Beam Data

$mp := 10$

Beam length (ft) = $length := 100$

Composite slab strength (ksi) = $fc := 4$

Concrete unit weight (kcf) = $\gamma c := 0.150$

Initial strength of concrete (ksi) = $fci := 6$

Final Strength of concrete (ksi) = $fcf := 8$

Modulus of beam concrete based on final (ksi) = $Ec := 33000 \cdot \gamma c^{1.5} \cdot \sqrt{fcf}$ $Ec = 5422.453$

Modulus of slab concrete (ksi) = $Esl := 33000 \cdot \gamma c^{1.5} \cdot \sqrt{fc}$ $Esl = 3834.254$

Number of Spans = $spans := 1$ $n := 0..\, spans - 1$ $n2 := 0..\, 1$

Which span is used in design = $comp1 := 1$

Length of all spans (ft) = $L_n := 100$

Should the haunch depth be used in calculations (yes or no) = $ha\_dec := \text{"yes"}$

Depress point to use for draped strands = $depress := 0.4$

Number of span points calculations shall be done to = $sp := 20$ $ns10 := 0..\, 10$
(Please choose only an even number of points)

Interior or Exterior beam used in design (intput "int" or "ext") = $aa := \text{"int"}$

Beam type to use

1 = AASHTO TYPE I
2 = AASHTO TYPE II
3 = AASHTO TYPE III
4 = AASHTO TYPE IV
5 = BT54
6 = BT63
7 = BT72
8 = IDOT 36 INCH
9 = IDOT 42 INCH
10 = IDOT 48 INCH
11 = IDOT 54 INCH
12 = Box

$type := 4$

Box Beam dimensions (if no box set to zero)

Width (in) = $a1 := 0$

Depth (in) = $a2 := 0$

Top flange (in) = $a3 := 0$

Bottom Flange (in) = $a4 := 0$

Web (in) = $a5 := 0$

Beam area (in^2) = $Area = 789$

Distance from bottom to cg (in) = $yb = 24.73$

Section inertia (in^2) = $Inc = 260730$

Beam weight (k/ft) = $bwt = 0.822$

Web thickness (in) = $web = 8$

Total beam depth (in) = $h = 54$

Width of top flange (in) = $fwt = 20$

Strand pattern Data

strand :=

PICK TYPE	Description english	DIAMETER in	AREA in^2	WEIGHT PER LENGTH lb/ft	Fpu ksi	STEEL TYPE
0	6/10-270k	0.6000	0.2170	0.7446	270	SR
1	6/10-270k-LL	0.6000	0.2170	0.7446	270	LL
2	9/16-270k	0.5625	0.1920	0.6588	270	SR
3	9/16-270k-LL	0.5625	0.1920	0.6588	270	LL
4	1/2-270k	0.5000	0.1530	0.5250	270	SR
5	**1/2-270k-LL**	**0.5000**	**0.1530**	**0.5250**	**270**	**LL**
6	**1/2-270k-SP**	**0.5000**	**0.1670**	**0.5730**	**270**	**LL**
7	7/16-270k	0.4375	0.1150	0.3946	270	SR
8	7/16-270k-LL	0.4375	0.1150	0.3946	270	LL
9	3/8-270k	0.3750	0.0800	0.2745	270	SR
10	3/8-270k-LL	0.3750	0.0800	0.2745	270	LL

Strand Type to use $s\_type := 1$

$Strand\_description := strand_{s\_type,0}$ Strand_description = "6/10-270k-LL"

$Strand\_diameter := strand_{s\_type,1}$ $Strand\_diameter = 0.6$

$Strand\_area := strand_{s\_type,2}$ $Strand\_area = 0.217$

$Strand\_weight := strand_{s\_type,3}$ $Strand\_weight = 0.745$

$Strand\_strength := strand_{s\_type,4}$ $Strand\_strength = 270$

$Strand\_type := strand_{s\_type,5}$ Strand_type = "LL"

