_4 \cdot \left(P_{CE.beam_j}\right) \cdots \\
\qquad + \gamma_5 \cdot \left(P_{BR.beam_j}\right) + \gamma_7 \cdot \left(P_{WS.bm_j}\right) + \gamma_8 \cdot P_{WL.beam_j} \\
P_{beam_{i,j}} \leftarrow Round\left(P_{beam_{i,j}}, 0.01 \cdot kip\right)
\end{array} \right. \\
\left(P_{beam}{}^T\right)^{\langle i \rangle}
\end{array} \right. \\
P_{beam}
\end{array}
\right.
$$

$fP_{LimitState}(1) =$

	1	2	3	4	5	6	7	8
1	60.53	130.81	170.53	75.97	56.11	56.11	56.11	56.11
2	60.53	118.36	151.46	151.46	151.46	56.11	56.11	56.11
3	60.53	86.7	137.16	159.48	144.19	137.16	116.06	56.11
4	60.53	122.8	170.53	83.98	56.11	56.11	56.11	56.11
5	60.53	111.8	151.46	158.03	151.46	56.11	56.11	56.11
6	60.53	85.16	137.16	159.48	142.65	138.7	117.6	56.11
7	60.53	114.79	170.53	92	56.11	56.11	56.11	56.11
8	60.53	105.23	151.46	164.59	151.46	56.11	56.11	...

$\cdot kip$

Matrix for all Limit States Vertical Superstructure Loads per Beam Line (Column)

$$
P_{cap.beam} := \left|
\begin{array}{l}
\text{for } i \in 1..rows(\gamma s) \\
\quad LoadCase_i \leftarrow fP_{LimitState}(i) \\
LoadCase
\end{array}
\right.
$$

$$P_{cap.beam} = \begin{pmatrix} \{180,9\} \\ \{1,9\} \\ \{180,9\} \\ \{180,9\} \\ \{180,9\} \\ \{180,9\} \\ \{180,9\} \\ \{1,9\} \\ \{180,9\} \\ \{60,9\} \end{pmatrix} \cdot kip \qquad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Service I"} \\ \text{"Service III"} \\ \text{"Sustained load: DL+0.2LL"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \\ \text{"DL+Fatigue I"} \end{pmatrix}$$

Strength, Service and Extreme Event Limit States

Coordinates for Bridge Superstructure and Substructure Section

XY coordinates for superstructure cross section view
(origin at left coping of deck, X-direction parallel to bridge transverse direction)

$t_{deck} := t_{slab} = 8.5 \cdot in$

$h_{beam} = 21.3 \cdot in$

$X_{deck_1} := 0 \cdot in$

$Y_{deck_1} := h_{beam} + t_{haunch}$

$X_{deck_2} := d_{coping.to.curb} \cdot 2 + W_{curb.to.curb}$

$Y_{deck_2} := Y_{deck_1}$

$X_{deck_3} := X_{deck_2}$

$Y_{deck_3} := Y_{deck_1} + t_{deck} + h_{barrier}$

$X_{deck_4} := X_{deck_3} - d_{coping.to.curb}$

$Y_{deck_4} := Y_{deck_3}$

$X_{deck_5} := X_{deck_4}$

$Y_{deck_5} := Y_{deck_4} - h_{barrier}$

$X_{deck_6} := X_{deck_4} - W_{curb.to.curb}$

$Y_{deck_6} := Y_{deck_5}$

$X_{deck_7} := X_{deck_6}$

$Y_{deck_7} := Y_{deck_5} + h_{barrier}$

$X_{deck_8} := X_{deck_1}$

$Y_{deck_8} := Y_{deck_7}$

$X_{deck_9} := X_{deck_1}$

$Y_{deck_9} := Y_{deck_1}$

$X_{beam}^T = (4.04 \quad 10.67 \quad 17.3 \quad 23.93 \quad 30.56 \quad 37.19 \quad 43.82 \quad 50.45 \quad 57.08)\ ft$

$k := 1 .. \#Beams$

$Y_{beam_k} := h_{beam}$

xy coordinates for substructure section view (origin at left fascia of cap, x-direction parallel to cap)

$$d_{X.to.x} := 0.5L_{cap} - \frac{0.5W_{trans.bridge}}{\cos(Skew)} = -0.61\ ft$$

$$x_{deck} := \frac{X_{deck}}{\cos(Skew)} + d_{X.to.x}$$

$$y_{deck} := Y_{deck}$$

$$x_{beam} := \frac{X_{beam}}{\cos(Skew)} + d_{X.to.x}$$

$$y_{beam} := Y_{beam}$$

$ip := 1 .. \#Cols$

$$x_{col_{ip}} := L_{cap.oh} + (ip - 1) \cdot Spacing_{col}$$

$$x_{col}^{T} = (4 \quad 14.38 \quad 24.76 \quad 35.14 \quad 45.52 \quad 55.9)\ ft$$

$$y_{col_{ip}} := -L_{col}$$

$$x_{cap} := \begin{pmatrix} 0 \cdot ft \\ L_{cap} \end{pmatrix}$$

$$y_{cap} := \begin{pmatrix} 0 \cdot ft \\ 0 \cdot ft \end{pmatrix}$$

Coordinates for Bridge Superstructure and Substructure Section

Write data out

Write Data out

$$DataOut_1 := \frac{x_{deck}}{ft} \qquad DataOut_2 := \frac{x_{beam}}{ft} \qquad DataOut_3 := \frac{x_{cap}}{ft} \qquad DataOut_4 := \frac{x_{col}}{ft}$$

$$DataOut_5 := \frac{y_{deck}}{ft} \qquad DataOut_6 := \frac{y_{beam}}{ft} \qquad DataOut_7 := \frac{y_{cap}}{ft} \qquad DataOut_8 := \frac{y_{col}}{ft}$$

$$DataOut_9 := \frac{H_{cap}}{ft} \qquad DataOut_{10} := \frac{W_{cap}}{ft} \qquad DataOut_{11} := \frac{W_{col}}{ft} \qquad DataOut_{12} := ColType$$

$$DataOut_{13} := \frac{H_{pedestal}}{in} \qquad DataOut_{14} := \frac{L_{pad}}{in}$$

$$DataOut_{15} := \frac{f_{c.cap}}{ksi} \qquad DataOut_{16} := \frac{f_{c.col}}{ksi}$$

$$DataOut_{17} := \frac{E_{cap}}{ksi} \qquad DataOut_{18} := \frac{E_{col}}{ksi} \qquad DataOut_{19} := \frac{\gamma_{conc}}{pcf}$$

$$DataOut_{20} := OutputDataFile$$

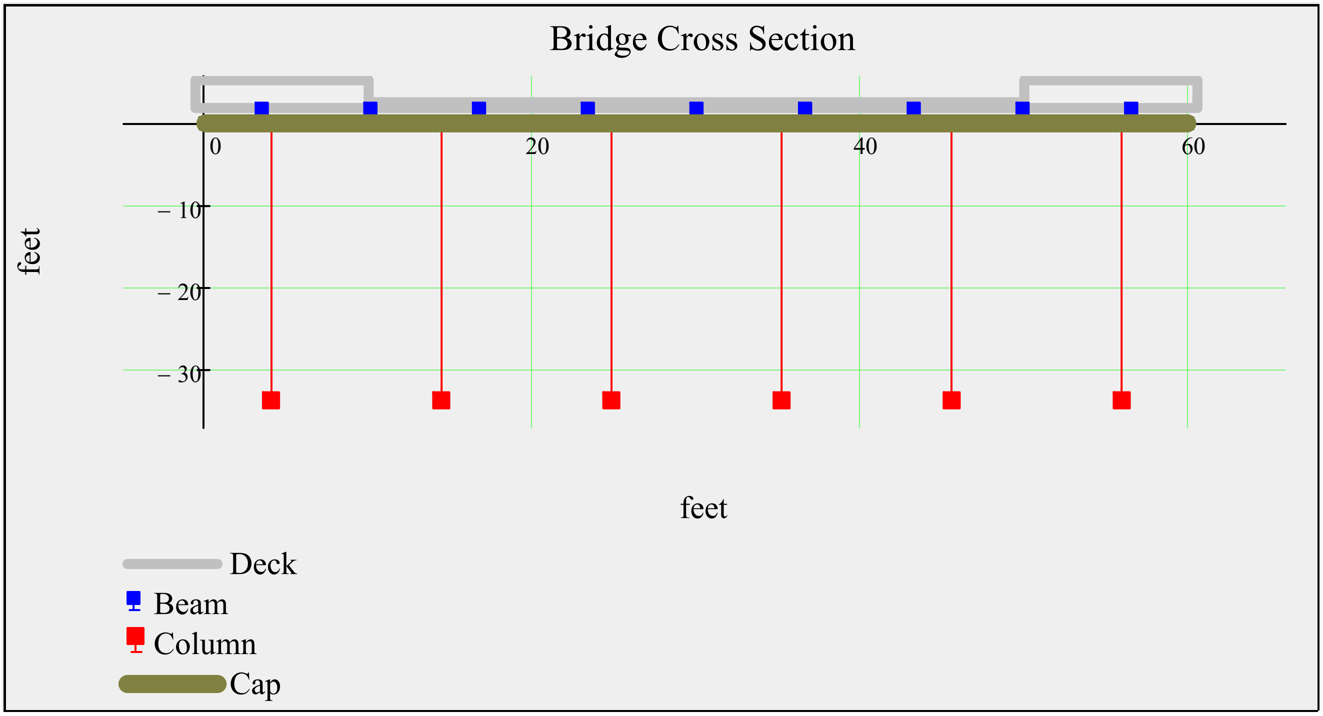
PRNPRECISION := 6

Data := WRITEPRN("data\BentGeometry.txt" , DataOut)

$$Data := WRITEPRN\left[\text{"data\textbackslash CapLoads.txt"}, \begin{pmatrix} \frac{P_{cap.beam}}{kip} \\ \frac{w_{cap.y}}{klf} \\ \frac{P_{cap.x}}{kip} \\ \frac{P_{cap.z}}{kip} \end{pmatrix}\right]$$

Write data out

Calculate Worksheet



Summary of unfactored cap loads

Summary of unfactored Vertical loads from Superstructure at each girder:

$P_{DC.beam}^{T} = (41.06 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 38.2 \quad 41.06)\cdot kip$

$P_{DW.beam}^{T} = (6.13 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 5.58 \quad 6.13)\cdot kip$

$mP_{LL.beam} =$

	1	2	3	4	5	6	7	8	9
1	0	42.69	65.38	11.35	0	0	0	0	0
2	0	35.57	54.48	54.48	54.48	0	0	0	0
3	0	17.48	46.31	59.07	50.33	46.31	34.25	0	0
4	0	38.11	65.38	15.93	0	0	0	0	0
5	0	31.82	54.48	58.24	54.48	0	0	0	...

$\cdot kip$

$$Max_{liveload.beam} = \begin{pmatrix} 1 & 2 & 3 & 4 & 5 & 6 & 7 & 8 & 9 \\ 0 & 42.7 & 65.4 & 73.6 & 73.9 & 73.6 & 65.4 & 42.7 & 0 \\ 0 & 1 & 1 & 16 & 37 & 45 & 51 & 60 & 0 \\ 0 & 1 & 1 & 2 & 2 & 2 & 1 & 1 & 0 \end{pmatrix} \quad \begin{pmatrix} \text{"Beam number"} \\ \text{"Max beam LL (kip)"} \\ \text{"Increment"} \\ \text{"number of lanes loaded"} \end{pmatrix}$$

$mP_{Fat.LL.beam} =$ (·kip)

	1	2	3	4	5	6	7	8	9
1	0	17.74	27.17	4.72	0	0	0	0	0
2	0	15.84	27.17	6.62	0	0	0	0	0
3	0	13.94	27.17	8.52	0	0	0	0	0
4	0	12.03	27.17	10.43	0	0	0	0	0
5	0	10.13	27.17	12.33	0	0	0	0	...

$P_{CE.beam}^{T} = (0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\cdot kip$

$P_{BR.beam}^{T} = (0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0\ \ 0)\cdot kip$

$P_{WS.beam_1}^{T} = \left(-7.23\ \ -5.42\ \ -3.61\ \ -1.81\ \ -1.59\times10^{-15}\ \ 1.81\ \ 3.61\ \ 5.42\ \ 7.23\right)\cdot kip$ *for Strength III*

$P_{WS.beam_2}^{T} = (-0.09\ \ -0.07\ \ -0.05\ \ -0.02\ \ 0\ \ 0.02\ \ 0.05\ \ 0.07\ \ 0.09)\cdot kip$ *for Strength V*

$P_{WS.beam_3}^{T} = (-0.07\ \ -0.05\ \ -0.04\ \ -0.02\ \ 0\ \ 0.02\ \ 0.04\ \ 0.05\ \ 0.07)\cdot kip$ *for Service I*

$P_{WS.beam_4}^{T} = (-3.63\ \ -2.72\ \ -1.81\ \ -0.91\ \ 0\ \ 0.91\ \ 1.81\ \ 2.72\ \ 3.63)\cdot kip$ *for Service IV*

$P_{WL.beam}^{T} = (-0.32\ \ -0.24\ \ -0.16\ \ -0.08\ \ 0\ \ 0.08\ \ 0.16\ \ 0.24\ \ 0.32)\cdot kip$

Self weight of cap and pedestals:

$w_{DC.cap} = 1.85\cdot klf$

Summary of unfactored lateral loads on cap:

Parallel to cap:

$P_{CE.x} = 0\cdot kip$

$P_{BR.x} = 0\cdot kip$

$P_{WS.x} = \begin{pmatrix} 11.37 \\ 4.3 \\ 3.29 \\ 6.39 \end{pmatrix}\cdot kip$

$P_{WL.x} = 3.72\cdot kip$

$P_{WA.x.100} = 0\cdot kip$

$P_{WA.x.500} = 0\cdot kip$

$P_{TU.x} = 0\cdot kip$

Perpendicular to cap:

$P_{CE.z} = 0\cdot kip$

$P_{BR.z} = 45.9\cdot kip$

$P_{WS.z} = \begin{pmatrix} 8.07 \\ 3.05 \\ 2.34 \\ 4.54 \end{pmatrix}\cdot kip$

for Strength III
for Strength V
for Service I
for Service IV

$P_{WL.z} = 1.41\cdot kip$

$P_{WA.z.100} = 0\cdot kip$

$P_{WA.z.500} = 0\cdot kip$

$P_{TU.z} = 0\cdot kip$

Summary of factored cap loads

$$P_{cap.beam} = \begin{pmatrix} \{180,9\} \\ \{1,9\} \\ \{180,9\} \\ \{180,9\} \\ \{180,9\} \\ \{180,9\} \\ \{180,9\} \\ \{1,9\} \\ \{180,9\} \\ \{60,9\} \end{pmatrix} \cdot kip \qquad w_{cap.y} = \begin{pmatrix} 2.32 \\ 2.32 \\ 2.32 \\ 1.85 \\ 1.85 \\ 1.85 \\ 1.67 \\ 1.67 \\ 1.67 \\ 1.85 \end{pmatrix} \cdot klf$$

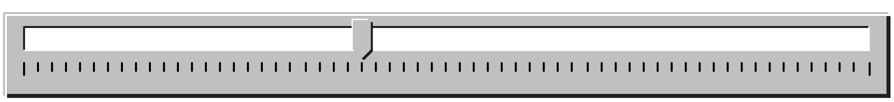
"Strength I max vertical"
"Strength III max vertical"
"Strength V max vertical"
"Service I"
"Service III"
"Sustained load: DL+0.2LL"
"Strength I min vertical"
"Strength III min vertical"
"Strength V min vertical"
"DL+Fatigue I"

$$P_{cap.x} = \begin{pmatrix} 0 \\ 11.37 \\ 8.02 \\ 7.01 \\ 0 \\ 0 \\ 0 \\ 11.37 \\ 8.02 \\ 0 \end{pmatrix} \cdot kip \qquad P_{cap.z} = \begin{pmatrix} 80.32 \\ 8.07 \\ 66.43 \\ 49.65 \\ 36.72 \\ 0 \\ 80.32 \\ 8.07 \\ 66.43 \\ 0 \end{pmatrix} \cdot kip$$

"Strength I max vertical"
"Strength III max vertical"
"Strength V max vertical"
"Service I"
"Service III"
"Sustained load: DL+0.2LL"
"Strength I min vertical"
"Strength III min vertical"
"Strength V min vertical"
"DL+Fatigue I"

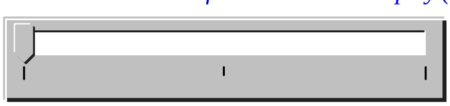
Graphical Display of Wheel Loads

Choose a specific location (increment) to display the wheel load locations



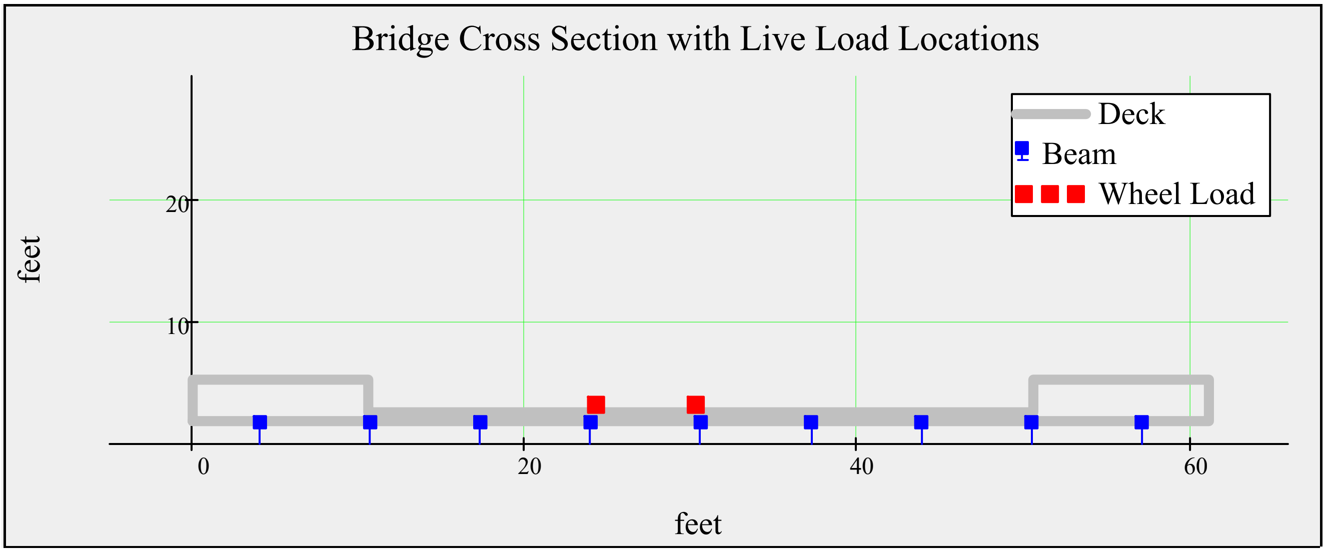
Increment = 24

Choose the number of lanes loaded to display (up to 3 loaded lanes)



#LanesLoaded = 1

Set Variables to Display LL



Beam forces for chosen increment and number of lanes loaded

$P_{LL.beam}^{T} = (0.0 \quad 0.0 \quad 0.0 \quad 59.5 \quad 59.9 \quad 0.0 \quad 0.0 \quad 0.0 \quad 0.0) \cdot kip$

Intermediate Bent-Cap Analysis & Design
Part 2. Frame Analysis

State Structures Design Office

Project =
Designed By =
Checked By =
Back Checked By =

Read In Geometry and Loads From Bent Cap Load Generator

DataFile = "HRB-NK.dat"

LEGEND:

input data is gray

results and warnings are yellow

TopCol := Fixed

Fixed or Pinned connection of columns to bent cap

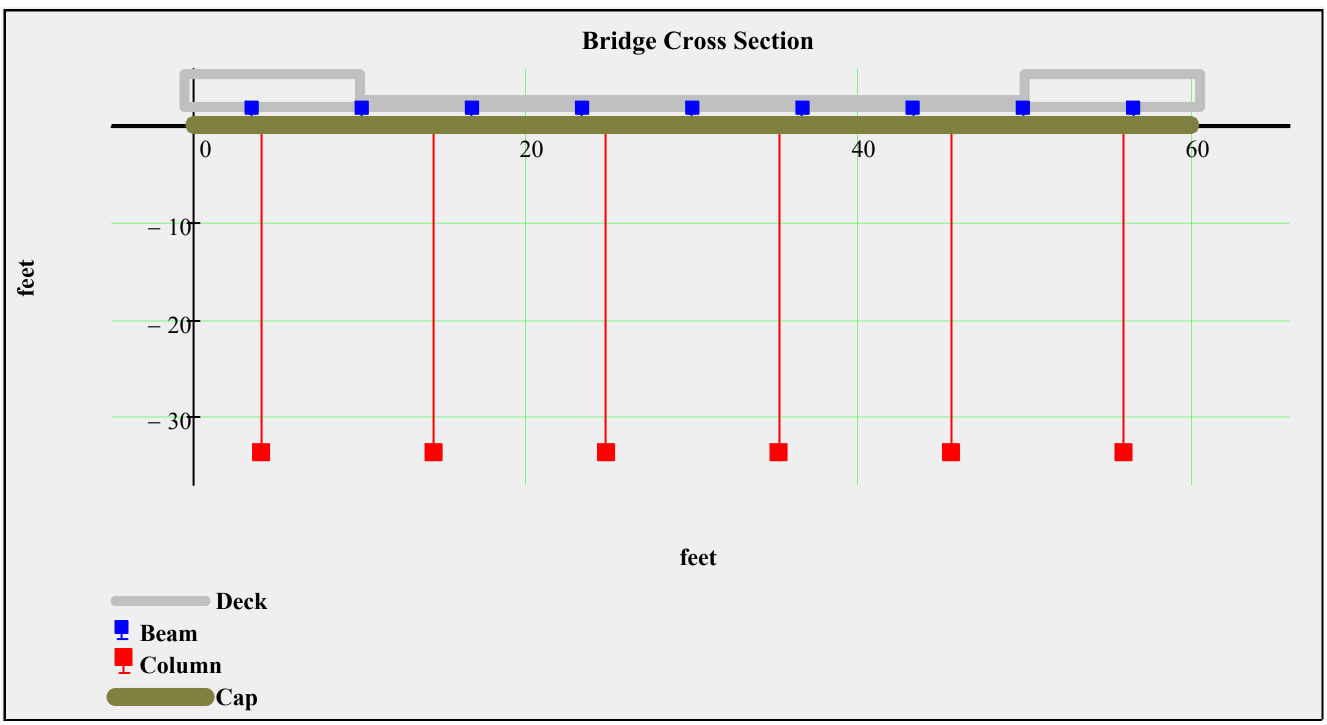
BeamLoad := Distributed

*Typically for cap design, the beam loads are treated as concentrated loads for simplicity and conservatism. When the center line of beam is close to face of support, the shear demand is overly conservative. Thus, a distributed option is given that treats the beam load as a line load over a width of (bearing pad width + 2 * pedestal height). When the concentrated loading case is selected, the program assumes a small positve number (3 in) for the distributed width.*