Transfer length = 60*bd $transfer := 60 \cdot Strand\_diameter$ $transfer = 36$

Calculations of Dead Loads, non-composite and composite

General Information

Out to out width (ft) = $oto := 40.5$

Beam spacing (ft) = $bs := 8$

Slab thickness (ft) = $slab := 8.25$ $ts := slab$

Wearing surface (ksf) = $wear := 0.025$

Number of beams = $beams := 5$

Width of one lane (ft) = $lane\_width := 10$

Multiple presence factor = $RF := 1.0$

Top slab to top beam (in) = $tstw := 12.75$

Haunch Selection $haunch := tstw - slab$ $haunch = 4.5$ $ha := if(ha\_dec = \text{"yes"}, haunch, 0)$ $ha = 4.5$

Beam weight per foot (k/ft) = $bwt = 0.822$

Max span length (ft) =
(for ETFW) $max\_span := length$ $max\_span = 100$

Width of top flange of beam (in) = $fwt = 20$

RAIL OR PARAPET DATA

Rail width on outside (ft) = $outside := 1.0$

Rail weight per foot (k/ft) = $railwt := 0.5$

Number of parapet's = $npar := 2$

MEDIAN BARRIER DATA

Median barrier width (ft) = $med\_width := 0$

Median barrier weight (k/ft) = $median := 0$

Number of barriers = $nmed := 0$

Diaphragm Data

Weight of Diaphragms (k) = $wdia := 1.664$

Note: Program assumes diaphragms are point loads at equal spaces over the length of the beam.

Number of Diaphragms (k) = $ndia := 2$

Optional Loads

If you do not wish to use any of the optional loads then simply set the values to zero. If SIP metal forms will be used then the first three should probably be used. However, it is most certanly not necessary to adjust for the deck grooving.

SIP form weight (psf) = $sipw := 3$

Depth of valley in SIP form (in) = $vald := 2$

Amount of deflection in SIP form (in) = $sipd := 0.5$

If the user so desires, you may adjust the deck weight for the deck grooving, just enter the depth of grooving. Enter a positive value for an increased thickness, and enter a negative value for an decreased thickness. This adjustment in really not necessary at all, and the user may set the value equal to 0.

$gt := .5$

Filler weight (k/ft) = $filler := \frac{fwt \cdot haunch}{144} \cdot \gamma c$ $filler = 0.094$

SIP form (k/ft) =
say (3 psf) $SIP := \left(bs - \frac{fwt}{12}\right) \cdot \frac{sipw}{1000}$ $SIP = 0.019$

Concrete in valley of SIP form (k/ft) =
(say each inch of valley is equal to
1/2" of concrete depth) $valley := \left(bs - \frac{fwt}{12}\right) \cdot \frac{vald}{24} \cdot \gamma c$ $valley = 0.079$

Weight from deflections (k/ft) =
(this assumes that the SIP form
will deflect, adding about 1/2"
depth for every 1" of deflection) $wdefl := \left(bs - \frac{fwt}{12}\right) \cdot \frac{sipd}{24} \cdot \gamma c$ $wdefl = 0.02$

Deck grooving (k/ft) =
(Say that the deck
grooving adds 1/4"
in depth) $groov := bs \cdot \frac{gt}{24} \cdot \gamma c$ $groov = 0.025$

Total optional loads (k/ft) = $optional := filler + SIP + valley + wdefl$ $optional = 0.212$

Final Composite and Non-Composite Loads

NON COMPOSITE DL (excluding beam weight) (DLnc) (DC)

$$DLnc := \max\left(\begin{pmatrix} \dfrac{oto \cdot \frac{slab}{12}}{beams} \cdot \gamma c \\ bs \cdot \frac{slab}{12} \cdot \gamma c \end{pmatrix}\right) + optional \qquad DLnc = 1.047$$

COMPOSITE DL (DW)

Roadway width (ft) = $roadway := oto - npar \cdot outside - med\_width$ $roadway = 38.5$

$$DLc := \frac{roadway \cdot wear + railwt \cdot npar + median \cdot nmed}{beams} + groov \qquad DLc = 0.417$$

Unit Load for Diaphragm, to be used only for Deflections (the actual point loads will be used for shear and moment)

$$dwt := \frac{wdia \cdot ndia}{length} \qquad dwt = 0.033$$

Unit weight to be used in in the calculation of Non-Composite DL Deflection

$$w\_defl := DLnc + \frac{railwt \cdot npar + median \cdot nmed}{beams} + dwt$$

Effective flange width (LRFD 4.6.2.6.1) (use the smaller of interior or exterior)