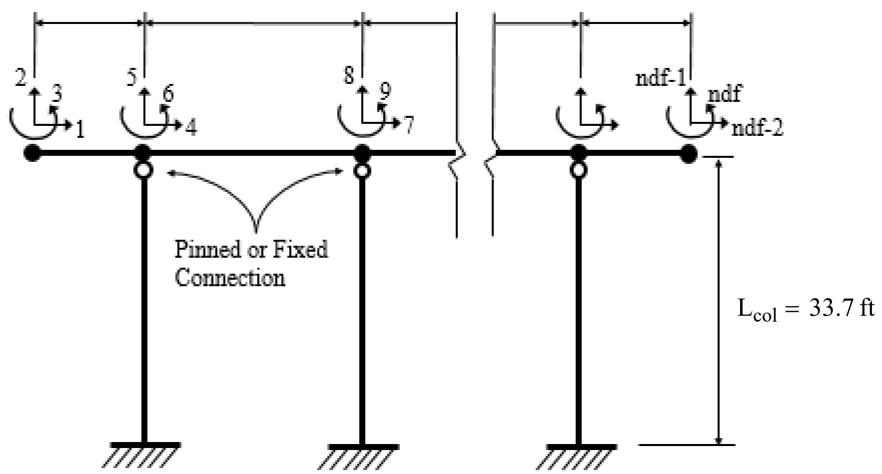
Distributed width of beam load

$$W_{load} := \begin{cases} (3in) & \text{if BeamLoad} = \text{"Concentrated"} \\ \max(3in, L_{pad} + 2 \cdot H_{pedestal}) & \text{otherwise} \end{cases}$$

$$W_{load} = 30 \cdot in$$



Model



Bent Cap Analysis Model

$\#Beams = 9$	*Number of beams*
$Spacing_{beam} = 6.63$ ft	*Spacing of beams (along length of cap)*
$\#Cols = 6$	*A minimum of two columns required*
$Spacing_{col} = 10.38$ ft	*Spacing of columns*
$L_{cap} = 59.9$ ft	*Length of cap*
$L_{cap.oh} = 4$ ft	*See Bent Cap Model (minimum length is half of column width)*
$f_{c.cap} = 5.5 \cdot ksi$	*Min.28-day compressive strength for cap*
$E_{cap} = 4428 \cdot ksi$	*Modulus of elasticity of cap*
$W_{cap} = 4$ ft	*Width of cap*
$H_{cap} = 3$ ft	*Height of cap*
$A_{cap} = 1728 \cdot in^2$	*Cross sectional area of cap beam*
$I_{cap} = 186624 \cdot in^4$	*Moment of inertia of cap*
$f_{c.col} = 6.0 \cdot ksi$	*Min.28-day compressive strength for columns*
$E_{col} = 4557 \cdot ksi$	*Modulus of elasticity of columns*
$W_{col} = 18 \cdot in$	*Width/Diameter of the column*
$ColType = 2$	*Column type: 1-round, 2-square*
$A_{col} = 324 \cdot in^2$	*Cross sectional area of column*
$I_{col} = 8748 \cdot in^4$	*Moment of inertia of column*

Develop Stiffness Matrix, Member Force Matrix and Solve for Cap Moment & Shear Forces

Define Member Stiffness Matrix

$nm := \#Cols + 1 = 7$ *Number of members*

$ndf := (nm + 1) \cdot 3 = 24$ *Number of degree of freedom*

$$
K(m) := \left|
\begin{array}{l}
L \leftarrow L_{cap.oh} \;\; \text{if } m = 1 \vee m = nm \\
L \leftarrow Spacing_{col} \;\; \text{otherwise} \\
\text{for } i \in 1..6 \\
\quad \text{for } j \in 1..6 \\
\quad\quad k_{i,j} \leftarrow 0 \\
\text{if } 1 \le m \le nm \\
\quad \left|
\begin{array}{l}
k_{1,1} \leftarrow \dfrac{E_{cap} \cdot A_{cap}}{L} \cdot \dfrac{in}{kip} \\
k_{1,4} \leftarrow -k_{1,1} \\
k_{2,2} \leftarrow \dfrac{12E_{cap} \cdot I_{cap}}{L^3} \cdot \dfrac{in}{kip} \\
k_{2,3} \leftarrow \dfrac{6E_{cap} \cdot I_{cap}}{L^2} \cdot \dfrac{1}{kip} \\
k_{2,5} \leftarrow -k_{2,2} \\
k_{2,6} \leftarrow k_{2,3} \\
k_{3,3} \leftarrow \dfrac{4E_{cap} \cdot I_{cap}}{L} \cdot \dfrac{1}{kip \cdot in} \\
k_{3,5} \leftarrow -k_{2,3} \\
k_{3,6} \leftarrow \dfrac{2E_{cap} \cdot I_{cap}}{L} \cdot \dfrac{1}{kip \cdot in} \\
k_{4,4} \leftarrow k_{1,1} \\
k_{5,5} \leftarrow k_{2,2} \\
k_{5,6} \leftarrow -k_{2,3} \\
k_{6,6} \leftarrow k_{3,3} \\
k \leftarrow k + k^T \\
\text{for } j \in 1..6 \\
\quad k_{j,j} \leftarrow \dfrac{k_{j,j}}{2}
\end{array}
\right. \\
k
\end{array}
\right.
$$

Define Structure Stiffness Matrix

$$
S := \left|
\begin{array}{l}
\text{for } i \in 1..ndf \\
\quad \text{for } j \in 1..ndf \\
\quad\quad S_{i,j} \leftarrow 0 \\
\text{for } m \in 1..nm \\
\quad \text{for } i \in 1..6 \\
\quad\quad \text{for } j \in 1..6 \\
\quad\quad\quad S_{(m-1)\cdot 3+i,(m-1)\cdot 3+j} \leftarrow K(m)_{i,j} \\
\text{for } m \in 1..nm-1 \\
\quad \text{if } TopCol = \text{"Pinned"} \\
\quad\quad S_{3\cdot m+1,3m+1} \leftarrow K(m)_{4,4} + K(m+1)_{1,1} + \frac{3E_{col}\cdot I_{col}}{L_{col}^{3}}\cdot\frac{in}{kip} \\
\quad\quad S_{3m+2,3m+2} \leftarrow K(m)_{5,5} + K(m+1)_{2,2} + \frac{E_{col}\cdot A_{col}}{L_{col}}\cdot\frac{in}{kip} \\
\quad\quad S_{3m+2,3m+3} \leftarrow K(m)_{5,6} + K(m+1)_{2,3} \\
\quad\quad S_{3m+3,3m+2} \leftarrow S_{3m+2,3m+3} \\
\quad\quad S_{3m+3,3m+3} \leftarrow K(m)_{6,6} + K(m+1)_{3,3} \\
\quad \text{otherwise} \\
\quad\quad S_{3\cdot m+1,3m+1} \leftarrow K(m)_{4,4} + K(m+1)_{1,1} + \frac{12E_{col}\cdot I_{col}}{L_{col}^{3}}\cdot\frac{in}{kip} \\
\quad\quad S_{3m+2,3m+2} \leftarrow K(m)_{5,5} + K(m+1)_{2,2} + \frac{E_{col}\cdot A_{col}}{L_{col}}\cdot\frac{in}{kip} \\
\quad\quad S_{3m+2,3m+3} \leftarrow K(m)_{5,6} + K(m+1)_{2,3} \\
\quad\quad S_{3m+3,3m+2} \leftarrow S_{3m+2,3m+3} \\
\quad\quad S_{3m+3,3m+3} \leftarrow K(m)_{6,6} + K(m+1)_{3,3} + \frac{4E_{col}\cdot I_{col}}{L_{col}}\cdot\frac{1}{kip\cdot in} \\
\quad\quad S_{3m+1,3m+3} \leftarrow \frac{6E_{col}\cdot I_{col}}{L_{col}^{2}}\cdot\frac{1}{kip} \\
\quad\quad S_{3m+3,3m+1} \leftarrow S_{3m+1,3m+3} \\
S
\end{array}
\right.
$$

Member Fixed-end Force Vector due to Partial Uniform Loads (Beam Vertical Loads)

$$
F_{f.py}(LS,m) := \left|
\begin{array}{l}
P \leftarrow P_{cap.beam_{LS}}{}^{T} \\
\text{if } m = 1 \\
\quad x1 \leftarrow 0 \\
\quad x2 \leftarrow L_{cap.oh}
\end{array}
\right.
$$

$\text{if } 1 < m < nm$

$x1 \leftarrow L_{cap.oh} + (m-2)Spacing_{col}$

$x2 \leftarrow x1 + Spacing_{col}$

otherwise

$x1 \leftarrow L_{cap.oh} + (m-2)Spacing_{col}$

$x2 \leftarrow x1 + L_{cap.oh}$

$L \leftarrow x2 - x1$

#LoadCases ← #LLCases

#LoadCases ← #FatLLCases if LS = 10

#LoadCases ← 1 if LS = 2 ∨ LS = 8

for lc ∈ 1 .. #LoadCases

for i ∈ 1 .. 6

$Q_{f_{i,lc}} \leftarrow 0$

for n ∈ 1 .. #Beams

$a \leftarrow \max\left(x1, x_{beam_n} - 0.5W_{load}\right) - x1$

$b \leftarrow \min\left(x2, x_{beam_n} + 0.5W_{load}\right) - x1$

if a < b

$q \leftarrow \frac{P_{n,lc}}{W_{load}}$

$$Q_{f_{3,lc}} \leftarrow Q_{f_{3,lc}} + \frac{q}{L^2}\cdot\int_a^b x\cdot(L-x)^2\,dx\cdot\frac{1}{kip\cdot in}$$

$$Q_{f_{6,lc}} \leftarrow Q_{f_{6,lc}} - \frac{q}{L^2}\cdot\int_a^b x^2\cdot(L-x)\,dx\cdot\frac{1}{kip\cdot in}$$

$$Q_{f_{2,lc}} \leftarrow Q_{f_{2,lc}} + \frac{q}{L^3}\cdot\int_a^b (L-x)^2\cdot(3x+L-x)\,dx\cdot\frac{1}{kip}$$

$$Q_{f_{5,lc}} \leftarrow Q_{f_{5,lc}} + \frac{q}{L^3}\cdot\int_a^b x^2\cdot(x+3L-3x)\,dx\cdot\frac{1}{kip}$$

Q_f

$F_{f.py}(1,1) = \text{BigMatrix}\{6,180\}$

Member Fixed-end Force Vector due to Uniform Loads

$$F_{f.w}(LS,m) := \begin{array}{|l} w \leftarrow w_{cap.y_{LS}} \\ L \leftarrow L_{cap.oh} \ \text{ if } \ m = 1 \vee m = nm \\ L \leftarrow Spacing_{col} \ \text{ if } \ 1 < m < nm \\ L \leftarrow 0 \ \text{ otherwise} \\ Q_{f_1} \leftarrow 0 \\ Q_{f_2} \leftarrow \frac{1}{2} \cdot w \cdot L \cdot \frac{1}{kip} \\ Q_{f_3} \leftarrow \frac{1}{12} \cdot w \cdot L^2 \cdot \frac{1}{kip \cdot in} \\ Q_{f_4} \leftarrow 0 \\ Q_{f_5} \leftarrow \frac{1}{2} \cdot w \cdot L \cdot \frac{1}{kip} \\ Q_{f_6} \leftarrow -\frac{1}{12} \cdot w \cdot L^2 \cdot \frac{1}{kip \cdot in} \\ Q_f \end{array}$$

$$F_{f.w}(1,1) = \begin{pmatrix} 0 \\ 4.63 \\ 37.07 \\ 0 \\ 4.63 \\ -37.07 \end{pmatrix}$$

Member Fixed-end Force Vector

$$F_f(LS,m) := \begin{array}{|l} F1 \leftarrow F_{f.py}(LS,m) \\ F2 \leftarrow F_{f.w}(LS,m) \\ \#LoadCases \leftarrow \#LLCases \\ \#LoadCases \leftarrow \#FatLLCases \ \text{ if } \ LS = 10 \\ \#LoadCases \leftarrow 1 \ \text{ if } \ LS = 2 \vee LS = 8 \\ \text{for } \ lc \in 1 .. \#LoadCases \\ \quad F_f^{\langle lc \rangle} \leftarrow F1^{\langle lc \rangle} + F2 \\ F_f \end{array}$$

$F_f(1,1) = \text{BigMatrix}\{6,180\}$

Structure Fixed-end Force Vector

$$
P_f(LS) := \left|
\begin{array}{l}
\text{for } m \in 1..nm \\
\quad F_m \leftarrow F_f(LS, m) \\
\#LoadCases \leftarrow \#LLCases \\
\#LoadCases \leftarrow \#FatLLCases \ \ \text{if } LS = 10 \\
\#LoadCases \leftarrow 1 \ \ \text{if } LS = 2 \vee LS = 8 \\
\text{for } lc \in 1..\#LoadCases \\
\quad \left|
\begin{array}{l}
P_{f_{1,lc}} \leftarrow (F_1)_{1,lc} \\
P_{f_{2,lc}} \leftarrow (F_1)_{2,lc} \\
P_{f_{3,lc}} \leftarrow (F_1)_{3,lc} \\
\text{for } m \in 1..nm - 1 \\
\quad \left|
\begin{array}{l}
P_{f_{3m+1,lc}} \leftarrow (F_m)_{4,lc} + (F_{m+1})_{1,lc} \\
P_{f_{3m+2,lc}} \leftarrow (F_m)_{5,lc} + (F_{m+1})_{2,lc} \\
P_{f_{3m+3,lc}} \leftarrow (F_m)_{6,lc} + (F_{m+1})_{3,lc}
\end{array}
\right. \\
P_{f_{3nm+1,lc}} \leftarrow (F_{nm})_{4,lc} \\
P_{f_{3nm+2,lc}} \leftarrow (F_{nm})_{5,lc} \\
P_{f_{3nm+3,lc}} \leftarrow (F_{nm})_{6,lc}
\end{array}
\right. \\
P_f
\end{array}
\right.
$$

$P_f(1) = \text{BigMatrix}\{24,180\}$

Form the Joint Load Vector

$$
P(LS) := \left|
\begin{array}{l}
X \leftarrow \dfrac{P_{cap.x_{LS}}}{kip} \\
\#LoadCases \leftarrow \#LLCases \\
\#LoadCases \leftarrow \#FatLLCases \ \ \text{if } LS = 10 \\
\#LoadCases \leftarrow 1 \ \ \text{if } LS = 2 \vee LS = 8 \\
\text{for } i \in 1..ndf \\
\quad \text{for } lc \in 1..\#LoadCases \\
\quad \quad P_{i,lc} \leftarrow 0 \\
\text{for } m \in 1..nm - 1 \\
\quad \text{for } lc \in 1..\#LoadCases \\
\quad \quad P_{3m+1,lc} \leftarrow \dfrac{X}{\#Cols} \\
P
\end{array}
\right.
$$

$P(1) = \text{BigMatrix}\{24,180\}$

Solve for Joint Displacements

$$D(LS) := S^{-1}\cdot\left(P(LS) - P_f(LS)\right)$$

$D(1) = \text{BigMatrix}\{24,180\}$

Member End Displacements

```
U(LS, m) := | DD ← D(LS)
            | #LoadCases ← #LLCases
            | #LoadCases ← #FatLLCases  if LS = 10
            | #LoadCases ← 1  if LS = 2 ∨ LS = 8
            | for i ∈ 1 .. 6
            |   for lc ∈ 1 .. #LoadCases
            |     U[i, lc ← DD[3(m−1)+i, lc
            | U
```

$U(1,1) = \text{BigMatrix}\{6,180\}$

Compute Member End Forces

$$F(LS,m) := K(m)\cdot U(LS,m) + F_f(LS,m)$$

$F(1,1) = \text{BigMatrix}\{6,180\}$

Matrix to store Member End Forces

```
MEF := | for LS ∈ 1 .. #LimitStates
       |   for m ∈ 1 .. nm
       |     MEF[LS, m ← F(LS, m)
       | MEF
```

Compute Forces Transferred to Columns

Function to compute the forces transferred to columns

$$fCOL(LS) := \left| \begin{array}{l} \#LoadCases \leftarrow \#LLCases \\ \#LoadCases \leftarrow \#FatLLCases \ \text{ if } \ LS = 10 \\ \#LoadCases \leftarrow 1 \ \text{ if } \ LS = 2 \lor LS = 8 \\ \text{for } \ n \in 1..\#Cols \\ \quad \text{for } \ lc \in 1..\#LoadCases \\ \quad \quad \left| \begin{array}{l} ColP_{n,lc} \leftarrow \left(MEF_{LS,n}\right)_{5,lc} \cdot kip + \left(MEF_{LS,n+1}\right)_{2,lc} \cdot kip \\ ColM_{n,lc} \leftarrow \left(MEF_{LS,n}\right)_{6,lc} \cdot kip \cdot in + \left(MEF_{LS,n+1}\right)_{3,lc} \cdot kip \cdot in \end{array} \right. \\ \left(\dfrac{ColP}{kip} \quad \dfrac{ColM}{kip \cdot in} \right)^T \end{array} \right.$$

Axial Force at Top of Column

$$ColP := \left| \begin{array}{l} \text{for } \ LS \in 1..\#LimitStates \\ \quad ColP_{LS} \leftarrow fCOL(LS)_1 \cdot kip \\ ColP \end{array} \right.$$

Bending Moment at Top of Column

$$ColM := \left| \begin{array}{l} \text{for } \ LS \in 1..\#LimitStates \\ \quad ColM_{LS} \leftarrow fCOL(LS)_2 \cdot kip \cdot in \\ ColM \end{array} \right.$$

Compute Bent Cap Shear and Moment Diagrams

Critical sections for max negative bending: center of colunms (left and right)

Critical sections for max positive bending: center of beams

Critical sections for max shear: face of columns (left and right)

$\#Sections := 4 \cdot \#Cols + 3 \cdot \#Beams = 51$ *Number of sections along the cap*

$$X := \left| \begin{array}{l} \text{for } \ i \in 1..\#Beams \\ \quad \left| \begin{array}{l} X_{3 \cdot i-2} \leftarrow x_{beam_i} - 0.5W_{load} \\ X_{3 \cdot i-1} \leftarrow x_{beam_i} \\ X_{3 \cdot i} \leftarrow x_{beam_i} + 0.5W_{load} \end{array} \right. \\ n \leftarrow 3 \cdot \#Beams \\ \text{for } \ j \in 1..\#Cols \\ \quad \left| \begin{array}{l} X_{n+4 \cdot j-3} \leftarrow x_{col_j} - 0.5W_{col} \\ X_{n+4 \cdot j-2} \leftarrow x_{col_j} - 0.01ft \\ X_{n+4 \cdot j-1} \leftarrow x_{col_j} + 0.01ft \\ X_{n+4 \cdot j} \leftarrow x_{col_j} + 0.5W_{col} \end{array} \right. \\ X \leftarrow sort(X) \\ X \end{array} \right.$$

$X_1 := \max(X_1, 0) \qquad X_{\#Sections} := \min(X_{\#Sections}, L_{cap})$

$X^T = (2.18 \;\; 3.25 \;\; 3.43 \;\; 3.99 \;\; 4.01 \;\; 4.68 \;\; 4.75 \;\; 8.81 \;\; 10.06 \;\; 11.31 \;\; 13.63 \;\; 14.37 \;\; 14.39 \;\; 15.13 \;\; 15.44 \;\; 16.69 \;\; 17.94 \;\; 22.$

Function to compute shear and moment demand at cap sections

$$
\begin{aligned}
fVM(LS) := \; & P \leftarrow {P_{cap.beam}}_{LS}^{T} \\
& w \leftarrow {w_{cap.y}}_{LS} \\
& \#LoadCases \leftarrow \#LLCases \\
& \#LoadCases \leftarrow \#FatLLCases \quad \text{if } LS = 10 \\
& \#LoadCases \leftarrow 1 \quad \text{if } LS = 2 \vee LS = 8 \\
& \text{for } s \in 1..\#Sections \\
& \quad m \leftarrow 1 \quad \text{if } X_s \le L_{cap.oh} \\
& \quad m \leftarrow \text{ceil}\left(\frac{X_s - L_{cap.oh}}{Spacing_{col}}\right) + 1 \quad \text{if } L_{cap.oh} < X_s \le L_{cap} - L_{cap.oh} \\
& \quad m \leftarrow nm \quad \text{otherwise} \\
& \quad x_{left} \leftarrow 0 \quad \text{if } m = 1 \\
& \quad x_{left} \leftarrow L_{cap.oh} + (m-2)\cdot Spacing_{col} \quad \text{otherwise} \\
& \quad V_{left} \leftarrow \left({MEF_{LS,m}}^{T}\right)^{\langle 2 \rangle}\cdot kip \\
& \quad M_{left} \leftarrow \left({MEF_{LS,m}}^{T}\right)^{\langle 3 \rangle}\cdot kip\cdot in \\
& \quad Xfs_s \leftarrow X_s \\
& \quad \text{"Redefine section coordinates to be at face of support if the section falls within faces of supports"} \\
& \quad \text{for } i \in 1..\#Cols \\
& \quad\quad Xfs_s \leftarrow x_{col_i} - 0.5W_{col} \quad \text{if } x_{col_i} - 0.5W_{col} < X_s \le x_{col_i} \\
& \quad\quad Xfs_s \leftarrow x_{col_i} + 0.5W_{col} \quad \text{if } x_{col_i} < X_s < x_{col_i} + 0.5W_{col} \\
& \quad \text{for } lc \in 1..\#LoadCases \\
& \quad\quad V_{s,lc} \leftarrow V_{left_{lc}} - w\cdot(X_s - x_{left}) \\
& \quad\quad Vfs_{s,lc} \leftarrow V_{left_{lc}} - w\cdot(Xfs_s - x_{left}) \\
& \quad\quad M_{s,lc} \leftarrow -M_{left_{lc}} - \frac{w}{2}\cdot(X_s - x_{left})^2 + V_{left_{lc}}\cdot(X_s - x_{left}) \\
& \quad\quad Mfs_{s,lc} \leftarrow -M_{left_{lc}} - \frac{w}{2}\cdot(Xfs_s - x_{left})^2 + V_{left_{lc}}\cdot(Xfs_s - x_{left}) \\
& \quad\quad \text{for } n \in 1..\#Beams \\
& \quad\quad\quad q \leftarrow \frac{P_{n,lc}}{W_{load}} \\
& \quad\quad\quad a \leftarrow \max(x_{left}, x_{beam_n} - 0.5W_{load}) \\
& \quad\quad\quad b \leftarrow \min(X \;\; x \;\; + 0.5W \;\;)
\end{aligned}
$$

$$b \leftarrow \min\left(X_s, x_{beam_n} + 0.5W_{load}\right)$$

$$bfs \leftarrow \min\left(Xfs_s, x_{beam_n} + 0.5W_{load}\right)$$

if $a < b$

$$V_{s,lc} \leftarrow V_{s,lc} - q\cdot(b - a)$$

$$M_{s,lc} \leftarrow M_{s,lc} - q\cdot(b - a)\cdot\left(X_s - \frac{a + b}{2}\right)$$

if $a < bfs$

$$Vfs_{s,lc} \leftarrow Vfs_{s,lc} - q\cdot(bfs - a)$$

$$Mfs_{s,lc} \leftarrow Mfs_{s,lc} - q\cdot(bfs - a)\cdot\left(Xfs_s - \frac{a + bfs}{2}\right)$$

$$\left(\frac{V}{kip} \quad \frac{Vfs}{kip} \quad \frac{M}{kip\cdot in} \quad \frac{Mfs}{kip\cdot in}\right)^T$$

Matrix to store shear and moment demands for all limit states

$$VM := \begin{array}{|l} \text{for } LS \in 1..\#LimitStates \\ \quad VM_{LS} \leftarrow fVM(LS) \\ VM \end{array}$$

Shear Diagram

$$V := \begin{array}{|l} \text{for } LS \in 1..\#LimitStates \\ \quad V_{LS} \leftarrow \left(VM_{LS}\right)_1\cdot kip \\ V \end{array}$$

Shear Diagram on the basis of shear at faces of support

$$Vfs := \begin{array}{|l} \text{for } LS \in 1..\#LimitStates \\ \quad Vfs_{LS} \leftarrow \left(VM_{LS}\right)_2\cdot kip \\ Vfs \end{array}$$

Moment Diagram

$$M := \begin{array}{|l} \text{for } LS \in 1..\#LimitStates \\ \quad M_{LS} \leftarrow \left(VM_{LS}\right)_3\cdot kip\cdot in \\ M \end{array}$$

Moment Diagram on the basis of moment at faces of support

$$Mfs := \begin{array}{|l} \text{for } LS \in 1..\#LimitStates \\ \quad Mfs_{LS} \leftarrow \left(VM_{LS}\right)_4\cdot kip\cdot in \\ Mfs \end{array}$$

Function to compute shear and moment envelopes along cap sections, and the correponding controlling load case

```
Envelope(A,LS) := | #LoadCases ← #LLCases
                  | #LoadCases ← #FatLLCases  if LS = 10
                  | #LoadCases ← 1  if LS = 2 ∨ LS = 8
                  | for s ∈ 1 .. #Sections
                  |   | max_s ← A_s,1
                  |   | max_lc_s ← 1
                  |   | min_s ← A_s,1
                  |   | min_lc_s ← 1
                  |   | for lc ∈ 2 .. #LoadCases   if #LoadCases > 1
                  |   |   | if A_s,lc > max_s
                  |   |   |   | max_s ← A_s,lc
                  |   |   |   | max_lc_s ← lc
                  |   |   | if A_s,lc < min_s
                  |   |   |   | min_s ← A_s,lc
                  |   |   |   | min_lc_s ← lc
                  | (max  max_lc  min  min_lc)^T
```

Matrices of shear and moment envelopes along cap sections, and the corresponding cotrolling load case
All matrices: #LimitStates x #Sections

```
maxV := | for LS ∈ 1 .. #LimitStates
        |   max^⟨LS⟩ ← Envelope(V_LS/kip, LS)_1
        | max^T·kip

maxV_lc := | for LS ∈ 1 .. #LimitStates
           |   max_lc^⟨LS⟩ ← Envelope(V_LS/kip, LS)_2
           | max_lc^T

minV := | for LS ∈ 1 .. #LimitStates
        |   min^⟨LS⟩ ← Envelope(V_LS/kip, LS)_3
        | min^T·kip

minV_lc := | for LS ∈ 1 .. #LimitStates
           |   min_lc^⟨LS⟩ ← Envelope(V_LS/kip, LS)_4
           | min_lc^T
```

$$maxVfs := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad max^{\langle LS \rangle} \leftarrow Envelope\left(\frac{Vfs_{LS}}{kip}, LS\right)_1 \\ max^T \cdot kip \end{array} \right.$$

$$maxVfs\_lc := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad max\_lc^{\langle LS \rangle} \leftarrow Envelope\left(\frac{Vfs_{LS}}{kip}, LS\right)_2 \\ max\_lc^T \end{array} \right.$$

$$minVfs := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad min^{\langle LS \rangle} \leftarrow Envelope\left(\frac{Vfs_{LS}}{kip}, LS\right)_3 \\ min^T \cdot kip \end{array} \right.$$

$$minVfs\_lc := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad min\_lc^{\langle LS \rangle} \leftarrow Envelope\left(\frac{Vfs_{LS}}{kip}, LS\right)_4 \\ min\_lc^T \end{array} \right.$$

$$maxM := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad max^{\langle LS \rangle} \leftarrow Envelope\left(\frac{M_{LS}}{kip \cdot in}, LS\right)_1 \\ max^T \cdot kip \cdot in \end{array} \right.$$

$$maxM\_lc := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad max\_lc^{\langle LS \rangle} \leftarrow Envelope\left(\frac{M_{LS}}{kip \cdot in}, LS\right)_2 \\ max\_lc^T \end{array} \right.$$

$$minM := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad min^{\langle LS \rangle} \leftarrow Envelope\left(\frac{M_{LS}}{kip \cdot in}, LS\right)_3 \\ min^T \cdot kip \cdot in \end{array} \right.$$

$$minM\_lc := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad min\_lc^{\langle LS \rangle} \leftarrow Envelope\left(\frac{M_{LS}}{kip \cdot in}, LS\right)_4 \\ min\_lc^T \end{array} \right.$$

$$maxMfs := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad max^{\langle LS \rangle} \leftarrow Envelope\left(\frac{Mfs_{LS}}{kip \cdot in}, LS\right)_1 \\ max^T \cdot kip \cdot in \end{array} \right.$$

$$maxMfs\_lc := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad max\_lc^{\langle LS \rangle} \leftarrow Envelope\left(\frac{Mfs_{LS}}{kip \cdot in}, LS\right)_2 \\ max\_lc^T \end{array} \right.$$

$$\text{minMfs} := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad \text{min}^{\langle LS \rangle} \leftarrow \text{Envelope}\left(\frac{Mfs_{LS}}{kip \cdot in}, LS\right)_3 \\ \text{min}^T \cdot kip \cdot in \end{array} \right.$$

$$\text{minMfs\_lc} := \left| \begin{array}{l} \text{for } LS \in 1..\#LimitStates \\ \quad \text{min\_lc}^{\langle LS \rangle} \leftarrow \text{Envelope}\left(\frac{Mfs_{LS}}{kip \cdot in}, LS\right)_4 \\ \text{min\_lc}^T \end{array} \right.$$

Develop Stiffness Matrix, Member Force Matrix and Solve for Cap Moment & Shear Forces

Summary Results

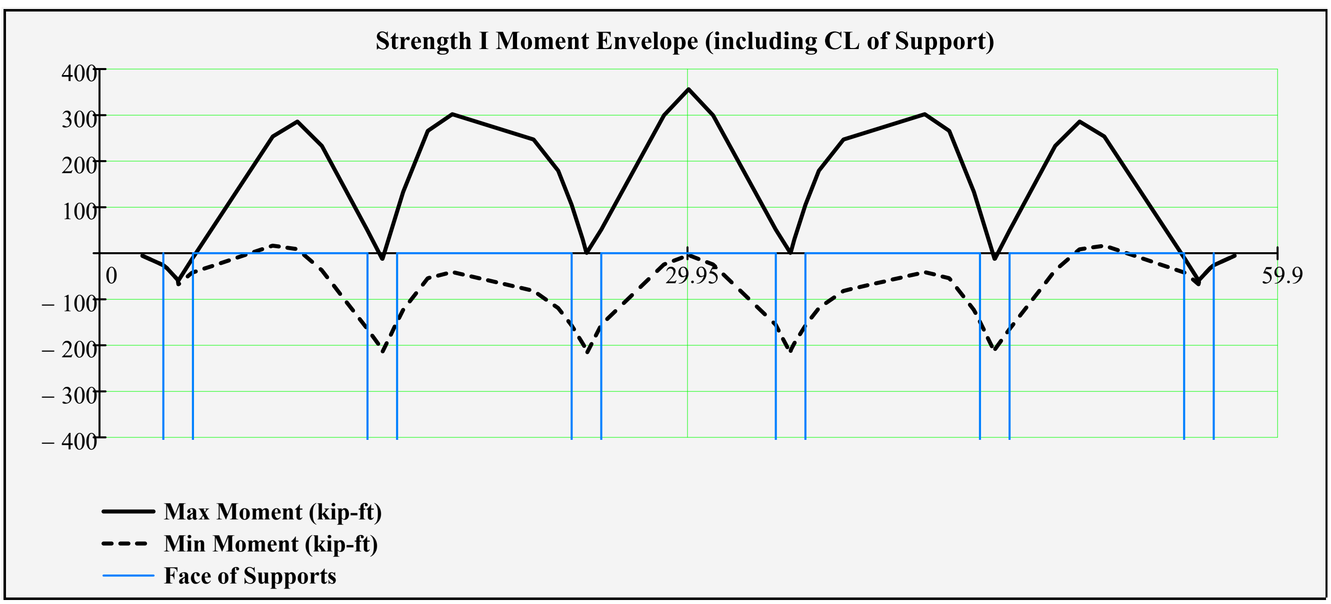
Coordinates used to sketch the columns

$$XX := \left| \begin{array}{l} \text{for } j \in 1..\#Cols \\ \quad \left| \begin{array}{l} XX_{4(j-1)+1} \leftarrow x_{col_j} - 0.5W_{col} \\ XX_{4(j-1)+2} \leftarrow x_{col_j} - 0.5W_{col} \\ XX_{4(j-1)+3} \leftarrow x_{col_j} + 0.5W_{col} \\ XX_{4(j-1)+4} \leftarrow x_{col_j} + 0.5W_{col} \end{array} \right. \\ XX \end{array} \right.$$

$$YY := \left| \begin{array}{l} \text{for } j \in 1..\#Cols \\ \quad \left| \begin{array}{l} YY_{4(j-1)+1} \leftarrow 0 \\ YY_{4(j-1)+2} \leftarrow -5000 \\ YY_{4(j-1)+3} \leftarrow -5000 \\ YY_{4(j-1)+4} \leftarrow 0 \end{array} \right. \\ YY \end{array} \right.$$

Summary Results

Limit State Strength I (max vertical load)



Maximum Moment (inlcuding CL of support)

Minimum Moment (inlcuding CL of support)

Moment value

$$\text{StrI\_maxM} := \max\left[\left(\text{maxM}^T\right)^{\langle 1 \rangle}\right] = 356.35 \cdot kip \cdot ft$$

$$\text{StrI\_minM} := \min\left[\left(\text{minM}^T\right)^{\langle 1 \rangle}\right] = -217.25 \cdot kip \cdot ft$$

Section #

$StrI\_maxM\_s\# := match\left[StrI\_maxM, \left(maxM^T\right)^{\langle 1 \rangle}\right] = (26)$

$StrI\_minM\_s\# := match\left[StrI\_minM, \left(minM^T\right)^{\langle 1 \rangle}\right] = \begin{pmatrix} 23 \\ 29 \end{pmatrix}$

Location

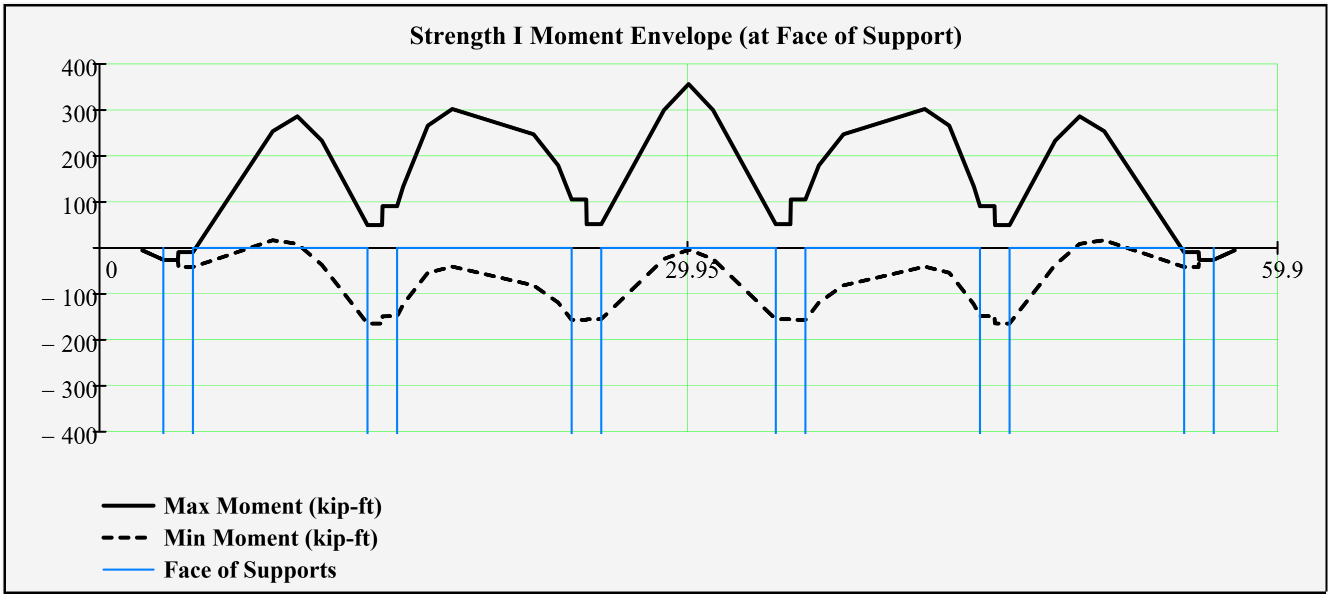
$StrI\_maxM\_loc := X_{\left(StrI\_maxM\_s\#_1\right)} = 29.95\ ft$

$StrI\_minM\_loc := X_{\left(StrI\_minM\_s\#_1\right)} = 24.77\ ft$

Load case

$StrI\_maxM\_lc := \left[\left(maxM\_lc^T\right)^{\langle 1 \rangle}\right]_{\left(StrI\_maxM\_s\#_1\right)} = 71$

$StrI\_minM\_lc := \left[\left(minM\_lc^T\right)^{\langle 1 \rangle}\right]_{\left(StrI\_minM\_s\#_1\right)} = 167$



Maximum Moment (at face of support)

Moment value

$StrI\_maxMfs := max\left[\left(maxMfs^T\right)^{\langle 1 \rangle}\right] = 356.35 \cdot kip \cdot ft$

Section #

$StrI\_maxMfs\_s\# := match\left[StrI\_maxMfs, \left(maxMfs^T\right)^{\langle 1 \rangle}\right]$

$StrI\_maxMfs\_s\# = (26)$

Location

$StrI\_maxMfs\_loc := X_{\left(StrI\_maxMfs\_s\#_1\right)} = 29.95\ ft$

Load case

$StrI\_maxMfs\_lc := \left[\left(maxMfs\_lc^T\right)^{\langle 1 \rangle}\right]_{\left(StrI\_maxMfs\_s\#_1\right)}$

$StrI\_maxMfs\_lc = 71$

Minimum Moment (at face of support)

$StrI\_minMfs := min\left[\left(minMfs^T\right)^{\langle 1 \rangle}\right] = -164.87 \cdot kip \cdot ft$

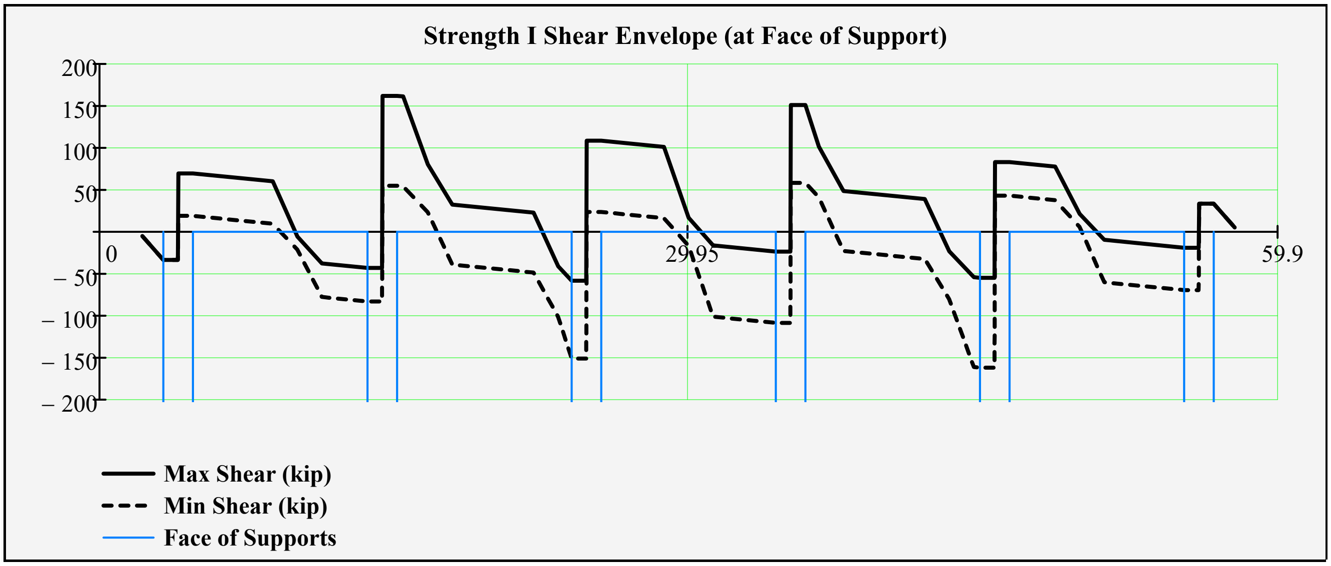
$StrI\_minMfs\_s\# := match\left[StrI\_minMfs, \left(minMfs^T\right)^{\langle 1 \rangle}\right]$

$StrI\_minMfs\_s\# = \begin{pmatrix} 11 \\ 12 \\ 40 \\ \ldots \end{pmatrix}$

$StrI\_minMfs\_loc := X_{\left(StrI\_minMfs\_s\#_1\right)} = 13.63\ ft$

$StrI\_minMfs\_lc := \left[\left(minMfs\_lc^T\right)^{\langle 1 \rangle}\right]_{\left(StrI\_minMfs\_s\#_1\right)}$

$StrI\_minMfs\_lc = 104$



Maximum Shear (at face of support)

Shear value

$StrI\_maxVfs := max\left[\left(maxVfs^{T}\right)^{\langle 1\rangle}\right] = 161.97\cdot kip$

Section #

$StrI\_maxVfs\_s\# := match\left[StrI\_maxVfs, \left(maxVfs^{T}\right)^{\langle 1\rangle}\right] = \begin{pmatrix} 13 \\ 14 \end{pmatrix}$

Location

$StrI\_maxVfs\_loc := X_{\left(StrI\_maxVfs\_s\#_1\right)} = 14.39\ ft$

Load case

$StrI\_maxVfs\_lc := \left[\left(maxVfs\_lc^{T}\right)^{\langle 1\rangle}\right]_{\left(StrI\_maxVfs\_s\#_1\right)} = 28$

Minimum Shear (at face of support)

$StrI\_minVfs := min\left[\left(minVfs^{T}\right)^{\langle 1\rangle}\right] = -161.97\cdot kip$

$StrI\_minVfs\_s\# := match\left[StrI\_minVfs, \left(minVfs^{T}\right)^{\langle 1\rangle}\right] = \begin{pmatrix} 38 \\ 39 \end{pmatrix}$

$StrI\_minVfs\_loc := X_{\left(StrI\_minVfs\_s\#_1\right)} = 44.77\ ft$

$StrI\_minVfs\_lc := \left[\left(minVfs\_lc^{T}\right)^{\langle 1\rangle}\right]_{\left(StrI\_minVfs\_s\#_1\right)} = 151$

Limit State Strength III (max vertical load)

Maximum Shear (including CL of support)

$StrIII\_maxV := max\left[\left(maxV^{T}\right)^{\langle 2\rangle}\right] = 71.49\cdot kip$

Minimum Shear (including CL of support)

$StrIII\_minV := min\left[\left(minV^{T}\right)^{\langle 2\rangle}\right] = -74.47\cdot kip$

Maximum Shear (at face of support)

$StrIII\_maxVfs := max\left[\left(maxVfs^{T}\right)^{\langle 2\rangle}\right] = 56.8\cdot kip$

Minimum Shear (at face of support)

$StrIII\_minVfs := min\left[\left(minVfs^{T}\right)^{\langle 2\rangle}\right] = -68.45\cdot kip$

Maximum Moment (including CL of support)

$StrIII\_maxM := max\left[\left(maxM^{T}\right)^{\langle 2\rangle}\right] = 77.76\cdot kip\cdot ft$

Minimum Moment (including CL of support)

$StrIII\_minM := min\left[\left(minM^{T}\right)^{\langle 2\rangle}\right] = -119.1\cdot kip\cdot ft$

Maximum Moment (at face of support)

Minimum Moment (at face of support)

$StrIII\_maxMfs := max\left[\left(maxMfs^T\right)^{\langle 2\rangle}\right] = 77.76 \cdot kip \cdot ft$

$StrIII\_minMfs := min\left[\left(minMfs^T\right)^{\langle 2\rangle}\right] = -81.46 \cdot kip \cdot ft$

Limit State Strength V (max vertical load)

Maximum Shear (including CL of support)

$StrV\_maxV := max\left[\left(maxV^T\right)^{\langle 3\rangle}\right] = 162.89 \cdot kip$

Minimum Shear (including CL of support)

$StrV\_minV := min\left[\left(minV^T\right)^{\langle 3\rangle}\right] = -167.66 \cdot kip$

Maximum Shear (at face of support)

$StrV\_maxVfs := max\left[\left(maxVfs^T\right)^{\langle 3\rangle}\right] = 136.32 \cdot kip$

Minimum Shear (at face of support)

$StrV\_minVfs := min\left[\left(minVfs^T\right)^{\langle 3\rangle}\right] = -141.7 \cdot kip$

Maximum Moment (including CL of support)

$StrV\_maxM := max\left[\left(maxM^T\right)^{\langle 3\rangle}\right] = 289.39 \cdot kip \cdot ft$

Minimum Moment (including CL of support)

$StrV\_minM := min\left[\left(minM^T\right)^{\langle 3\rangle}\right] = -201.51 \cdot kip \cdot ft$

Maximum Moment (at face of support)

$StrV\_maxMfs := max\left[\left(maxMfs^T\right)^{\langle 3\rangle}\right] = 289.39 \cdot kip \cdot ft$

Minimum Moment (at face of support)

$StrV\_minMfs := min\left[\left(minMfs^T\right)^{\langle 3\rangle}\right] = -146.03 \cdot kip \cdot ft$

Max/Min Factored Shear and Moment of Strength I, III and V at Bent Cap Sections

$s := 1 .. \#Sections$

Including CL of support

$maxV_{u.cl_s} := max\left(maxV_{1,s}, maxV_{2,s}, maxV_{3,s}\right)$

$minV_{u.cl_s} := min\left(minV_{1,s}, minV_{2,s}, minV_{3,s}\right)$

$maxM_{u.cl_s} := max\left(maxM_{1,s}, maxM_{2,s}, maxM_{3,s}\right)$

$minM_{u.cl_s} := min\left(minM_{1,s}, minM_{2,s}, minM_{3,s}\right)$

At face of support

$maxV_{u.fs_s} := max\left(maxVfs_{1,s}, maxVfs_{2,s}, maxVfs_{3,s}\right)$

$minV_{u.fs_s} := min\left(minVfs_{1,s}, minVfs_{2,s}, minVfs_{3,s}\right)$

$maxM_{u.fs_s} := max\left(maxMfs_{1,s}, maxMfs_{2,s}, maxMfs_{3,s}\right)$

$minM_{u.fs_s} := min\left(minMfs_{1,s}, minMfs_{2,s}, minMfs_{3,s}\right)$

Factored Moment of Service I at Bent Cap Sections

Including CL of support

$maxM_{SerI.cl_s} := maxM_{4,s}$

$minM_{SerI.cl_s} := minM_{4,s}$

At face of support

$maxM_{SerI.fs_s} := maxMfs_{4,s}$

$minM_{SerI.fs_s} := minMfs_{4,s}$

Factored Moment of Service III at Bent Cap Sections

Including CL of support

$maxM_{SerIII.cl_s} := maxM_{5,s}$

$minM_{SerIII.cl_s} := minM_{5,s}$

At face of support

$maxM_{SerIII.fs_s} := maxMfs_{5,s}$

$minM_{SerIII.fs_s} := minMfs_{5,s}$

Moment of Sustained Load: DL+0.2LL at Bent Cap Sections

Including CL of support

$maxM_{SL.cl_s} := maxM_{6,s}$ $\quad$ $minM_{SL.cl_s} := minM_{6,s}$

At face of support

$maxM_{SL.fs_s} := maxMfs_{6,s}$ $\quad$ $minM_{SL.fs_s} := minMfs_{6,s}$

Moment of DL+Fatigue I at Bent Cap Sections

Including CL of support

$maxM_{FatI.cl_s} := maxM_{10,s}$ $\quad$ $minM_{FatI.cl_s} := minM_{10,s}$

At face of support

$maxM_{FatI.fs_s} := maxMfs_{10,s}$ $\quad$ $minM_{FatI.fs_s} := minMfs_{10,s}$

Factored Design Forces at Top of Columns

Maximum Axial Load at top of columns

```
maxColP := | for n ∈ 1 .. #Cols
           |   | for LS ∈ 1 .. 3
           |   |    maxColP[LS,n] ← max[(ColP[LS]^T)^<n>]
           |   | for LS ∈ 7 .. 9
           |   |    maxColP[LS−3,n] ← max[(ColP[LS]^T)^<n>]
           | maxColP
```