Interior - smaller of the following

1. 1/4 span length
2. center to center beams
3. 12*T+B ; B = larger of the web thickness or 1/2 top flange width

$etfw1 := \frac{length}{4} \cdot 12$ $etfw1 = 300$

$etfw2 := bs \cdot 12$ $etfw2 = 96$

$etfw3 := 12 \cdot slab + \frac{fwt}{2}$ $etfw3 = 109$

$ETFW\_int := \min\left(\begin{pmatrix} etfw1 \\ etfw2 \\ etfw3 \end{pmatrix}\right)$ $ETFW\_int = 96$

Exterior - 1/2 effective width of adjacent interior beam plus the smaller of the following

1. 1/8 Effective Span
2. 6*ts + B ; B = largter of the web thickness or 1/2 top flange width
3. overhang

$etfw1 := \frac{length}{8} \cdot 12$ $etfw1 = 150$

$etfw2 := 6 \cdot slab + \frac{fwt}{2}$ $etfw2 = 59.5$

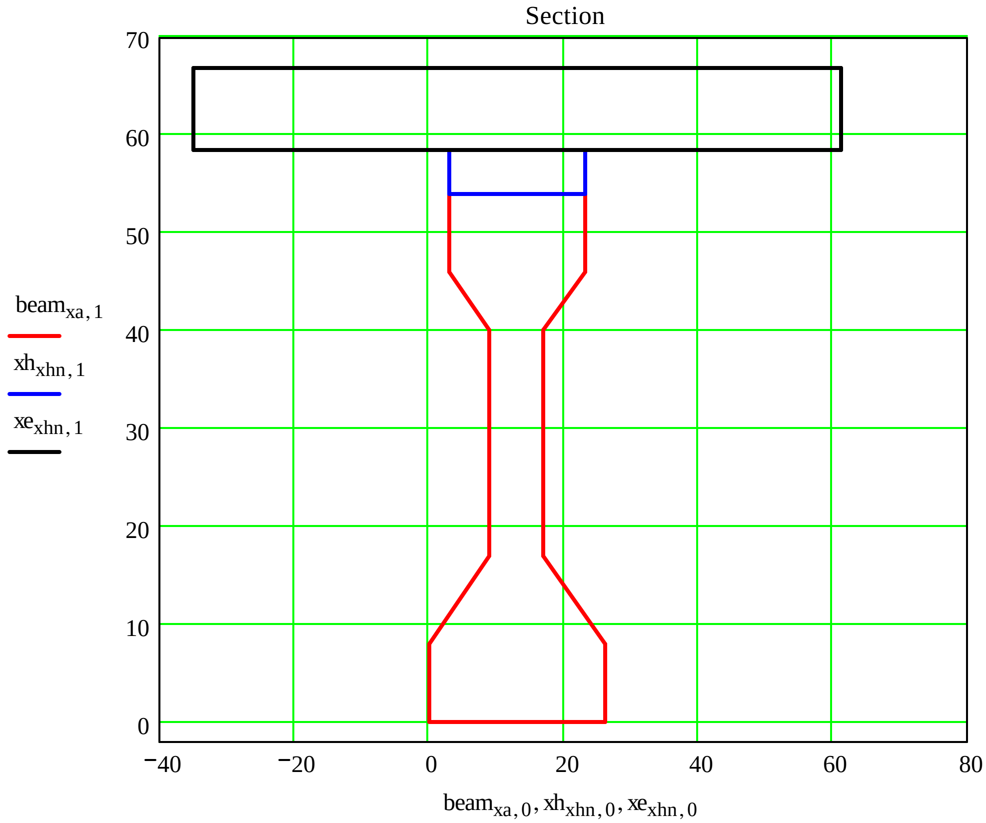
$etfw3 := \frac{oto - (beams - 1) \cdot bs}{2} \cdot 12$ $etfw3 = 51$