Col #: 1 2 ....

$$maxColP = \begin{pmatrix} 141.06 & 236.72 & 267.86 & 267.86 & 236.72 & 141.06 \\ 87.84 & 105.84 & 111.19 & 114.98 & 117.9 & 114.89 \\ 128.57 & 208.2 & 232.65 & 232.32 & 208.17 & 135.42 \\ 116.07 & 209.08 & 239.9 & 239.9 & 209.08 & 116.07 \\ 62.84 & 78.2 & 83.22 & 87.02 & 90.26 & 89.88 \\ 103.56 & 180.56 & 204.69 & 204.36 & 180.53 & 110.42 \end{pmatrix} \cdot kip \quad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \end{pmatrix}$$

Minimum Axial Load at top of columns

$$minColP := \left| \begin{array}{l} \text{for } n \in 1..\#Cols \\ \quad \left| \begin{array}{l} \text{for } LS \in 1..3 \\ \quad minColP_{LS,n} \leftarrow \min\left[\left(ColP_{LS}{}^{T}\right)^{\langle n \rangle}\right] \\ \text{for } LS \in 7..9 \\ \quad minColP_{LS-3,n} \leftarrow \min\left[\left(ColP_{LS}{}^{T}\right)^{\langle n \rangle}\right] \end{array} \right. \\ minColP \end{array} \right.$$

Col #: *1* *2* *....*

$$minColP = \begin{pmatrix} 90.52 & 107.42 & 117.52 & 117.52 & 107.42 & 90.52 \\ 87.84 & 105.84 & 111.19 & 114.98 & 117.9 & 114.89 \\ 89.57 & 108.45 & 116.68 & 116.35 & 108.42 & 96.43 \\ 65.53 & 79.77 & 89.56 & 89.56 & 79.77 & 65.53 \\ 62.84 & 78.2 & 83.22 & 87.02 & 90.26 & 89.88 \\ 64.57 & 80.81 & 88.71 & 88.38 & 80.78 & 71.43 \end{pmatrix} \cdot kip \qquad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \end{pmatrix}$$

Maximum Moment at top of columns (in the plane of bent-cap)

$$maxColM := \left| \begin{array}{l} \text{for } n \in 1..\#Cols \\ \quad \left| \begin{array}{l} \text{for } LS \in 1..3 \\ \quad maxColM_{LS,n} \leftarrow \max\left[\left(ColM_{LS}{}^{T}\right)^{\langle n \rangle}\right] \\ \text{for } LS \in 7..9 \\ \quad maxColM_{LS-3,n} \leftarrow \max\left[\left(ColM_{LS}{}^{T}\right)^{\langle n \rangle}\right] \end{array} \right. \\ maxColM \end{array} \right.$$

Col #: *1* *2* *....*

$$maxColM = \begin{pmatrix} 10 & 8 & 8 & 6 & -0 & -1 \\ -30 & -32 & -32 & -32 & -32 & -33 \\ -14 & -16 & -17 & -18 & -23 & -23 \\ 10 & 8 & 8 & 6 & -0 & -1 \\ -30 & -32 & -32 & -32 & -32 & -32 \\ -14 & -16 & -17 & -18 & -23 & -23 \end{pmatrix} ft \cdot kip \qquad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \end{pmatrix}$$

Maximum Moment at top of columns (in the plane of bent-cap)

$$minColM := \left| \begin{array}{l} \text{for } n \in 1..\#Cols \\ \quad \left| \begin{array}{l} \text{for } LS \in 1..3 \\ \quad minColM_{LS,n} \leftarrow \min\left[\left(ColM_{LS}^{T}\right)^{\langle n \rangle}\right] \\ \text{for } LS \in 7..9 \\ \quad minColM_{LS-3,n} \leftarrow \min\left[\left(ColM_{LS}^{T}\right)^{\langle n \rangle}\right] \end{array} \right. \\ minColM \end{array} \right.$$

Col #: 1 2 ....

$$minColM = \begin{pmatrix} 1 & 0 & -6 & -8 & -8 & -10 \\ -30 & -32 & -32 & -32 & -32 & -33 \\ -21 & -22 & -28 & -29 & -29 & -30 \\ 1 & 0 & -6 & -8 & -8 & -10 \\ -30 & -32 & -32 & -32 & -32 & -32 \\ -21 & -22 & -28 & -29 & -29 & -30 \end{pmatrix} ft \cdot kip \qquad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \end{pmatrix}$$

Factored Shear Force at top of columns (in the plane of bent-cap)

$$ColV_x := \left| \begin{array}{l} \text{for } n \in 1..\#Cols \\ \quad \left| \begin{array}{l} \text{for } LS \in 1..3 \\ \quad ColV_{x_{LS,n}} \leftarrow \dfrac{P_{cap.x_{LS}}}{\#Cols} \\ \text{for } LS \in 7..9 \\ \quad ColV_{x_{LS-3,n}} \leftarrow \dfrac{P_{cap.x_{LS}}}{\#Cols} \end{array} \right. \\ ColV_x \end{array} \right.$$

Col #: 1 2 ....

$$ColV_x = \begin{pmatrix} 0 & 0 & 0 & 0 & 0 & 0 \\ 1.89 & 1.89 & 1.89 & 1.89 & 1.89 & 1.89 \\ 1.34 & 1.34 & 1.34 & 1.34 & 1.34 & 1.34 \\ 0 & 0 & 0 & 0 & 0 & 0 \\ 1.89 & 1.89 & 1.89 & 1.89 & 1.89 & 1.89 \\ 1.34 & 1.34 & 1.34 & 1.34 & 1.34 & 1.34 \end{pmatrix} \cdot kip \qquad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \end{pmatrix}$$

Factored Shear Force at top of columns (perpendicular to bent-cap)

$$ColV_z := \left| \begin{array}{l} \text{for } n \in 1..\#Cols \\ \quad \left| \begin{array}{l} \text{for } LS \in 1..3 \\ \quad ColV_{x_{LS,n}} \leftarrow \dfrac{P_{cap.z_{LS}}}{\#Cols} \\ \text{for } LS \in 7..9 \\ \quad ColV_{x_{LS-3,n}} \leftarrow \dfrac{P_{cap.z_{LS}}}{\#Cols} \end{array} \right. \\ ColV_x \end{array} \right.$$

Col #: 1 2 ....

$$ColV_z = \begin{pmatrix} 13.39 & 13.39 & 13.39 & 13.39 & 13.39 & 13.39 \\ 1.34 & 1.34 & 1.34 & 1.34 & 1.34 & 1.34 \\ 11.07 & 11.07 & 11.07 & 11.07 & 11.07 & 11.07 \\ 13.39 & 13.39 & 13.39 & 13.39 & 13.39 & 13.39 \\ 1.34 & 1.34 & 1.34 & 1.34 & 1.34 & 1.34 \\ 11.07 & 11.07 & 11.07 & 11.07 & 11.07 & 11.07 \end{pmatrix} \cdot kip \qquad \begin{pmatrix} \text{"Strength I max vertical"} \\ \text{"Strength III max vertical"} \\ \text{"Strength V max vertical"} \\ \text{"Strength I min vertical"} \\ \text{"Strength III min vertical"} \\ \text{"Strength V min vertical"} \end{pmatrix}$$

Factored Column Net Axial Force (for pile pocket connection punching shear check)

Net column axial force for punching shear check (column axial force subtract what is directly on top of column)

$$
\text{PunchingV} := \left|
\begin{array}{l}
j \leftarrow 0 \\
\text{for } LS \in 1,2,3,7,8,9 \\
\quad \left|
\begin{array}{l}
\#\text{LoadCases} \leftarrow \#\text{LLCases} \\
\#\text{LoadCases} \leftarrow 1 \quad \text{if } LS = 2 \vee LS = 6 \vee LS = 8 \\
\text{for } lc \in 1 .. \#\text{LoadCases} \\
\quad \left|
\begin{array}{l}
\text{"V is the shear diagram along all cap sections"} \\
VV \leftarrow \left(V_{LS}\right)^{\langle lc \rangle} \\
\text{for } n \in 1 .. \#\text{Cols} \\
\quad \left|
\begin{array}{l}
X_{left} \leftarrow x_{col_n} - 0.5W_{col} \\
X_{right} \leftarrow x_{col_n} + 0.5W_{col} \\
\text{"find the shear at left and right of column"} \\
V_{left} \leftarrow \text{lookup}\left(X_{left}, X, VV\right) \\
V_{right} \leftarrow \text{lookup}\left(X_{right}, X, VV\right) \\
PV_{lc,n} \leftarrow V_{right_1} - V_{left_1}
\end{array}
\right.
\end{array}
\right. \\
j \leftarrow j + 1 \\
\text{for } n \in 1 .. \#\text{Cols} \\
\quad \text{PunchingV}_{j,n} \leftarrow \max\left(PV^{\langle n \rangle}\right)
\end{array}
\right. \\
\text{PunchingV}
\end{array}
\right.
$$

Col #: 1 2 ....

$$
\text{PunchingV} = \begin{pmatrix}
102.96 & 233.24 & 224.05 & 224.05 & 233.24 & 102.96 \\
99.24 & 233.24 & 224.05 & 224.05 & 233.24 & 102.96 \\
90.7 & 204.73 & 195.21 & 194.83 & 204.7 & 97.08 \\
87.16 & 206.58 & 200.05 & 200.05 & 206.58 & 87.16 \\
83.44 & 206.58 & 200.05 & 200.05 & 206.58 & 87.16 \\
74.89 & 178.06 & 171.21 & 170.84 & 178.03 & 81.28
\end{pmatrix} \cdot \text{kip}
\qquad
\begin{pmatrix}
\text{"Strength I max vertical"} \\
\text{"Strength III max vertical"} \\
\text{"Strength V max vertical"} \\
\text{"Strength I min vertical"} \\
\text{"Strength III min vertical"} \\
\text{"Strength V min vertical"}
\end{pmatrix}
$$

Summary Results

Write Design Forces Out

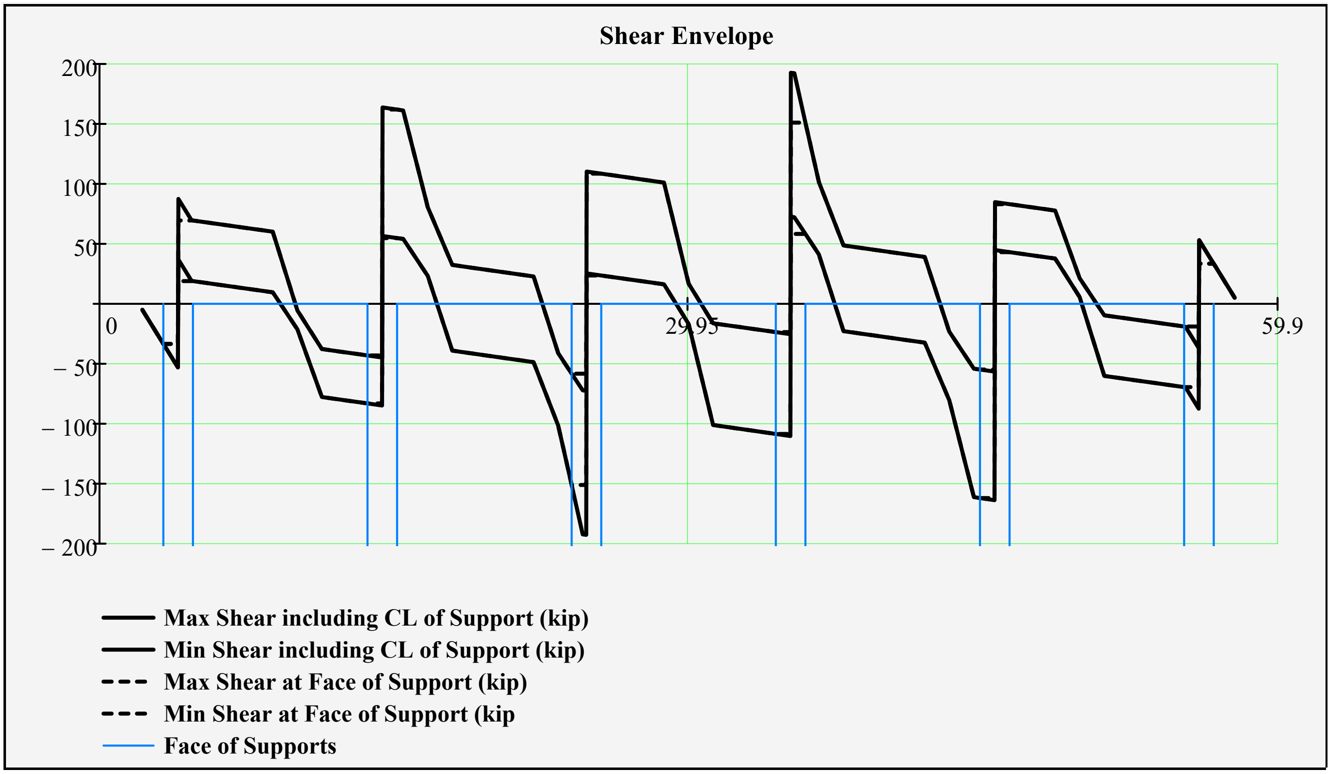
Choose a Limit State to Display the Shear and Moment Envelops

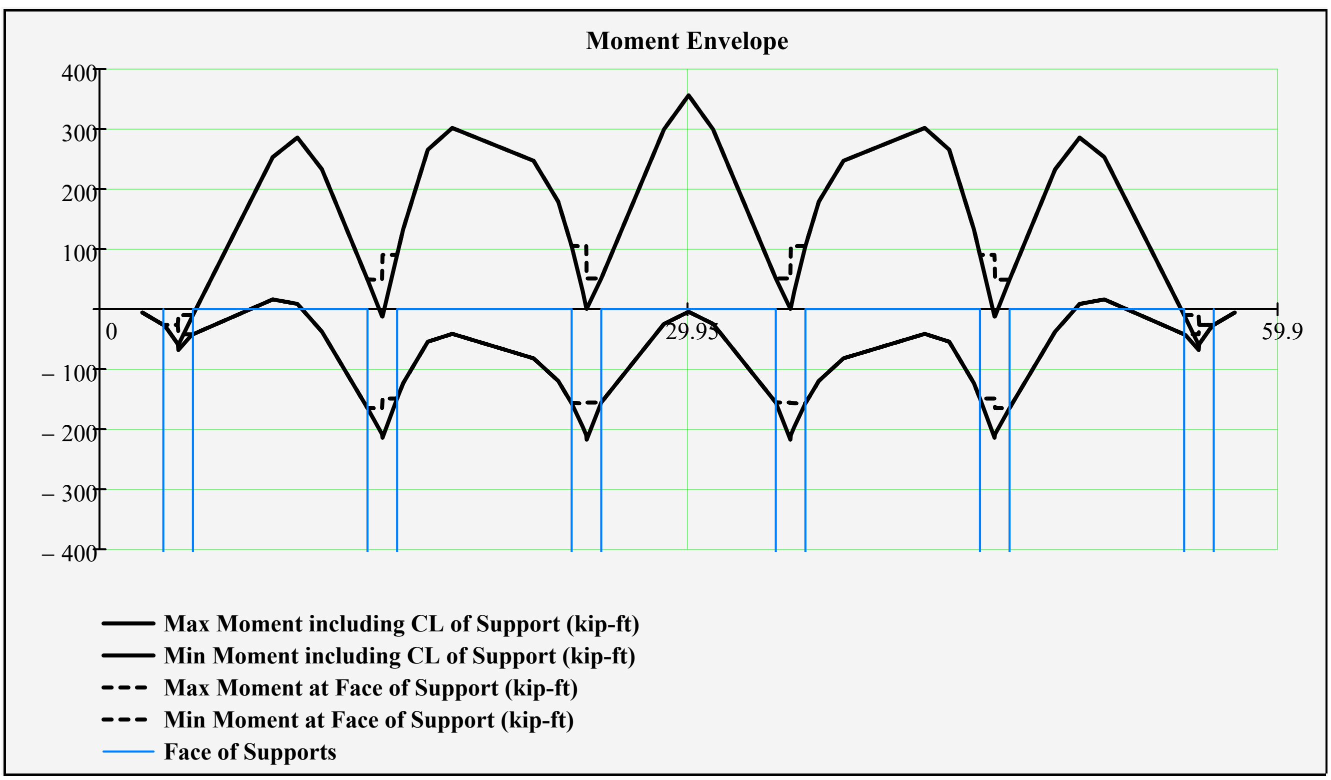
Calculate Worksheet

Choose a Design Limit State

LS := Strength I max Vertical

LS = 1





Choose a Specific Load Case to Display the Shear and Moment Diagrams

#LoadCases := if(LS = 10, #FatLLCases, if(LS = 2 ∨ LS = 8, 1, #LLCases)) = 180

Choose a Live Load Case

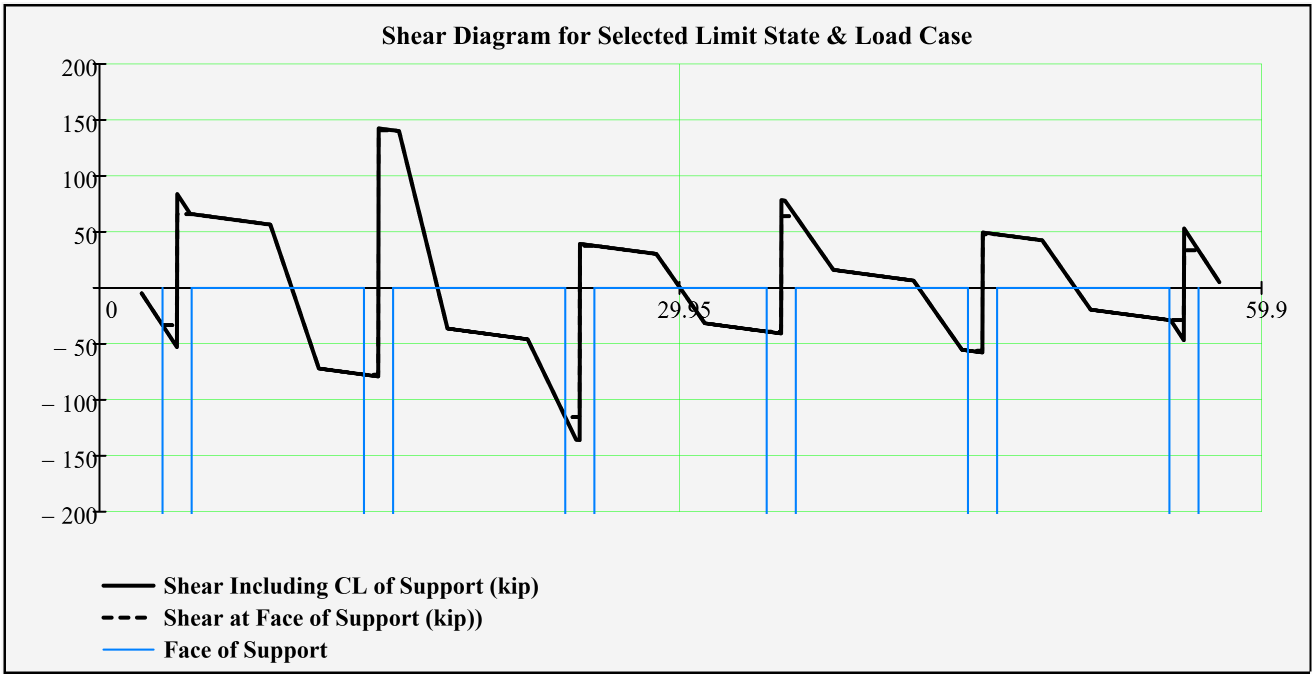
LoadCase = 95 LS = 1

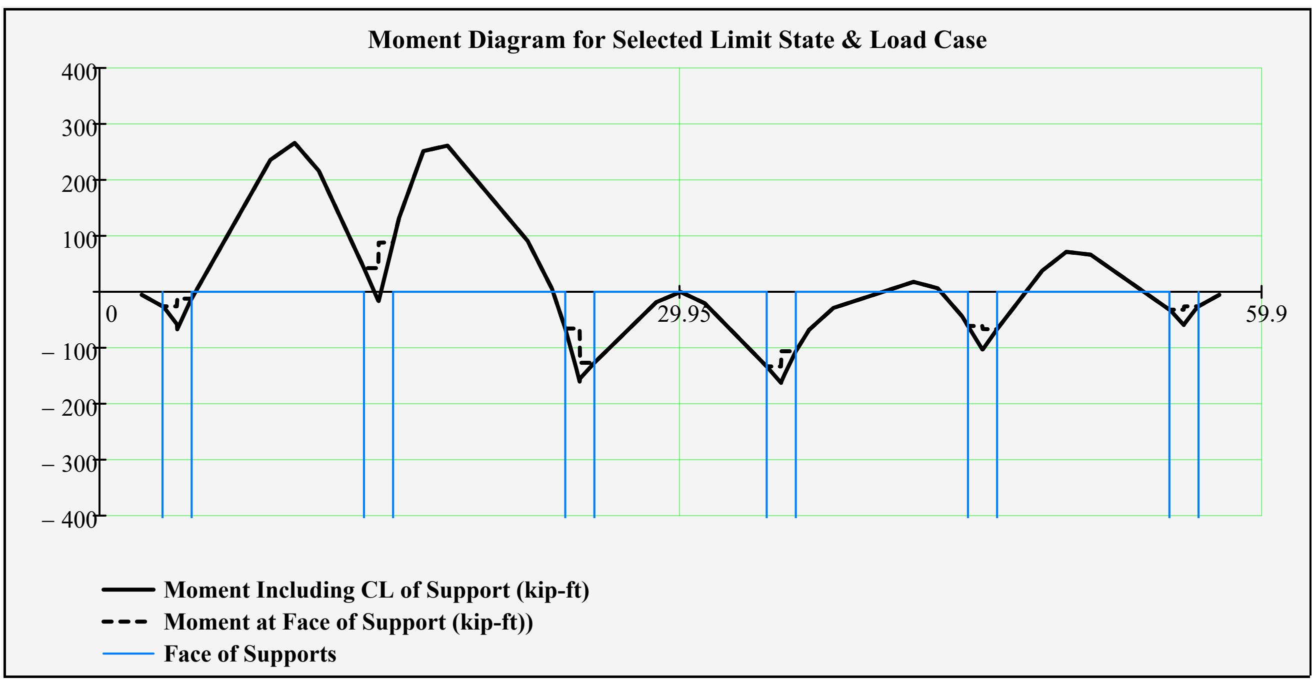
Factored beam loads

$$\left(P_{cap.beam_{LS}}^{T}\right)^{\langle LoadCase \rangle T} = (60.53 \quad 122.8 \quad 170.53 \quad 83.98 \quad 56.11 \quad 56.11 \quad 56.11 \quad 56.11 \quad 60.53$$

Factored Lateral Load on Bent Cap

$$P_{cap.x_{LS}} = 0 \cdot kip$$





Intermediate Bent-Cap Analysis & Design
Part 3. Code Checks (GFRP)

State Structures Design Office

Project =
Designed By =
Checked By =
Back Checked By =

Read In Geometry and Design Loads

DataFile = "HRB-NK.dat"

Load Data

GFRP Design References

- *FDOT Structures Manual, Volume 4, 2018*
- *FDOT Specifications Section 932-3, July 2018*
- *AASHTO Bridge Design Specifications for GFRP Reinforced Concrete, 2nd Edition*

Bent Cap and Column Properties

$H_{cap} = 3\ ft$	*Height of bent cap*
$W_{cap} = 4\ ft$	*Width of bent cap*
$L_{cap} = 59.9\ ft$	*Length of bent cap*
$L_{cap.oh} = 4\ ft$	*Length of cap overhang*
$W_{col} = 1.5\ ft$	*Column diameter/width*
$ColType = 2$	*Column type: 1-round; 2-square*
$\#Cols = 6$	*Number of columns*
$Spacing_{col} = 10.38\ ft$	*Spacing of columns*
$f_{c.cap} = 5.5 \cdot ksi$	*Minimum 28-day compressive strength for substructure*
$E_{cap} = 4428 \cdot ksi$	*Modulus of elasticity for concrete substructure*
$f_{c.col} = 6.0 \cdot ksi$	*Minimum 28-day compressive strength for substructure*
$E_{col} = 4557 \cdot ksi$	*Modulus of elasticity for concrete substructure*
$\gamma_{conc} = 150 \cdot pcf$	*Unit weight of reinforced concrete*

Tapered Ends/Voids in Bent Cap (input 0 if not applicable)

Height of taper...................	0	*ft*
Length of taper...................	0	*ft*
Height of void....................	0	*ft*
Width of void.....................	0	*ft*
Length of void....................	0	*ft*

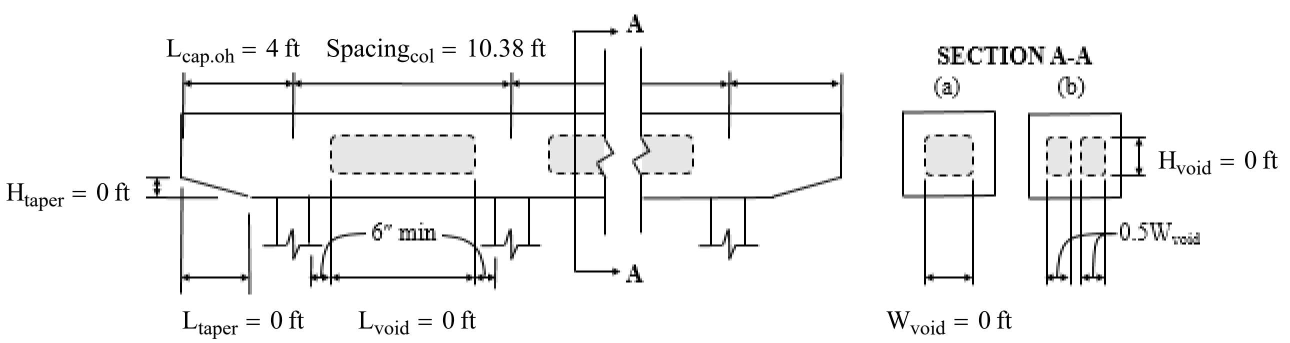
▶ Tapered ends/Voids inputs

Check tapered ends geometry

$Check_{taperGeo} := if(H_{taper} \le H_{cap} \wedge L_{taper} \le L_{cap.oh} - .5W_{col}, "OK", "NG") = "OK"$

Check voids geometry

$Check_{voidGeo} := if(H_{void} \le H_{cap} - 1.5ft \wedge W_{void} \le W_{cap} - 1.5ft \wedge L_{void} < Spacing_{col} - W_{col} - 12in, "OK", "NG") = "OK"$



2-Point Support/Pickup Locations (for Precast Bent Cap only, input 0 if not applicable)

Distance of support/pick-up location to cap end................	0	*ft*

Design Loads - Moments and Shears (Torques not considered)

Critical section for shear should be at face of support.

Conservatively take design negative moment at the CL of support; Except for bent caps built integrally with supports (full moment connection), design may be based on the moments at face of support.

Critical section for flexural design..	2	*1 - at center line of support* *2 - at face of support*

Reinforcement (Symmetrical to CL of Bent Cap)

A few recommendations on bar size and spacing are available to minimize problems during casting.

- *Use the same size and spacing of reinforcing for both the negative and positive moment regions. This minimizes construction errors where the top steel is mistakenly placed at the bottom or vice versa.*
- *If this arrangement is not possible, give preference to maintaining the same spacing between the top and bottom reinforcement. Same grid pattern allows grouted ducts placement and the concrete vibrator to be more effective in reaching the full depth of the cap, especially for multi-layer reinforcing.*

GFRP Material and Design Properties

C_E = Environmental reduction factor ... 0.7 *[AASHTO Table 2.4-1]*

E_f = Tensile modulus of elasticity..... 6500 *ksi* *[FDOT Specs 932]*

Nominal Area of GFRP reinforcing bars [FDOT Spec 932-3]

$$A(Bar) := \begin{cases} 0.049in^2 & \text{if } Bar = 2 \\ 0.11in^2 & \text{if } Bar = 3 \\ 0.20in^2 & \text{if } Bar = 4 \\ 0.31in^2 & \text{if } Bar = 5 \\ 0.44in^2 & \text{if } Bar = 6 \\ 0.60in^2 & \text{if } Bar = 7 \\ 0.79in^2 & \text{if } Bar = 8 \\ 1.00in^2 & \text{if } Bar = 9 \\ 1.27in^2 & \text{if } Bar = 10 \\ 0 & \text{otherwise} \end{cases}$$

Minimum Guaranteed Tensile Load of GFRP reinforcing bars [FDOT Spec 932-3]

$$P_T(Bar) := \begin{cases} 6.1kip & \text{if } Bar = 2 \\ 13.2kip & \text{if } Bar = 3 \\ 21.6kip & \text{if } Bar = 4 \\ 29.1kip & \text{if } Bar = 5 \\ 40.9kip & \text{if } Bar = 6 \\ 54.1kip & \text{if } Bar = 7 \\ 66.8kip & \text{if } Bar = 8 \\ 82.0kip & \text{if } Bar = 9 \\ 98.2kip & \text{if } Bar = 10 \\ 0 & \text{otherwise} \end{cases}$$

Tensile Strength of GFRP reinforcing bars

$$f_{fu}(Bar) := \frac{P_T(Bar)}{A(Bar)}$$

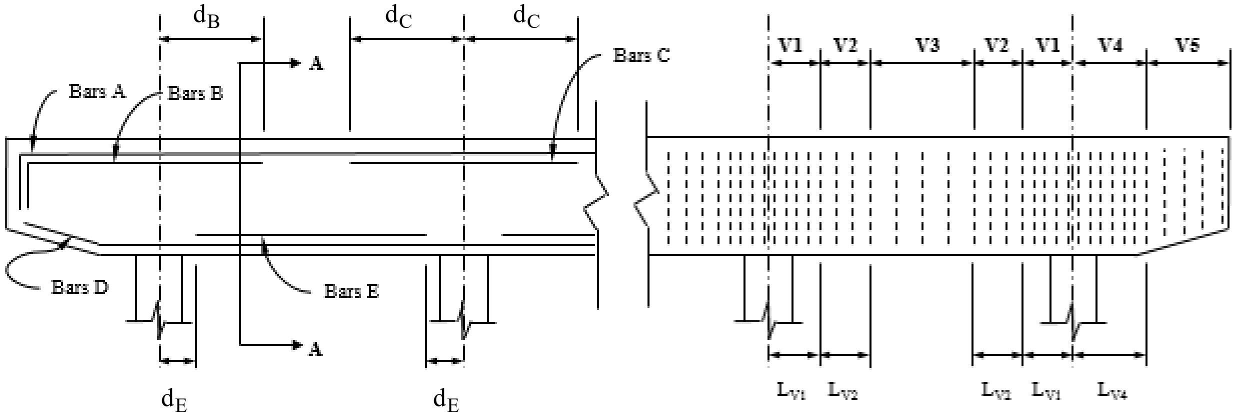
Diameter of GFRP reinforcing bars [FDOT Specs 932]

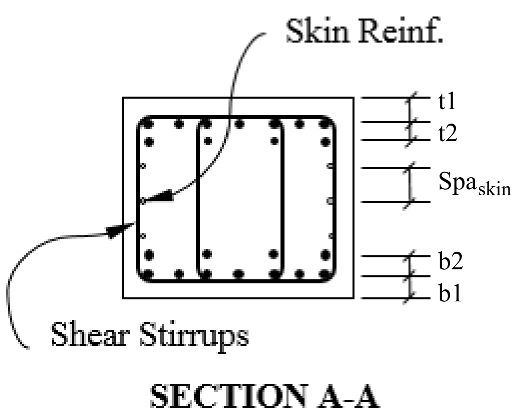
$$d(Bar) := \begin{cases} 1.128 \cdot in & \text{if } Bar = 9 \\ 1.27 \cdot in & \text{if } Bar = 10 \\ \frac{Bar}{8} \cdot in & \text{otherwise} \end{cases}$$

Bend radius of GFRP reinforcing bars [FDOT Developmental Standards D21310]

$$r(Bar) := \begin{cases} 1.5in & \text{if } Bar = 2 \\ 2.25in & \text{if } Bar = 3 \\ 2.25in & \text{if } Bar = 4 \\ 2.25in & \text{if } Bar = 5 \\ 2.25in & \text{if } Bar = 6 \\ 3in & \text{if } Bar = 7 \\ 3in & \text{if } Bar = 8 \\ 4.5in & \text{if } Bar = 9 \\ 5in & \text{if } Bar = 10 \\ 0 & \text{otherwise} \end{cases}$$

Flexural reinforcement





Flexural reinforcement can be placed up to 2 layers in current version.

Top Reinforcement (Negative Moment)

8 *Size of top reinforcing bars (A, B & C),* $Bar\#_{top}$

4 *Distance from c.g. of 1st layer bars to cap top face (in),* t1

0 *Distance from c.g. of 2nd layer bars to c.g. of 1st layer bars (in.),* t2

Bottom Reinforcement (Positive Moment)

8 *Size of bottom reinforcing bars (D & E),* $Bar\#_{bot}$

4 *Distance from c.g. of 1st layer bars to cap bottom face (in.),* b1

0 *Distance from c.g. of 2nd layer bars to c.g. of 1st layer bars (in.),* b2

Bars A: Continuous Top Reinforcement

Bars A placed in 1st Layer

8 *Number of bars,* $\#Bars_{A1}$

Bars A placed in 2nd Layer

0 *Number of bars,* $\#Bars_{A2}$

Bars B: Supplemental Top Reinforcement over Exterior Columns

15 *Length of Bars B beyond CL of Exterior Column (in.),* d_B

Development Length not considered in current version, input length of portion that is cosidered fully developed.

Bars B placed in 1st Layer

4 *Number of bars,* $\#Bars_{B1}$

Bars B placed in 2nd Layer

0 *Number of bars,* $\#Bars_{B2}$

Bars C: Supplemental Top Reinforcement Centered on CL of Interior Columns

15 *Length of Bars C beyond CL of Interior Column (in.),* d_C

Development Length not considered in current version, input length of portion that is cosidered fully developed.

Bars C placed in 1st Layer

4 *Number of bars,* $\#Bars_{C1}$

Bars C placed in 2nd Layer

0 *Number of bars,* $\#Bars_{C2}$

Bars D: Continuous Bottom Reinforcement

Bars D placed in 1st Layer

8 *Number of bars,* $\#Bars_{D1}$

Bars D placed in 2nd Layer

0 *Number of bars,* $\#Bars_{D2}$

Bars E: Supplemental Bottom Reinforcement Centered on Interior Spans

15 *Distance from CL of column to end of Bars E (in.),* d_E

Development Length not considered in current version, input distance to portion that is considered fully developed.

Bars E placed in 1st Layer

4 *Number of bars,* $\#Bars_{E1}$

Bars E placed in 2nd Layer

0 *Number of bars,* $\#Bars_{E2}$

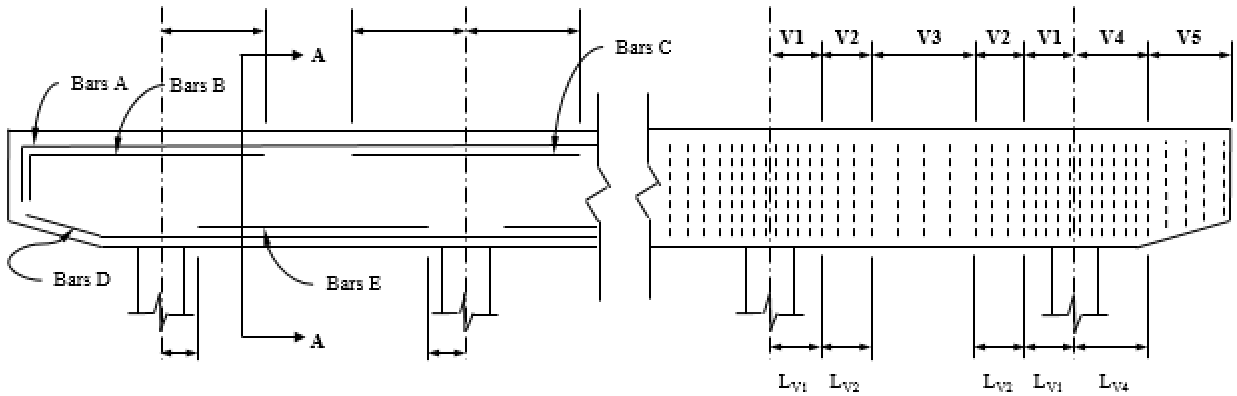
Spacing of Flexural Reinforcement

For crack control check, the bar spacing is calculated assuming a uniform distribution of bars on the 1st layer. Designer/user to calculate the actual spacing of bars if bars are not evenly distributed or bundled together. The maximum allowable spacing is plotted along the cap under the "Crack Control" section and the maximum allowable spacing at the most critical cap section is reported at the end of the program.

3 *Concrete cover on the two sides (in.)*

Shear Reinforcement

In order to maintain a level of flexibility for modeling and to allow for multiple spacings for shear reinforcement along *the cap, the spacing of the stirrups over the pile (non-critical fo shear) is assumed to be the same as the first zone beyond the face of the pile.*



Zone V1

4 *Size of stirrup bar,* $Bar\#_{V1}$

4 *No. of bar legs,* $\#Legs_{V1}$

4 *Spacing (in.),* Spa_{V1}

36 *Length of Zone V1 (in.),* L_{V1}

Zone V2

4 *Size of stirrup bar,* $Bar\#_{V2}$

4 *No. of bar legs,* $\#Legs_{V2}$

6 *Spacing (in.),* Spa_{V2}

36 *Length of Zone V2 (in.),* L_{V2}

Zone V3

4 *Size of stirrup bar,* $Bar\#_{V3}$

4 *No. of bar legs,* $\#Legs_{V3}$

12 *Spacing (in.),* Spa_{V3}

Zone V4 (Cap overhang)

4 *Size of stirrup bar,* $Bar\#_{V4}$

4 *No. of bar legs,* $\#Legs_{V4}$

12 *Spacing (in.),* Spa_{V4}

36 *Length of Zone V1 (in.),* L_{V4}

Zone V5 (Cap overhang)

4 *Size of stirrup bar,* $Bar\#_{V5}$

4 *No. of bar legs,* $\#Legs_{V5}$

12 *Spacing (in.),* Spa_{V5}

Skin Reinforcement

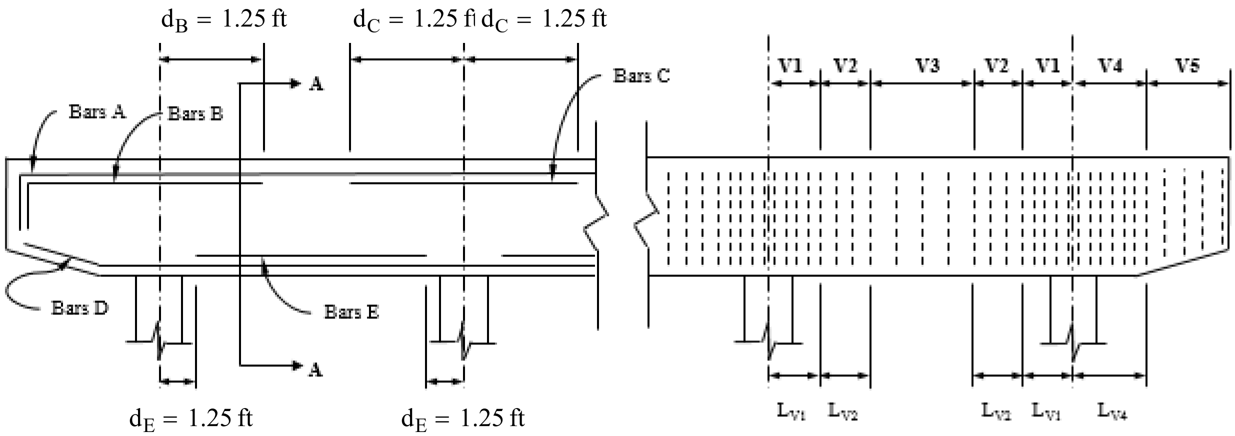
5 *Size of bar,* $Bar\#_{skin}$

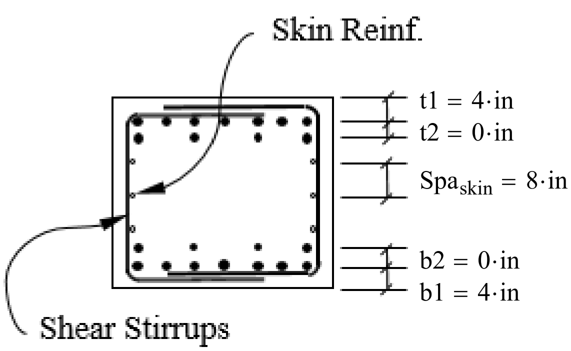
2 *Number of bars on each side face,* $\#Bars_{skin}$

8 *Spacing of skin reinforcement (in.),* Spa_{skin}

Calculate Worksheet

Reinforcement inputs





SECTION A-A

LRFD Moment, Shear, Min & Max Reinforcement, and Lifting Moment Checks

Design Sections

Cap Design Sections at increment := 0.5ft

$$X1 := \begin{array}{|l} X1_1 \leftarrow 0 \\ n \leftarrow \text{ceil}\left(\dfrac{L_{cap.oh} - 0.5W_{col}}{\text{increment}}\right) - 1 \\ \text{for } i \in 1..n \quad \text{if } n \geq 1 \\ \quad X1_{1+i} \leftarrow \text{increment}\cdot i \\ m \leftarrow \text{ceil}\left(\dfrac{\text{Spacing}_{col} - W_{col}}{\text{increment}}\right) - 1 \\ \text{for } j \in 1..\#Cols - 1 \\ \quad \text{for } k \in 1..m \\ \quad\quad X1_{n+1+m\cdot(j-1)+k} \leftarrow L_{cap.oh} + \text{Spacing}_{col}\cdot(j-1) + 0.5W_{col} + \text{increment}\cdot k \\ \text{for } i \in 1..n \quad \text{if } n \geq 1 \\ \quad X1_{n+1+m\cdot j+i} \leftarrow L_{cap.oh} + \text{Spacing}_{col}\cdot(\#Cols - 1) + 0.5W_{col} + \text{increment}\cdot i \\ X1_{2n+2+m\cdot j} \leftarrow L_{cap} \\ X1 \end{array}$$

Combine Cap Design Sections based on increments with Critical Sections based on demands

$$X_{design} := \text{stack}(X1, X)$$

$$X_{design} := \text{sort}(X_{design})$$

$$\#AllSections := \text{rows}(X_{design}) = 150$$

Function to convert design forces at critical sections to be at all design sections based on interpolation

$$\text{AllDesignSections}(V) := \begin{array}{|l} \text{for } i \in 1..\#AllSections \\ \quad \begin{array}{|l} VV_i \leftarrow 0 \\ s \leftarrow \text{rows}(X) \\ \text{for } j \in 1..s-1 \\ \quad \begin{array}{|l} VV_i \leftarrow V_j \text{ if } \left|X_{design_i} - X_j\right| \leq 0.0001\text{ft} \\ VV_i \leftarrow V_j + \dfrac{(V_{j+1} - V_j)}{X_{j+1} - X_j}\cdot\left(X_{design_i} - X_j\right) \text{ if } X_j < X_{design_i} < X_{j+1} \end{array} \\ VV_i \leftarrow V_s \text{ if } \left|X_{design_i} - X_s\right| \leq 0.0001\text{ft} \\ VV_i \leftarrow \dfrac{V_s}{L_{cap} - X_s}\cdot\left(L_{cap} - X_{design_i}\right) \text{ if } X_{design_i} > X_s \\ VV_i \leftarrow \dfrac{V_1}{X_1}\cdot X_{design_i} \text{ if } X_{design_i} < X_1 \end{array} \\ VV \end{array}$$

$maxV_u := AllDesignSections(maxV_u)$ *Max Shear - Strength I, III, & V*

$minV_u := AllDesignSections(minV_u)$ *Min Shear - Strength I, III, & V*

$maxM_u := AllDesignSections(maxM_u)$ *Max Positive Moment (+M) - Strength I, III, & V*

$minM_u := AllDesignSections(minM_u)$ *Max Negative Moment (-M) - Strength I, III, & V*

$maxM_{SerI} := AllDesignSections(maxM_{SerI})$ *Max Moment (+M) - Service I*

$minM_{SerI} := AllDesignSections(minM_{SerI})$ *Max Negative Moment (-M) - Service I*

$maxM_{SL} := AllDesignSections(maxM_{SL})$ *Max Positive Moment of Sustained Load - DL+0.2LL*

$minM_{SL} := AllDesignSections(minM_{SL})$ *Max Negative Moment of Sustained Load - DL+0.2LL*

$maxM_{FatI} := AllDesignSections(maxM_{FatI})$ *Max Moment (+M) - DL+Fatigue I LL*

$minM_{FatI} := AllDesignSections(minM_{FatI})$ *Max Negative Moment (-M) - DL+Fatigue I LL*

Depth of Cap

$$d_{cap} := \left| \begin{array}{l} \text{for } i \in 1..\#AllSections \\ \quad \left| \begin{array}{l} d_{cap_i} \leftarrow H_{cap} - H_{taper} + X_{design_i} \cdot \dfrac{H_{taper}}{L_{taper}} \quad \text{if } X_{design_i} \le L_{taper} \\ d_{cap_i} \leftarrow H_{cap} - \left(X_{design_i} - L_{cap} + L_{taper}\right) \cdot \dfrac{H_{taper}}{L_{taper}} \quad \text{if } X_{design_i} \ge L_{cap} - L_{taper} \\ d_{cap_i} \leftarrow H_{cap} \quad \text{otherwise} \end{array} \right. \\ d_{cap} \end{array} \right.$$

Width of the web

$$b_{w.cap} := \left| \begin{array}{l} \text{for } i \in 1..\#AllSections \\ \quad \left| \begin{array}{l} b_{w.cap_i} \leftarrow W_{cap} \\ \text{for } j \in 1..\#Cols - 1 \\ \quad \left| \begin{array}{l} Void_{start} \leftarrow L_{cap.oh} + (j-1) \cdot Spacing_{col} + 0.5\left(Spacing_{col} - L_{void}\right) \\ Void_{end} \leftarrow L_{cap.oh} + j \cdot Spacing_{col} - 0.5\left(Spacing_{col} - L_{void}\right) \\ b_{w.cap_i} \leftarrow W_{cap} - W_{void} \quad \text{if } Void_{start} < X_{design_i} < Void_{end} \end{array} \right. \end{array} \right. \\ b_{w.cap} \end{array} \right.$$

Flexural Design

Flexural Resistance [AASHTO BDS for GFRP 2.6]

With multiple layers of reinforcement, because FRP materials have no plastic strain region, the stress in each *reinforcement layer will vary depending on its distance from the neutral axis. The analysis of the flexural capacity should be based on a strain compatibility approach.*

For failure mode of concrete crushing

$$C = \alpha_1 \cdot f_{c.cap} \cdot W_{cap} \cdot \beta_1 \cdot c$$

$$T_1 = A_{f1} \cdot E_f \cdot \frac{\varepsilon_{cu}}{c} \cdot \left(d_{cap} - b1 - c\right)$$

$$T_2 = A_{f2} \cdot E_f \cdot \frac{\varepsilon_{cu}}{c} \cdot \left(d_{cap} - b1 - b2 - c\right)$$

For failure mode of FRP rupture

$$C = \alpha_1 \cdot f_{c.cap} \cdot W_{cap} \cdot \beta_1 \cdot c_b$$

$$T_1 = A_{f1} \cdot f_{fu}$$

$$T_2 = A_{f2} \cdot f_{fu} \cdot \frac{\left(d_{cap} - b1 - b2 - c_b\right)}{\left(d_{cap} - b1 - c_b\right)}$$

$$f_{c.cap} = 5.5 \cdot ksi$$

$$\alpha_1 := \min\left[\max\left[0.85 - 0.02 \cdot \left(\frac{f_{c.cap} - 10ksi}{ksi}\right), 0.75\right], 0.85\right] = 0.85$$ *Stress block factor*

$$\beta_1 := \min\left[\max\left[0.85 - 0.05 \cdot \left(\frac{f_{c.cap} - 4ksi}{ksi}\right), 0.65\right], 0.85\right] = 0.78$$ *Stress block factor*

$$\varepsilon_{cu} := 0.003$$ *Ultimate strain in concrete*

Design tensile strength of GFRP

$$f_{fd.pos} := C_E \cdot f_{fu}\left(Bar\#_{bot}\right) = 59.19 \cdot ksi \qquad f_{fd.neg} := C_E \cdot f_{fu}\left(Bar\#_{top}\right) = 59.19 \cdot ksi$$

Design tensile strain at rupture of GFRP

$$\varepsilon_{fd.pos} := \frac{f_{fd.pos}}{E_f} = 0.009 \qquad \varepsilon_{fd.neg} := \frac{f_{fd.neg}}{E_f} = 0.009$$

Distance from extreme compression fiber to neutral axis at balanced strain condition

$$c_{b.pos} := \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd.pos}}\left(d_{cap} - b1\right) \qquad c_{b.neg} := \frac{\varepsilon_{cu}}{\varepsilon_{cu} + \varepsilon_{fd.neg}}\left(d_{cap} - t1\right)$$

Area of GFRP bars

```
A_f.bot := | for i ∈ 1 .. #AllSections
           |   | A_f1.bot_i ← A(Bar#_bot)·#Bars_D1
           |   | A_f2.bot_i ← A(Bar#_bot)·#Bars_D2
           |   | for j ∈ 1 .. #Cols − 1
           |   |   | BarEstart ← L_cap.oh + (j − 1)·Spacing_col + d_E
           |   |   | BarEend ← L_cap.oh + j·Spacing_col − d_E
           |   |   | if BarEstart ≤ X_design_i ≤ BarEend
           |   |   |   | A_f1.bot_i ← A(Bar#_bot)·#Bars_D1 + A(Bar#_bot)·#Bars_E1
           |   |   |   | A_f2.bot_i ← A(Bar#_bot)·#Bars_D2 + A(Bar#_bot)·#Bars_E2
           | (A_f1.bot  A_f2.bot)

A_f.top := | for i ∈ 1 .. #AllSections
           |   | A_f1.top_i ← A(Bar#_top)·#Bars_A1
           |   | A_f2.top_i ← A(Bar#_top)·#Bars_A2
           |   | for j ∈ 1 .. #Cols
           |   |   | if j = 1
           |   |   |   | BarBstart ← 0
           |   |   |   | BarBend ← L_cap.oh + d_B
           |   |   |   | if BarBstart ≤ X_design_i ≤ BarBend
           |   |   |   |   | A_f1.top_i ← A(Bar#_top)·#Bars_A1 + A(Bar#_top)·#Bars_B1
           |   |   |   |   | A_f2.top_i ← A(Bar#_top)·#Bars_A2 + A(Bar#_top)·#Bars_B2
           |   |   | if j = #Cols
           |   |   |   | BarBstart ← L_cap − L_cap.oh − d_B
           |   |   |   | BarBend ← L_cap
           |   |   |   | if BarBstart ≤ X_design_i ≤ BarBend
           |   |   |   |   | A_f1.top_i ← A(Bar#_top)·#Bars_A1 + A(Bar#_top)·#Bars_B1
           |   |   |   |   | A_f2.top_i ← A(Bar#_top)·#Bars_A2 + A(Bar#_top)·#Bars_B2
           |   |   | otherwise
           |   |   |   | BarCstart ← L_cap.oh + (j − 1)·Spacing_col − d_C
           |   |   |   | BarCend ← L_cap.oh + (j − 1)·Spacing_col + d_C
           |   |   |   | if BarCstart ≤ X_design_i ≤ BarCend
           |   |   |   |   | A_f1.top_i ← A(Bar#_top)·#Bars_A1 + A(Bar#_top)·#Bars_C1
           |   |   |   |   | A_f2.top_i ← A(Bar#_top)·#Bars_A2 + A(Bar#_top)·#Bars_C2
           | (A_f1.top  A_f2.top)
```

Area of GFRP bars provided on 1st layer

$A_{f1.bot} := A_{f.bot_{1,1}}$

$A_{f1.top} := A_{f.top_{1,1}}$

Area of GFRP bars provided on 2nd layer

$A_{f2.bot} := A_{f.bot_{1,2}}$

$A_{f2.top} := A_{f.top_{1,2}}$

Function to solve for distance of extreme compression fiber to the neutral axis

Guess value $c := 5in$

Given

$$\alpha_1 \cdot f_{c.cap} \cdot W_{cap} \cdot \beta_1 \cdot c = A_{f1} \cdot \left[E_f \cdot \frac{\varepsilon_{cu}}{c} \cdot (d - b1 - c)\right] + A_{f2} \cdot \left[E_f \cdot \frac{\varepsilon_{cu}}{c} \cdot (d - b1 - b2 - c)\right]$$

for compression controlled case

$func(A_{f1}, A_{f2}, d, b1, b2) := Find(c)$

$$c_{pos} := \left| \begin{array}{l} \text{for } i \in 1..\#AllSections \\ \quad c_{pos_i} \leftarrow func(A_{f1.bot_i}, A_{f2.bot_i}, d_{cap_i}, b1, b2) \\ \quad c_{pos_i} \leftarrow c_{b.pos_i} \text{ if } c_{pos_i} < c_{b.pos_i} \\ c_{pos} \end{array} \right.$$

$$c_{neg} := \left| \begin{array}{l} \text{for } i \in 1..\#AllSections \\ \quad c_{neg_i} \leftarrow func(A_{f1.top_i}, A_{f2.top_i}, d_{cap_i}, t1, t2) \\ \quad c_{neg_i} \leftarrow c_{b.neg_i} \text{ if } c_{neg_i} < c_{b.neg_i} \\ c_{neg} \end{array} \right.$$

$i := 1..\#AllSections$

$$T_{1.pos_i} := A_{f1.bot_i} \cdot \min\left[E_f \cdot \frac{\varepsilon_{cu}}{c_{pos_i}} \cdot (d_{cap_i} - b1 - c_{pos_i}), f_{fd.pos}\right]$$

$$T_{2.pos_i} := A_{f2.bot_i} \cdot \min\left[E_f \cdot \frac{\varepsilon_{cu}}{c_{pos_i}} \cdot (d_{cap_i} - b1 - b2 - c_{pos_i}), f_{fd.pos} \cdot \frac{(d_{cap_i} - b1 - b2 - c_{pos_i})}{(d_{cap_i} - b1 - c_{pos_i})}\right]$$

$$M_{n.pos_i} := T_{1.pos_i} \cdot (d_{cap_i} - b1 - 0.5\beta_1 \cdot c_{pos_i}) + T_{2.pos_i} \cdot (d_{cap_i} - b1 - b2 - 0.5\beta_1 \cdot c_{pos_i})$$

$$T_{1.neg_i} := A_{f1.top_i} \cdot \min\left[E_f \cdot \frac{\varepsilon_{cu}}{c_{neg_i}} \cdot (d_{cap_i} - t1 - c_{neg_i}), f_{fd.neg}\right]$$

$$T_{2.neg_i} := A_{f2.top_i} \cdot \min\left[E_f \cdot \frac{\varepsilon_{cu}}{c_{neg_i}} \cdot (d_{cap_i} - t1 - t2 - c_{neg_i}), f_{fd.neg} \cdot \frac{(d_{cap_i} - t1 - t2 - c_{neg_i})}{(d_{cap_i} - t1 - c_{neg_i})}\right]$$

$$M_{n.neg_i} := T_{1.neg_i} \cdot (d_{cap_i} - t1 - 0.5\beta_1 \cdot c_{neg_i}) + T_{2.neg_i} \cdot (d_{cap_i} - t1 - t2 - 0.5\beta_1 \cdot c_{neg_i})$$

Tensile strain in extreme tension GFRP at nominal resistance

$$\varepsilon_{ft.pos_i} := \frac{d_{cap_i} - b1 - c_{pos_i}}{c_{pos_i}} \cdot \varepsilon_{cu}$$

$$\varepsilon_{ft.neg_i} := \frac{d_{cap_i} - t1 - c_{neg_i}}{c_{neg_i}} \cdot \varepsilon_{cu}$$

Strength reduction factor for flexure

$$f\phi(\varepsilon_{ft}, \varepsilon_{fd}) := \begin{cases} 0.55 & \text{if } \varepsilon_{ft} \ge \varepsilon_{fd} \\ \left(1.55 - \dfrac{\varepsilon_{ft}}{\varepsilon_{fd}}\right) & \text{if } 0.8\cdot\varepsilon_{fd} < \varepsilon_{ft} < \varepsilon_{fd} \\ 0.75 & \text{otherwise} \end{cases}$$

$\phi_{pos_i} := f\phi(\varepsilon_{ft.pos_i}, \varepsilon_{fd.pos})$

$\phi_{neg_i} := f\phi(\varepsilon_{ft.neg_i}, \varepsilon_{fd.neg})$

Factored flexural resistance

$M_{r.pos_i} := \phi_{pos_i}\cdot M_{n.pos_i}$

$M_{r.neg_i} := \phi_{neg_i}\cdot M_{n.neg_i}$

Check $M_r \ge M_u$

$$Check_{M.r.pos} := \left| \begin{array}{l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \ \text{ if } M_{r.pos_i} < maxM_{u_i} \\ Check \end{array} \right.$$

$$Check_{M.r.neg} := \left| \begin{array}{l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \ \text{ if } M_{r.neg_i} < -minM_{u_i} \\ Check \end{array} \right.$$

$Check_{M.r.pos} = \text{"OK"}$

$Check_{M.r.neg} = \text{"OK"}$

Demand to capacity ratio

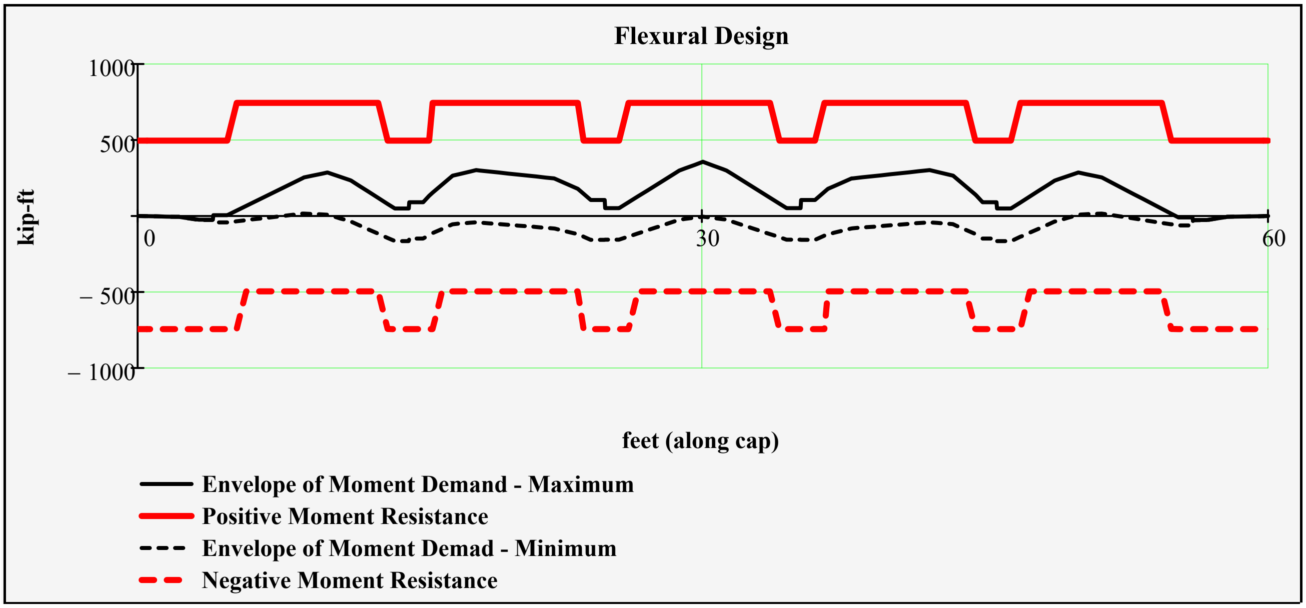
$$DCR_{M.pos_i} := \frac{maxM_{u_i}}{M_{r.pos_i}}$$

$$DCR_{M.neg_i} := \frac{-minM_{u_i}}{M_{r.neg_i}}$$

Maximum demand to capacity ratio

$\max(DCR_{M.pos}) = 0.48$

$\max(DCR_{M.neg}) = 0.24$



Cracking Moment

$f_r := 7.5 \cdot \sqrt{f_{c.cap} \cdot psi} = 556 \cdot psi$ *Modulus of Rupture*

Section modulus of cap

$$S_{cap} := \left| \begin{array}{l} \text{for } i \in 1 .. \#AllSections \\ \quad \left| \begin{array}{l} S_{cap_i} \leftarrow \frac{1}{6} W_{cap} \cdot \left(d_{cap_i}\right)^2 \\ \text{for } j \in 1 .. \#Cols - 1 \\ \quad \left| \begin{array}{l} Void_{start} \leftarrow L_{cap.oh} + (j-1) \cdot Spacing_{col} + 0.5\left(Spacing_{col} - L_{void}\right) \\ Void_{end} \leftarrow L_{cap.oh} + j \cdot Spacing_{col} - 0.5\left(Spacing_{col} - L_{void}\right) \\ S_{cap_i} \leftarrow \frac{1}{6} W_{cap} \cdot \left(d_{cap_i}\right)^2 - \frac{1}{6} W_{void} \cdot H_{void}{}^3 \cdot \frac{1}{d_{cap_i}} \text{ if } Void_{start} < X_{design_i} < Void_{end} \end{array} \right. \end{array} \right. \\ S_{cap} \end{array} \right.$$

$M_{cr} := f_r \cdot S_{cap}$ *Cracking moment*

$\max\left(M_{cr}\right) = 480.57 \cdot kip \cdot ft$ *Maximum cracking moment along the cap*

Minimum GFRP Reinforcement [AASHTO BDS for GFRP 2.6.3.3]

Required flexural resistance for minimum reinforcement

$M_{minAf.bot_i} := \min\left(1.33 \cdot maxM_{u_i}, 1.6 \cdot M_{cr_i}\right)$

$M_{minAf.top_i} := \min\left(-1.33 \cdot minM_{u_i}, 1.6 \cdot M_{cr_i}\right)$

$\max(M_{minAf.bot}) = 473.94 \cdot kip \cdot ft$

$\max(M_{minAf.top}) = 219.27 \cdot kip \cdot ft$

Check minimum GFRP reinforcement

$Check_{minAf.bot} :=$ | Check ← "OK"
for $i \in 1..\#AllSections$
  Check ← "NG" if $M_{r.pos_i} < M_{minAf.bot_i}$
Check

$Check_{minAf.top} :=$ | Check ← "OK"
for $i \in 1..\#AllSections$
  Check ← "NG" if $M_{r.neg_i} < M_{minAf}$
Check

$Check_{minAf.bot} =$ "OK"

$Check_{minAf.top} =$ "OK"

Crack Control [AASHTO BDS for GFRP 2.6.7]

$CrackLimit := 0.028in$ — *limiting crack width*

$C_b := 0.83$ — *the bond reduction factor*

$\phi_{Mcr} := 0.25$ — *cracking moment coefficient*

$n := \frac{E_f}{E_{cap}} = 1.47$ — *Modular ratio*

Function to calculate reinforcing stress under service loads

Note: In lieu of the requirements of [AASHTO BDS for GFRP 2.6.7] or [ACI 440.1.8.3.1], the stress in the tension reinforcing should be derived based on gross section properties when the moment due to Service I loading conditions is less than 1/4 of the cracking moment. The cracking moment is calculated based on the modulus of rupture, as specified in Article 5.4.2.6 of the AASHTO LRFD Bridge Design Specifications.

Commentary: Given that this is a serviceability check, the probability of cracking is extremely remote when the service loads is less than a 1/4 of the cracking load.

$$ff_f(M, d_f, d_{cap}, S_{cap}, k, I_{cr}, M_{cr}) := \begin{cases} 0 & \text{if } M \le 0 \\ n \cdot \dfrac{M}{S_{cap}} \cdot \dfrac{d_f - 0.5d_{cap}}{0.5d_{cap}} & \text{if } 0 < M \le \phi_{Mcr} M_{cr} \\ \dfrac{n \cdot d_f(1-k)}{I_{cr}} \cdot M & \text{otherwise} \end{cases}$$

if moment is negative, stress is set to 0; if moment is less than 1/4 of the cracking moment, stress is calculated based on uncracked section.

For GFRP reinforcement stress under service load, lump sum mutiple layers of reinforcement at their centroid of gravity

$$d_{f.pos_i} := \frac{A_{f1.bot_i} \cdot (d_{cap_i} - b1) + A_{f2.bot_i} \cdot (d_{cap_i} - b1 - b2)}{A_{f1.bot_i} + A_{f2.bot_i}}$$

$$d_{f.neg_i} := \frac{A_{f1.top_i} \cdot (d_{cap_i} - t1) + A_{f2.top_i} \cdot (d_{cap_i} - t1 - t2)}{A_{f1.top_i} + A_{f2.top_i}}$$

Reinforcement ratio

$$\rho_{pos_i} := \frac{A_{f1.bot_i} + A_{f2.bot_i}}{W_{cap} \cdot d_{f.pos_i}}$$

$$\rho_{neg_i} := \frac{A_{f1.top_i} + A_{f2.top_i}}{W_{cap} \cdot d_{f.neg_i}}$$

$$k_{pos_i} := \sqrt{2 \cdot \rho_{pos_i} \cdot n + (\rho_{pos_i} \cdot n)^2} - \rho_{pos_i} \cdot n$$

$$k_{neg_i} := \sqrt{2 \cdot \rho_{neg_i} \cdot n + (\rho_{neg_i} \cdot n)^2} - \rho_{neg_i} \cdot n$$

The cracked moment of inertia

$$I_{cr.pos_i} := \frac{1}{3} W_{cap} \cdot (d_{f.pos_i})^3 (k_{pos_i})^3 + n \cdot (A_{f1.bot_i} + A_{f2.bot_i}) \cdot (d_{f.pos_i})^2 (1 - k_{pos_i})^2$$

$$I_{cr.neg_i} := \frac{1}{3} W_{cap} \cdot (d_{f.neg_i})^3 (k_{neg_i})^3 + n \cdot (A_{f1.top_i} + A_{f2.top_i}) \cdot (d_{f.neg_i})^2 (1 - k_{neg_i})^2$$

Stress in the bottom reinforcing due to Service I limit state moment

$$f_{f.SerI.bot_i} := ff_f(maxM_{SerI_i}, d_{f.pos_i}, d_{cap_i}, S_{cap_i}, k_{pos_i}, I_{cr.pos_i}, M_{cr_i})$$

$$\max(f_{f.SerI.bot}) = 8.96 \cdot ksi$$

Stress in the top reinforcing due to Service I limit state moment

$f_{f.SerI.top_i} := ff_f\left(-minM_{SerI_i}, d_{f.neg_i}, d_{cap_i}, S_{cap_i}, k_{neg_i}, I_{cr.neg_i}, M_{cr_i}\right)$

$max\left(f_{f.SerI.top}\right) = 0.15 \cdot ksi$

Distance from extreme tension fiber to center of closest bar

$d_{c.bot} := b1 = 4 \cdot in$

$d_{c.top} := t1 = 4 \cdot in$

Ratio of distance between n.a. and tension face to distance between n.a. and centroid of reinforcement

$$\zeta_{pos_i} := \frac{d_{cap_i} - k_{pos_i} \cdot d_{f.pos_i}}{d_{f.pos_i} - k_{pos_i} \cdot d_{f.pos_i}}$$

$$\zeta_{neg_i} := \frac{d_{cap_i} - k_{neg_i} \cdot d_{f.neg_i}}{d_{f.neg_i} - k_{neg_i} \cdot d_{f.neg_i}}$$

Limit on d.c

$$d_{c.bot.max_i} := if\left(f_{f.SerI.bot_i} = 0, 100in, \frac{C_b \cdot E_f \cdot CrackLimit}{2 \cdot f_{f.SerI.bot_i} \cdot \zeta_{pos_i}}\right)$$

$min\left(d_{c.bot.max}\right) = 7.37 \cdot in$

$$d_{c.top.max_i} := if\left(f_{f.SerI.top_i} = 0, 100in, \frac{C_b \cdot E_f \cdot CrackLimit}{2 \cdot f_{f.SerI.top_i} \cdot \zeta_{neg_i}}\right)$$

$min\left(d_{c.top.max}\right) = 100 \cdot in$

$Check_{dc.bot} := if\left(d_{c.bot} < min\left(d_{c.bot.max}\right), \text{"OK"}, \text{"NG"}\right)$

$Check_{dc.bot} = \text{"OK"}$

$Check_{dc.top} := if\left(d_{c.top} < min\left(d_{c.top.max}\right), \text{"OK"}, \text{"NG"}\right)$

$Check_{dc.top} = \text{"OK"}$

clear cover

$c_{c.bot} := d_{c.bot} - 0.5 \cdot d\left(Bar\#_{bot}\right) = 3.5 \cdot in$

$c_{c.top} := d_{c.top} - 0.5 \cdot d\left(Bar\#_{top}\right) = 3.5 \cdot in$

$$s_{max.bot_i} := min\left[1.15 \cdot \frac{C_b \cdot E_f \cdot CrackLimit}{\left(f_{f.SerI.bot_i} + 0.0001ksi\right)} - 2.5c_{c.bot}, 0.92 \cdot \frac{C_b \cdot E_f \cdot CrackLimit}{\left(f_{f.SerI.bot_i} + 0.0001ksi\right)}\right]$$

maximum spacing for crack control

$min\left(s_{max.bot}\right) = 10.63 \cdot in$

$$s_{max.top_i} := min\left[1.15 \cdot \frac{C_b \cdot E_f \cdot CrackLimit}{\left(f_{f.SerI.top_i} + 0.0001ksi\right)} - 2.5c_{c.top}, 0.92 \cdot \frac{C_b \cdot E_f \cdot CrackLimit}{\left(f_{f.SerI.top_i} + 0.0001ksi\right)}\right]$$

maximum spacing for crack control

$min\left(s_{max.top}\right) = 941.82 \cdot in$

For crack control check here, the bar spacing is calculated assuming a uniform distribution of bars on the 1st layer. Designer to calculate the actual spacing if bars are not evenly distributed or bundled together.

$StirrupSize := max\left(Bar\#_{V1}, Bar\#_{V2}, Bar\#_{V3}, Bar\#_{V4}, Bar\#_{V5}\right) = 4$

$$Spa_{bar.bot} := \begin{array}{l} \text{for } i \in 1..\#AllSections \\ \quad Spa_{bar.bot_i} \leftarrow \dfrac{W_{cap} - 2\cdot SideCover - 2\cdot d(StirrupSize) - d(Bar\#_{bot})}{\#Bars_{D1} - 1} \\ \quad \text{for } j \in 1..\#Cols - 1 \\ \quad\quad BarEstart \leftarrow L_{cap.oh} + (j-1)\cdot Spacing_{col} + d_E \\ \quad\quad BarEend \leftarrow L_{cap.oh} + j\cdot Spacing_{col} - d_E \\ \quad\quad Spa_{bar.bot_i} \leftarrow \dfrac{W_{cap} - 2\cdot SideCover - 2\cdot d(StirrupSize) - d(Bar\#_{bot})}{\#Bars_{D1} + \#Bars_{E1} - 1} \text{ if } BarEstart \le X_{design_i} \le BarEend \\ Spa_{bar.bot} \end{array}$$

$$Spa_{bar.top} := \begin{array}{l} \text{for } i \in 1..\#AllSections \\ \quad Spa_{bar.top_i} \leftarrow \dfrac{W_{cap} - 2\cdot SideCover - 2\cdot d(StirrupSize) - d(Bar\#_{top})}{\#Bars_{A1} - 1} \\ \quad \text{for } j \in 1..\#Cols \\ \quad\quad \text{if } j = 1 \\ \quad\quad\quad BarBstart \leftarrow 0 \\ \quad\quad\quad BarBend \leftarrow L_{cap.oh} + d_B \\ \quad\quad\quad Spa_{bar.top_i} \leftarrow \dfrac{W_{cap} - 2\cdot SideCover - 2\cdot d(StirrupSize) - d(Bar\#_{top})}{\#Bars_{A1} + \#Bars_{B1} - 1} \text{ if } BarBstart \le X_{design_i} \le BarE \\ \quad\quad \text{if } j = \#Cols \\ \quad\quad\quad BarBstart \leftarrow L_{cap} - L_{cap.oh} - d_B \\ \quad\quad\quad BarBend \leftarrow L_{cap} \\ \quad\quad\quad Spa_{bar.top_i} \leftarrow \dfrac{W_{cap} - 2\cdot SideCover - 2\cdot d(StirrupSize) - d(Bar\#_{top})}{\#Bars_{A1} + \#Bars_{B1} - 1} \text{ if } BarBstart \le X_{design_i} \le BarE \\ \quad\quad \text{otherwise} \\ \quad\quad\quad BarCstart \leftarrow L_{cap.oh} + (j-1)\cdot Spacing_{col} - d_C \\ \quad\quad\quad BarCend \leftarrow L_{cap.oh} + (j-1)\cdot Spacing_{col} + d_C \\ \quad\quad\quad Spa_{bar.top_i} \leftarrow \dfrac{W_{cap} - 2\cdot SideCover - 2\cdot d(StirrupSize) - d(Bar\#_{top})}{\#Bars_{A1} + \#Bars_{C1} - 1} \text{ if } BarCstart \le X_{design_i} \le BarC \\ Spa_{bar.top} \end{array}$$

$$Check_{smax.bot} := \begin{array}{l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \text{ if } Spa_{bar.bot_i} > s_{max.bot_i} \\ Check \end{array}$$

$$Check_{smax.top} := \begin{array}{l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \text{ if } Spa_{bar.top_i} > s_{ma} \\ Check \end{array}$$

$Check_{smax.bot} = \text{"OK"}$

$Check_{smax.top} = \text{"OK"}$

$$Check_{crack.control.bot} := \begin{cases} \text{"OK"} \\ \text{"NG, bar spacing exceeds maximum"} & \text{if } Check_{smax.bot} = \text{"NG"} \\ \text{"NG, additional reinforcement required"} & \text{if } Check_{dc.bot} = \text{"NG"} \end{cases}$$

$Check_{crack.control.bot} = \text{"OK"}$

$$Check_{crack.control.top} := \begin{cases} \text{"OK"} \\ \text{"NG, bar spacing exceeds maximum"} & \text{if } Check_{smax.top} = \text{"NG"} \\ \text{"NG, additional reinforcement required"} & \text{if } Check_{dc.top} = \text{"NG"} \end{cases}$$

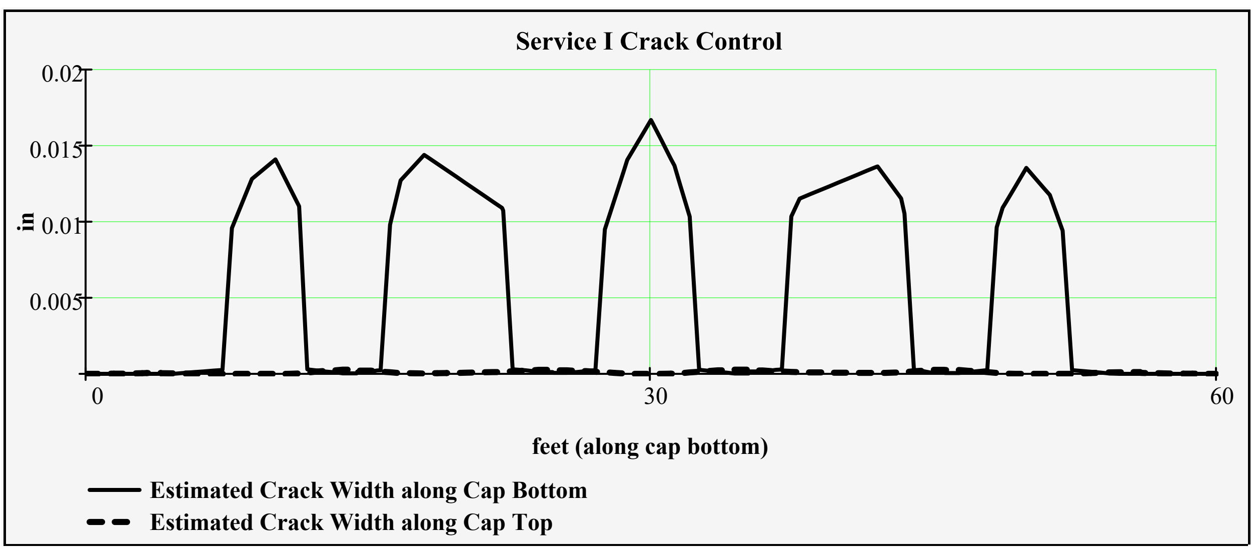
$Check_{crack.control.top} = \text{"OK"}$

Estimated Service I crack width

$$CrackW_{SerI.bot_i} := 2\frac{f_{f.SerI.bot_i}}{C_b \cdot E_f} \cdot \zeta_{pos_i} \cdot \sqrt{d_{c.bot}^{\,2} + \left(\frac{Spa_{bar.bot_i}}{2}\right)^2} \qquad CrackW_{SerI.top_i} := 2\frac{f_{f.SerI.top_i}}{C_b \cdot E_f} \cdot \zeta_{neg_i} \cdot \sqrt{d_{c.top}^{\,2} + \left(\frac{Spa_{bar.top_i}}{2}\right)^2}$$

Maximum Service I crack width

$\max(CrackW_{SerI.bot}) = 0.017 \cdot in$ $\qquad$ $\max(CrackW_{SerI.top}) = 0.000 \cdot in$



Creep Rupture Limit State [AASHTO BDS for GFRP 2.5.3]

To avoid creep ruptureof the FRP reinforcement under sustained stresses or failure due to cyclic stresses and fatigue of the FRP reinforcement, the stress levels in the FRP under these stress conditions should be limited.

$C_c := 0.30$ *Creep rupture reduction factor*

Creep rupture stress limit of GFRP

$CreepLimit_{pos_i} := C_c \cdot f_{fd.pos}$

$\max(CreepLimit_{pos}) = 17.76 \cdot ksi$

$CreepLimit_{neg_i} := C_c \cdot f_{fd.neg}$

$\max(CreepLimit_{neg}) = 17.76 \cdot ksi$

Stress in the bottom reinforcing due to sustained load (DL+0.2LL)

$f_{f.SL.bot_i} := ff_f\left(maxM_{SL_i}, d_{f.pos_i}, d_{cap_i}, S_{cap_i}, k_{pos_i}, I_{cr.pos_i}, M_{cr_i}\right)$

$\max(f_{f.SL.bot}) = 0.11 \cdot ksi$

Stress in the top reinforcing due to sustained load (DL+0.2LL)

$f_{f.SL.top_i} := ff_f\left(-minM_{SL_i}, d_{f.neg_i}, d_{cap_i}, S_{cap_i}, k_{neg_i}, I_{cr.neg_i}, M_{cr_i}\right)$

$\max(f_{f.SL.top}) = 0.08 \cdot ksi$

$Check_{creep.bot} :=$
```
Check ← "OK"
for  i ∈ 1 .. #AllSections
  Check ← "NG"  if  f_f.SL.bot_i > C_c·f_fd.pos
Check
```

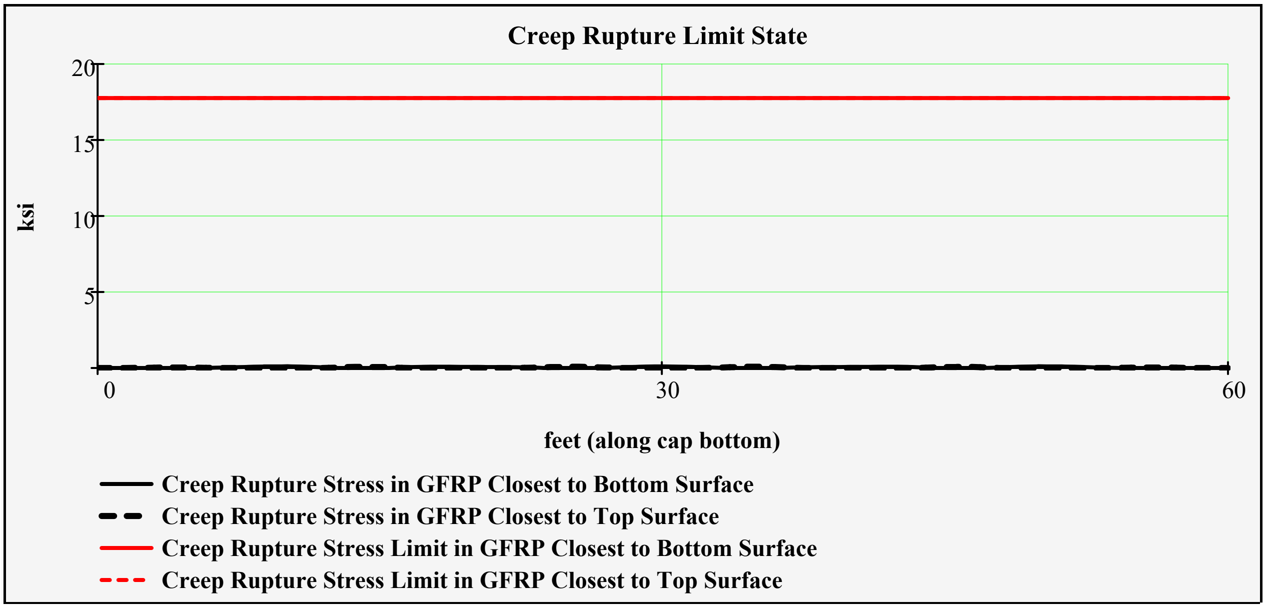
$Check_{creep.bot} = \text{"OK"}$

$Check_{creep.top} :=$
```
Check ← "OK"
for  i ∈ 1 .. #AllSections
  Check ← "NG"  if  f_f.SL.top_i > C_c·f_fd
Check
```

$Check_{creep.top} = \text{"OK"}$

$Check_{creep} :=$
```
"OK"
"NG, sustained tensile stress in bottom reinforcing exceeds limit"  if  (Check_creep.bot = "NG")·(Check_creep.top
"NG, sustained tensile stress in top reinforcing exceeds limit"  if  (Check_creep.bot = "OK")·(Check_creep.top = "I
"NG, sustained tensile stress in top and bottom reinforcing exceeds limit"  if  (Check_creep.bot = "NG")·(Check
```

$Check_{creep} = \text{"OK"}$



Fatigue Limit State [AASHTO BDS for GFRP 2.5.4]

$C_f := 0.25$ *Fatigue rupture reduction factor*

Fatigue rupture stress limit of GFRP

$FatigueLimit_{pos_i} := C_f \cdot f_{fd.pos}$ $FatigueLimit_{neg_i} := C_f \cdot f_{fd.neg}$

$\max(FatigueLimit_{pos}) = 14.8 \cdot ksi$ $\max(FatigueLimit_{neg}) = 14.8 \cdot ksi$

Stress in the bottom reinforcing due to sustained load (DL+1.5FatigueLL)

$f_{f.Fat.bot_i} := ff_f\left(maxM_{FatI_i}, d_{f.pos_i}, d_{cap_i}, S_{cap_i}, k_{pos_i}, I_{cr.pos_i}, M_{cr_i}\right)$

$\max(f_{f.Fat.bot}) = 5.94 \cdot ksi$

Stress in the top reinforcing due to sustained load (DL+1.5FatigueLL)

$f_{f.Fat.top_i} := ff_f\left(-minM_{FatI_i}, d_{f.neg_i}, d_{cap_i}, S_{cap_i}, k_{neg_i}, I_{cr.neg_i}, M_{cr_i}\right)$

$\max(f_{f.Fat.top}) = 0.11 \cdot ksi$

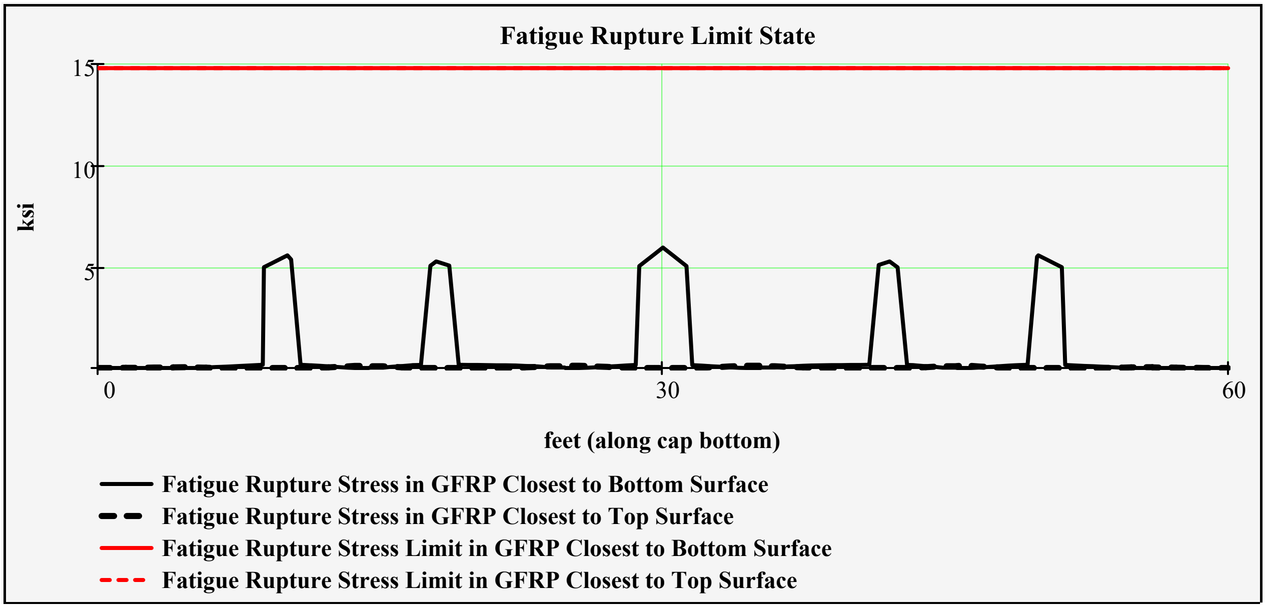
$Check_{fatigue.bot} :=$
```
Check ← "OK"
for i ∈ 1 .. #AllSections
  Check ← "NG" if f_f.Fat.bot_i > C_f·f_fd.pos
Check
```

$Check_{fatigue.top} :=$
```
Check ← "OK"
for i ∈ 1 .. #AllSections
  Check ← "NG" if f_f.Fat.top_i > C_f·f
Check
```

$Check_{fatigue.bot} = \text{"OK"}$ $Check_{fatigue.top} = \text{"OK"}$

$Check_{fatigue} :=$ | "OK"
| "NG, tensile stress in bottom reinforcing exceeds limit" if $(Check_{fatigue.bot} = \text{"NG"}) \cdot (Check_{fatigue.top} = \text{"OK}$
| "NG, tensile stress in top reinforcing exceeds limit" if $(Check_{fatigue.bot} = \text{"OK"}) \cdot (Check_{fatigue.top} = \text{"NG"})$
| "NG, tensile stress in top and bottom reinforcing exceeds limit" if $(Check_{fatigue.bot} = \text{"NG"}) \cdot (Check_{fatigue.to}$

$Check_{fatigue} = \text{"OK"}$



Skin Reinforcement [LRFD 5.7.3.4]

$d_{l.pos} := H_{cap} - b1 = 2.67\,ft$

$d_{l.neg} := H_{cap} - t1 = 2.67\,ft$

Distance from the extreme compression fiber to the centroid of extreme tension steel element

$Check_{skin.reinf.reqd} := \text{if}(d_{l.pos} \geq 3ft \vee d_{l.neg} \geq 3ft, \text{"Skin Reinf Required"}, \text{"Skin Reinf Not Required"})$

$Check_{skin.reinf.reqd} = \text{"Skin Reinf Not Required"}$

$Bar\#_{skin} = 5$ *Size of bar*

$\#Bars_{skin} = 2$ *Number of bars on each side face*

$A_{skin} := \frac{A(Bar\#_{skin}) \cdot \#Bars_{skin}}{2} = 0.31 \cdot in^2$

Area of skin reinforcement to one side face of the flexural tension side of the section (assume bars placed through full depth of section)

$h_{skin.pos} := \frac{d_{l.pos}}{2} = 1.33 \cdot ft$

Depth of the member on the flexural tension side of the section over which the skin reinforcement should be applied

$$h_{skin.neg} := \frac{d_{l.neg}}{2} = 1.33 \cdot ft$$

Area of skin reinforcement required on each side face of the flexural tension side of the section

$$A_{skin.reqd} := \begin{cases} 0 \text{ if } Check_{skin.reinf.reqd} = \text{"Skin Reinf Not Required"} \\ \min\left[\left[0.012 \cdot \left(\frac{d_{l.pos}}{in} - 30\right) \cdot \frac{in^2}{ft}\right] \cdot h_{skin.pos}, \left[0.012 \cdot \left(\frac{d_{l.neg}}{in} - 30\right) \cdot \frac{in^2}{ft}\right] \cdot h_{skin.neg}, \frac{A_{f1.top} + A_{f2.top}}{4}, \frac{A_{f1.bot} + A_{f2.bot}}{4}\right. \end{cases}$$

$$A_{skin.reqd} = 0 \cdot in^2$$

$$AreaR_{skin} := \frac{A_{skin.reqd}}{A_{skin} + 0.0001 in^2} = 0.00$$

Area Ratio of Shrinkage Reinforcement - Required to Provided

$$Check_{AreaSkinReinf} := \begin{cases} \text{if } Check_{skin.reinf.reqd} = \text{"Skin Reinf Required"} \\ \quad \begin{cases} \text{"NG" if } AreaR_{skin} > 1.005 \\ \text{"OK" otherwise} \end{cases} \\ \text{"Skin Reinf Not Required" otherwise} \end{cases}$$

$$Check_{AreaSkinReinf} = \text{"Skin Reinf Not Required"}$$

$$Spa_{skin.reqd} := \begin{cases} 0 \text{ if } Check_{skin.reinf.reqd} = \text{"Skin Reinf Not Required"} \\ \min\left(\frac{d_{l.pos}}{6}, \frac{d_{l.neg}}{6}, 12in\right) \text{ otherwise} \end{cases}$$

$$Spa_{skin.reqd} = 0 \cdot in$$

Required maximum spacing of skin reinforcement

$$Spa_{skin} = 8 \cdot in$$

Actual spacing of skin reinforcement

$$SpaR_{skin} := \frac{Spa_{skin}}{Spa_{skin.reqd} + 0.001 in} = 8000.00$$

Spacing Ratio of Shrinkage Reinforcement - Provided to Required

$$Check_{SpaSkinReinf} := \begin{cases} \text{if } Check_{skin.reinf.reqd} = \text{"Skin Reinf Required"} \\ \quad \begin{cases} \text{"NG" if } SpaR_{skin} > 1.005 \\ \text{"OK" otherwise} \end{cases} \\ \text{"Skin Reinf Not Required" otherwise} \end{cases}$$

$$Check_{SpaSkinReinf} = \text{"Skin Reinf Not Required"}$$

Shear Design [AASHTO BDS for GFRP 2.7]

Nominal Shear Resistance of the Concrete

The nominal shear resistance of the concrete shall be calculated as Eq. 2.7.3.4-1

$V_c = 0.0316 \cdot \beta \sqrt{f_c} b_v \cdot d_v$

Where $\beta = 5.0 \cdot k$ *for concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement specified in Aticale 2.7.2.4 using the simplified procedure*

Critical section of shear is typically at face of support, where the moment is negative. Use k *and* d *values calculated based on the negative bending case.*

$\beta_i := 5.0 k_{neg_i}$

$d_{v_i} := \max\left(d_{f.neg_i} - 0.5\beta_1 \cdot c_{neg_i}, 0.9 \cdot d_{f.neg_i}, 0.72 \cdot d_{cap_i}\right)$

Width of the web

$$
\begin{aligned}
b_v := \; & \text{for } i \in 1 .. \#AllSections \\
& \quad b_{w_i} \leftarrow W_{cap} \\
& \quad \text{for } j \in 1 .. \#Cols - 1 \\
& \quad\quad Void_{start} \leftarrow L_{cap.oh} + (j - 1) \cdot Spacing_{col} + 0.5\left(Spacing_{col} - L_{void}\right) \\
& \quad\quad Void_{end} \leftarrow L_{cap.oh} + j \cdot Spacing_{col} - 0.5\left(Spacing_{col} - L_{void}\right) \\
& \quad\quad b_{w_i} \leftarrow W_{cap} - W_{void} \;\; \text{if } Void_{start} < X_{design_i} < Void_{end} \\
& b_w
\end{aligned}
$$

$V_{c_i} := 0.0316 \cdot \beta_i \cdot \sqrt{f_{c.cap} \cdot ksi} \cdot b_{v_i} \cdot d_{v_i}$

$\max(V_c) = 64.75 \cdot kip$

$\min(V_c) = 53.52 \cdot kip$

Shear Resistance by Transverse Reinforcement

$f_{fd}(Bar) := C_E \cdot f_{fu}(Bar)$ *Design tensile strength of GFRP*

$f_{fb}(Bar) := \min\left(0.05 \cdot \frac{r(Bar)}{d(Bar)} + 0.3, 1\right) \cdot f_{fd}(Bar)$ *Design tensile strength of the bend of GFRP*

$f_{fv}(Bar) := \min\left(0.004 E_f, f_{fb}(Bar)\right)$ *Tensile strength of GFRP for shear design*

$$
\begin{aligned}
VS := \Big|\ & \text{for } i \in 1..\#AllSections\\
& \quad A_{v_i} \leftarrow A(Bar\#_{V1})\cdot\#Legs_{V1}\\
& \quad Spa_{v_i} \leftarrow Spa_{V1}\\
& \quad f_{v_i} \leftarrow f_{fv}(Bar\#_{V1})\\
& \quad \text{for } j \in 1..\#Cols - 1\\
& \quad\quad ZoneV2_{start1} \leftarrow L_{cap.oh} + (j-1)\cdot Spacing_{col} + L_{V1}\\
& \quad\quad ZoneV2_{end1} \leftarrow L_{cap.oh} + (j-1)\cdot Spacing_{col} + L_{V1} + L_{V2}\\
& \quad\quad ZoneV2_{start2} \leftarrow L_{cap.oh} + j\cdot Spacing_{col} - L_{V1} - L_{V2}\\
& \quad\quad ZoneV2_{end2} \leftarrow L_{cap.oh} + j\cdot Spacing_{col} - L_{V1}\\
& \quad\quad \text{if } \left(ZoneV2_{start1} \le X_{design_i} < ZoneV2_{end1}\right) \vee \left(ZoneV2_{start2} < X_{design_i} \le ZoneV2_{end2}\right)\\
& \quad\quad\quad A_{v_i} \leftarrow A(Bar\#_{V2})\cdot\#Legs_{V2}\\
& \quad\quad\quad Spa_{v_i} \leftarrow Spa_{V2}\\
& \quad\quad\quad f_{v_i} \leftarrow f_{fv}(Bar\#_{V2})\\
& \quad\quad \text{if } ZoneV2_{end1} \le X_{design_i} \le ZoneV2_{start2}\\
& \quad\quad\quad A_{v_i} \leftarrow A(Bar\#_{V3})\cdot\#Legs_{V3}\\
& \quad\quad\quad Spa_{v_i} \leftarrow Spa_{V3}\\
& \quad\quad\quad f_{v_i} \leftarrow f_{fv}(Bar\#_{V3})\\
& \quad \text{if } \left(0 \le X_{design_i} \le L_{cap.oh} - L_{V4}\right) \vee \left(L_{cap} - L_{cap.oh} + L_{V4} \le X_{design_i} \le L_{cap}\right)\\
& \quad\quad A_{v_i} \leftarrow A(Bar\#_{V5})\cdot\#Legs_{V5}\\
& \quad\quad Spa_{v_i} \leftarrow Spa_{V5}\\
& \quad\quad f_{v_i} \leftarrow f_{fv}(Bar\#_{V5})\\
& \quad \text{if } \left(L_{cap.oh} - L_{V4} < X_{design_i} < L_{cap.oh}\right) \vee \left(L_{cap} - L_{cap.oh} < X_{design_i} < L_{cap} - L_{cap.oh} + L_{V4}\right)\\
& \quad\quad A_{v_i} \leftarrow A(Bar\#_{V4})\cdot\#Legs_{V4}\\
& \quad\quad Spa_{v_i} \leftarrow Spa_{V4}\\
& \quad\quad f_{v_i} \leftarrow f_{fv}(Bar\#_{V4})\\
& \quad V_{f_i} \leftarrow \min\left(\frac{A_{v_i}\cdot f_{v_i}\cdot d_{v_i}}{Spa_{v_i}}, 0.25\cdot\sqrt{f_{c.cap}\cdot ksi}\cdot b_{v_i}\cdot d_{v_i}\right)\\
& \left(\frac{A_v}{in^2}\ \ \frac{Spa_v}{in}\ \ \frac{f_v}{ksi}\ \ \frac{V_f}{kip}\right)
\end{aligned}
$$

$A_{fv} := VS_{1,1}\cdot in^2$

Area of shear reinforcement per location

$Spa_{fv} := VS_{1,2} \cdot in$ *Spacing of shear reinforcement per location*

$f_{fv} := VS_{1,3} \cdot ksi$ *Tensile strength of FRP in shear design*

$V_f := VS_{1,4} \cdot kip$ *Shear resistance provided by FRP*

$V_n := V_c + V_f$ *Shear strength*

$\phi_v := 0.75$ *Resistance factor for shear & torsion*

$V_{r_i} := \phi_v \cdot V_{n_i}$ *Factored resistance*

$V_{u_i} := \max\left(\left|maxV_{u_i}\right|, \left|minV_{u_i}\right|\right)$ *Shear demand*

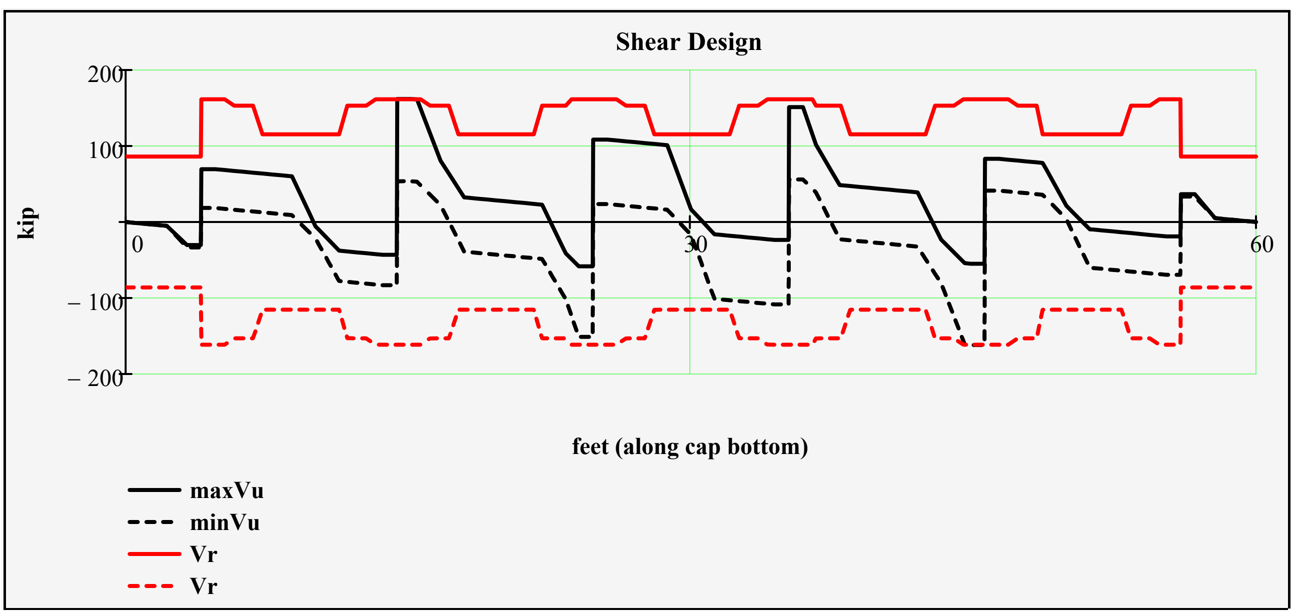
$$Check_{V.r} := \begin{array}{|l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \;\; \text{if } V_{r_i} < V_{u_i} \\ Check \end{array}$$

Check $V_r \ge V_u$

$Check_{V.r} = \text{"NG"}$

$DCR_{V_i} := \frac{V_{u_i}}{V_{r_i}}$ *Demand to capacity ratio*

$\max(DCR_V) = 1.00$ *The maximum demand to capacity ratio*



Check Spacing of stirrups

$$A_{fv.min_i} := \frac{0.05 \cdot b_{v_i} \cdot Spa_{fv_i}}{\frac{f_{fv_i}}{ksi}}$$ *Minimum transverse reinforcement*

$$Check_{A.v.min} := \begin{array}{|l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \;\; \text{if } A_{fv_i} < A_{fv.min_i} \wedge V_{u_i} > 0.5\phi_v \cdot V_{c_i} \\ Check \end{array}$$ *Check* $A_v \geq A_{v.min}$

$Check_{A.v.min} = \text{"NG"}$

$$Spa_{max_i} := \min\left(\frac{d_{v_i}}{2}, 24in\right)$$ *Maximum transverse reinforcement spacing*

$CriticalSpa_{reqd.shear} := \min(Spa_{max}) = 14.46 \cdot in$ *The most critical spacing along cap sections*

$$Check_{shear.spa} := \begin{array}{|l} Check \leftarrow \text{"OK"} \\ \text{for } i \in 1..\#AllSections \\ \quad Check \leftarrow \text{"NG"} \;\; \text{if } Spa_{fv_i} > Spa_{max_i} \\ Check \end{array}$$ *Check shear reinforcement spacing*

$Check_{shear.spa} = \text{"OK"}$

Check Longitudinal Reinforcement [AASHTO BDS for GFRP 2.7.3.7]

The check is applicable at the end bearing support areas. Therefore, this check is ignored.

Shrinkage and Temperature Reinforcement [AASHTO BDS for GFRP 2.9.6]

For conservatism, use the lowest f_{fu} *for the calculation of the minimum shrinkage and temperature reinforcement ratio*

$f_{fd.pos} = 59.19 \cdot ksi$ *Strength of Positive Reinforcement*

$f_{fd.neg} = 59.19 \cdot ksi$ *Strength of Negative Reinforcement*

$f_{fd.skin} := f_{fd}(Bar\#_{skin}) = 65.71 \cdot ksi$ *Strength of Skin Reinforcement*

$maxVbar := \max(Bar\#_{V1}, Bar\#_{V2}, Bar\#_{V3}, Bar\#_{V4}, Bar\#_{V5}) = 4$ *Maximum bar size of Shear Reinforcement*

$f_{fd.shear} := f_{fd}(maxVbar) = 75.6 \cdot ksi$ *Lowest Strength of Shear Reinforcement*

$f_{fd.ts} := \min(f_{fd.pos}, f_{fd.neg}, f_{fd.skin}, f_{fd.shear}) = 59.19 \cdot ksi$ *Design strength for calculation of Shrinkage and Temperature Reinforcement Ratio (Lowest Strength* *of All Reinforcement for conservatism)*

$$\rho_{f.ts} := 0.0018 \cdot \frac{60ksi}{f_{fd.ts}} \cdot \frac{29000ksi}{E_f} = 0.0081$$

$$\rho_{f.ts} := \max\left(0.0014, \min\left(\rho_{f.ts}, 0.0036\right)\right) = 0.0036$$

$$A_{shrink.reqd} := \frac{\rho_{f.ts} \cdot W_{cap} \cdot H_{cap}}{2\left(W_{cap} + H_{cap}\right)} = 0.44 \cdot \frac{in^2}{ft}$$

Area of required minimum shrinkage and temperature reinforcement per foot [LRFD 5.10.8]

$$A_{shrink.top} := \frac{A\left(Bar\#_{top}\right) \cdot \#Bars_{A1}}{W_{cap}} = 1.58 \cdot \frac{in^2}{ft}$$

Area of reinforcement near top surface

$$A_{shrink.bot} := \frac{A\left(Bar\#_{bot}\right) \cdot \#Bars_{D1}}{W_{cap}} = 1.58 \cdot \frac{in^2}{ft}$$

Area of reinforcement near bottom surface

$$A_{shrink.side} := \frac{A\left(Bar\#_{skin}\right) \cdot \#Bars_{skin} + A\left(Bar\#_{top}\right) + A\left(Bar\#_{bot}\right)}{H_{cap}} = 0.73 \cdot \frac{in^2}{ft}$$

Area of reinforcement near side surface

Area of shear reinforcement near surface (in^2/ft)

$A_{shrink.v} :=$
for $i \in 1 .. \#AllSections$
  $A_{v_i} \leftarrow A\left(Bar\#_{V1}\right)$
  $Spa_{v_i} \leftarrow Spa_{V1}$
  for $j \in 1 .. \#Cols - 1$
    $ZoneV2_{start1} \leftarrow L_{cap.oh} + (j - 1) \cdot Spacing_{col} + L_{V1}$
    $ZoneV2_{end1} \leftarrow L_{cap.oh} + (j - 1) \cdot Spacing_{col} + L_{V1} + L_{V2}$
    $ZoneV2_{start2} \leftarrow L_{cap.oh} + j \cdot Spacing_{col} - L_{V1} - L_{V2}$
    $ZoneV2_{end2} \leftarrow L_{cap.oh} + j \cdot Spacing_{col} - L_{V1}$
    if $\left(ZoneV2_{start1} \le X_{design_i} < ZoneV2_{end1}\right) \vee \left(ZoneV2_{start2} < X_{design_i} \le ZoneV2_{end2}\right)$
      $A_{v_i} \leftarrow A\left(Bar\#_{V2}\right)$
      $Spa_{v_i} \leftarrow Spa_{V2}$
    if $ZoneV2_{end1} \le X_{design_i} \le ZoneV2_{start2}$
      $A_{v_i} \leftarrow A\left(Bar\#_{V3}\right)$
      $Spa_{v_i} \leftarrow Spa_{V3}$
  if $\left(0 \le X_{design_i} \le L_{cap.oh} - L_{V4}\right) \vee \left(L_{cap} - L_{cap.oh} + L_{V4} \le X_{design_i} \le L_{cap}\right)$
    $A_{v_i} \leftarrow A\left(Bar\#_{V5}\right)$
    $Spa_{v_i} \leftarrow Spa_{V5}$
  if $\left(L_{cap.oh} - L_{V4} < X_{design_i} < L_{cap.oh}\right) \vee \left(L_{cap} - L_{cap.oh} < X_{design_i} < L_{cap} - L_{cap.oh} + L_{V4}\right)$
    $A_{v_i} \leftarrow A\left(Bar\#_{V4}\right)$

$$\left| \begin{array}{l} \left| \begin{array}{l} \left| \; Spa_{v_i} \leftarrow Spa_{V4} \right. \\ A_{shrink.v_i} \leftarrow \dfrac{A_{v_i}}{Spa_{v_i}} \end{array} \right. \\ A_{shrink.v} \end{array} \right.$$

$$\min(A_{shrink.v}) = 0.2 \cdot \frac{in^2}{ft}$$

Least amount of shear reinforcement along the cap near surface (in^2/ft)

$$A_{shrink.v.avg} := \frac{\sum_{i=2}^{\#AllSections} \left[A_{shrink.v_i} \cdot \left(X_{design_i} - X_{design_{i-1}} \right) \right]}{L_{cap}} = 0.47 \cdot \frac{in^2}{ft}$$

The area ratio of shrinkage reinforcement uses the cummulative average area of reinforcment for all sections along the cap.

$$A_{shrink} := \min(A_{shrink.top}, A_{shrink.bot}, A_{shrink.side}, A_{shrink.v.avg}) = 0.47 \cdot \frac{in^2}{ft}$$

Least amount of provided shrinkage reinforcement of all surfaces both directions

$$AreaR_{shrink} := \frac{A_{shrink.reqd}}{A_{shrink}} = 0.94$$

Area Ratio of Shrinkage Reinforcement - Required to Provided

$$Check_{AreaShrinkReinf} := \left| \begin{array}{ll} \text{"NG"} & \text{if } AreaR_{shrink} > 1.005 \\ \text{"OK"} & \text{otherwise} \end{array} \right.$$

$$Check_{AreaShrinkReinf} = \text{"OK"}$$

$$Spa_{shrink.reqd} := 12in$$

Maximum spacing of shrinkage and temperature reinforcement

$$Spa_{shrink.top} := \frac{W_{cap}}{\#Bars_{A1}} = 6 \cdot in$$

Spacing of reinforcement near top surface

$$Spa_{shrink.bot} := \frac{W_{cap}}{\#Bars_{D1}} = 6 \cdot in$$

Spacing of reinforcement near bottom surface

$$Spa_{shrink.side} := Spa_{skin} = 8 \cdot in$$

Spacing of reinforcement near side surface

$$Spa_{shrink.shear} := \max(Spa_{fv}) = 12 \cdot in$$

Spacing of shear reinforcement (max along the cap)

$$Spa_{shrink} := \max(Spa_{shrink.top}, Spa_{shrink.bot}, Spa_{shrink.side}, Spa_{shrink.shear}) = 12 \cdot in$$

Critical spacing

$$SpaR_{shrink} := \frac{Spa_{shrink}}{Spa_{shrink.reqd}} = 1.00$$

Spacing Ratio of Shrinkage Reinforcement - Provided to Required

$$Check_{SpaShrinkReinf} := \left| \begin{array}{ll} \text{"NG"} & \text{if } SpaR_{shrink} > 1.005 \\ \text{"OK"} & \text{otherwise} \end{array} \right.$$

$Check_{SpaShrinkReinf} = "OK"$

Mass Concrete Provisions [SDG 3.9]

$Surface_{cap} := 2 \cdot W_{cap} \cdot H_{cap} + \left(2W_{cap} + 2H_{cap}\right) \cdot L_{cap} = 862.6\ ft^2$ *Surface area of bent cap*

$Volume_{cap} := W_{cap} \cdot H_{cap} \cdot L_{cap} = 718.8 \cdot ft^3$ *Volume of bent cap*

Mass concrete provisions apply if the volume to surface area ratio, $\frac{Volume_{cap}}{Surface_{cap}} = 0.83\ ft$ *, exceeds 1 ft and the minimum dimension exceeds 3 feet*

$$SDG_{3.9} := \begin{cases} \text{"Use mass concrete provisions"} & \text{if } \frac{Volume_{cap}}{Surface_{cap}} > 1.0 \cdot ft \wedge W_{cap} > 3ft \wedge H_{cap} > 3ft \\ \text{"Use regular concrete provisions"} & \text{otherwise} \end{cases}$$

$SDG_{3.9} = $ "Use regular concrete provisions"

Lifting

$Dist_{left} := Dist$ $Dist_{right} := Dist$ *2-point Support/pick-up locations*

Moment & Stress Calculations

$L_{cap} = 59.9\ ft$ *Length of cap*

$Dist_b := L_{cap} - Dist_{left} - Dist_{right} = 59.9\ ft$

$w_{DC.cap} := \gamma_{conc} W_{cap} \cdot H_{cap} = 1.8 \cdot klf$ *Dead load due to cap self weight (To be conservative, voids not considered)*

$R1 := \frac{w_{DC.cap} \cdot L_{cap}}{2} = 53.91 \cdot kip$ *Reactions*

$R2 := R1 = 53.91 \cdot kip$

$$M_{DC} := \left| \begin{array}{l} \text{for } n \in 1 .. \#AllSections \\ \quad \left| \begin{array}{l} distr_n \leftarrow \left(-w_{DC.cap} \cdot X_{design_n}\right) \cdot \frac{X_{design_n}}{2} \\ mlifting_n \leftarrow distr_n \text{ if } X_{design_n} \le Dist_{left} \\ mlifting_n \leftarrow distr_n + R1 \cdot \left(X_{design_n} - Dist_{left}\right) \text{ if } \left(X_{design_n} \ge Dist_{left}\right) \cdot \left(X_{design_n} \le Dist_{left} + Dist_b\right) \\ mlifting_n \leftarrow distr_n + R1 \cdot \left(X_{design_n} - Dist_{left}\right) + R2 \cdot \left(X_{design_n} - Dist_{left} - Dist_b\right) \text{ if } X_{design_n} > \left(Dist_{left} + Dist_b\right) \end{array} \right. \\ mlifting \end{array} \right.$$

$M_{DC_i} := \text{if}\left(Dist = 0, 0, M_{DC_i}\right)$ *Set lifting moment to be 0 if input Dist = 0*

$IM := 1.25$ *Dynamic impact allowance*

$M_{lifting} := IM \cdot M_{DC}$ *Service I moment demand with dynamic impact allowance*

$M_{u.lifting} := 1.25 \cdot IM \cdot M_{DC}$ *Strength I moment demand with dynamic impact allowance*

$\max\left(M_{u.lifting}\right) = 0 \cdot kip \cdot ft$ *Maximum Strength I lifting moment along the cap*

$\min\left(M_{u.lifting}\right) = 0 \cdot kip \cdot ft$ *Minimum Strength I lifting moment along the cap*

$$
Check_{M.u.lifting} := \left|
\begin{array}{l}
Check \leftarrow \text{"Not Applicable"} \quad \text{if } Dist = 0 \\
\text{otherwise} \\
\quad \left| \begin{array}{l}
Check \leftarrow \text{"OK"} \\
\text{for } i \in 1 .. \#AllSections \\
\quad Check \leftarrow \text{"NG"} \quad \text{if } M_{r.pos_i} < M_{u.lifting_i} \vee M_{r.neg_i} < -M_{u.lifting_i}
\end{array} \right. \\
Check
\end{array}
\right.
$$

$Check_{M.u.lifting} = \text{"Not Applicable"}$

Cracking Moment

$\max\left(M_{cr}\right) = 480.57 \cdot kip \cdot ft$

Crack Control Calculations (if cracked)

Stress in the bottom reinforcing due to Service I limit state moment

$f_{f.lifting.bot_i} := ff_f\left(M_{lifting_i}, d_{f.pos_i}, d_{cap_i}, S_{cap_i}, k_{pos_i}, I_{cr.pos_i}, M_{cr_i}\right)$

$\max\left(f_{f.lifting.bot}\right) = 0 \cdot ksi$

Stress in the top reinforcing due to Service I limit state moment

$f_{f.lifting.top_i} := ff_f\left(-M_{lifting_i}, d_{f.neg_i}, d_{cap_i}, S_{cap_i}, k_{neg_i}, I_{cr.neg_i}, M_{cr_i}\right)$

$\max\left(f_{f.lifting.top}\right) = 0 \cdot ksi$

Estimated crack width at lifting

$$CrackW_{lifting.bot_i} := 2 \frac{f_{f.lifting.bot_i}}{C_b \cdot E_f} \cdot \zeta_{pos_i} \cdot \sqrt{d_{c.bot}^{\ 2} + \left(\frac{Spa_{bar.bot_i}}{2}\right)^2}$$

$$CrackW_{lifting.top_i} := 2 \frac{f_{f.lifting.top_i}}{C_b \cdot E_f} \cdot \zeta_{neg_i} \cdot \sqrt{d_{c.top}^{\ 2} + \left(\frac{Spa_{bar.top_i}}{2}\right.}$$

$\max(CrackW_{lifting.bot}) = 0.000 \cdot in$

$\max(CrackW_{lifting.top}) = 0.000 \cdot in$

Check crack width against the max allowable of 0.028in recommended by AASHTO

$CrackLimit = 0.028 \cdot in$

$Check_{crack.control.lifting} :=$

```
Check ← "Not Applicable"   if Dist = 0
otherwise
    Check ← "OK"
    for i ∈ 1 .. #AllSections
        Check ← "NG"   if CrackW_lifting.bot_i > CrackLimit ∨ CrackW_lifting.top_i > CrackLimit
Check
```

$Check_{crack.control.lifting} = \text{"Not Applicable"}$

LRFD Moment, Shear, Min & Max Reinforcement, and Lifting Moment Checks

Summary of LRFD and SDG Checks

Save Data

Positive Moment

$Check_{M.r.pos} = \text{"OK"}$

$max(DCR_{M.pos}) = 0.48$

$Check_{minAf.bot} = \text{"OK"}$

$Check_{crack.control.bot} = \text{"OK"}$

$max(CrackW_{SerI.bot}) = 0.017 \cdot in$

$Check_{creep.bot} = \text{"OK"}$

$max(f_{f.SL.bot}) = 0.11 \cdot ksi$

$Check_{fatigue.bot} = \text{"OK"}$

$max(f_{f.Fat.bot}) = 5.94 \cdot ksi$

$C_f \cdot f_{fd.pos} = 14.8 \cdot ksi$

Negative Moment

$Check_{M.r.neg} = \text{"OK"}$

$max(DCR_{M.neg}) = 0.24$ *The maximum demand to capacity ratio*

$Check_{minAf.top} = \text{"OK"}$

$Check_{crack.control.top} = \text{"OK"}$

$max(CrackW_{SerI.top}) = 0.000 \cdot in$ *The maximum crack width*

$Check_{creep.top} = \text{"OK"}$

$max(f_{f.SL.top}) = 0.08 \cdot ksi$ *The maximum stress under sustained load (DL+0.2LL)*

$Check_{fatigue.top} = \text{"OK"}$

$max(f_{f.Fat.top}) = 0.11 \cdot ksi$ *The maximum stress under fatigue (DL+1.5LL.fatigue)*

$C_f \cdot f_{fd.neg} = 14.8 \cdot ksi$ *Fatigue stress limit*

Shear Checks

$Check_{V.r} = \text{"NG"}$

$max(DCR_V) = 1.00$ *The maximum demand to capacity ratio*

$Check_{A.v.min} = \text{"NG"}$

$Check_{shear.spa} = \text{"OK"}$

$CriticalSpa_{reqd.shear} = 14.5 \cdot in$ *The allowable spacing for shear reinforcement at the most critical cap section*

Skin Reinforcement

$Check_{AreaSkinReinf} = \text{"Skin Reinf Not Required"}$

$A_{skin.reqd} = 0.00 \cdot in^2$

$Check_{SpaSkinReinf} = \text{"Skin Reinf Not Required"}$

$Spa_{skin.reqd} = 0.00 \cdot in$

Shrinkage and Temperature Reinforcement

$Check_{AreaShrinkReinf} = \text{"OK"}$

$A_{shrink.reqd} = 0.44 \cdot \frac{in^2}{ft}$

$Check_{SpaShrinkReinf} = \text{"OK"}$

$Spa_{shrink.reqd} = 12.00 \cdot in$

Lifting Checks

$Check_{M.u.lifting} = \text{"Not Applicable"}$

Mass concrete requirements

$SDG_{3.9} = \text{"Use regular concrete provisions"}$

$Check_{crack.control.lifting}$ = "Not Applicable"