$ETFW\_ext := \min\left(\begin{pmatrix} etfw1 \\ etfw2 \\ etfw3 \end{pmatrix}\right)$ $ETFW\_int = 96$

Effective flange width used in design

$ETFW := \left| \begin{array}{l} ETFW\_ext \ \text{ if } aa = \text{"ext"} \\ ETFW\_int \ \text{ otherwise} \end{array} \right.$ $ETFW = 96$

Section Diagram



Composite moment of Inertia

Effective compression slab width (in) = $ETFW = 96$

Modular ratio = $\eta := \frac{\sqrt{fc}}{\sqrt{fcf}}$ $\eta = 0.707$

Transformed slab width (in) = $b := ETFW \cdot \eta$ $b = 67.882$

Slab thickness (in) = $ts = 8.25$

Composite distance from bottom to c.g. (in) = $ybc := \frac{b \cdot ts \cdot \left(h + ha + \frac{ts}{2}\right) + Area \cdot yb}{b \cdot ts + Area}$ $ybc = 40.462$

Composite N.A. to top beam (in) = $ytb := h - ybc$ $ytb = 13.538$

Composite N.A. to top slab (in) = $yts := h + ts + ha - ybc$ $yts = 26.288$

Composite moment of inertia (in^t) = $Ic := Inc + \frac{b \cdot ts^3}{12} + Area \cdot (yb - ybc)^2 + b \cdot ts \cdot \left(yts - \frac{ts}{2}\right)^2$

$Ic = 734265.849$

Composite Section Modulus

Section modulus bottom of beam (in^3) = $Sbc := \frac{Ic}{ybc}$ $Sbc = 18147.259$

Section modulus top beam (in^3) = $Stb := \frac{Ic}{ytb}$ $Stb = 54235.51$

Section modulus top concrete (in^3) = $Stc := \frac{Ic}{yts} \cdot \frac{1}{\eta}$ $Stc = 39500.538$

Non-Composite Section Modulus

Section modulus bottom of beam (in^3) = $Sb := \frac{Inc}{yb}$ $Sb = 10543.065$

Section modulus top beam (in^3) = $St := \frac{Inc}{h - yb}$ $St = 8907.755$

Live Load Distribution Factors

LRFD 3.6.1.1.1 - Number of design lanes

$$lanes := floor\left(\frac{roadway}{12}\right) \qquad lanes = 3$$

Table 4.6.2.2.2.b-1 - Interior beam distribution factor

Range of applicability ; 3.5 <= S <= 16
4.5 <= ts <= 20
20 <= L <= 240
Nb >= 4
10,000 <= Kg <= 7,000,000

Distance from N.A. non composite beam and CL. deck (in) =

$$eg := \left(h + \frac{ts}{2}\right) - yb \qquad eg = 33.395$$

$$kg := \sqrt{\frac{fcf}{fc}} \cdot \left(Inc + Area \cdot eg^2\right) \qquad kg = 1613113.272$$

$$DFM_I := 0.075 + \left(\frac{bs}{9.5}\right)^{0.6} \cdot \left(\frac{bs}{length}\right)^{0.2} \cdot \left(\frac{kg}{12 \cdot length \cdot slab^3}\right)^{0.1} \qquad DFM_I = 0.669$$

Table 4.6.2.2.2.d-1 - Exterior beam distribution factor for Moment

Range of applicibility -1.0 <= de <= 5.5

$$de := \frac{oto - (beams - 1) \cdot bs}{2} - 1 \qquad de = 3.25$$

$$\left| \begin{array}{ll} \text{"OK"} & \text{if } (-1.0 \le de)(de \le 5.5) \\ \text{"NG"} & \text{otherwise} \end{array} \right. = \text{"OK"}$$

$$e := max\left(\begin{pmatrix} 0.77 + \frac{de}{9.1} \\ 1.0 \end{pmatrix}\right) \qquad e = 1.127$$

$$DFM_E := DFM_I \cdot e \qquad DFM_E = 0.754$$

Distribution Factor for Moment Used in Design

$$LLDFM := \begin{cases} DFM_I & \text{if } aa = \text{"int"} \\ \text{otherwise} & \begin{cases} DFM_E & \text{if } aa = \text{"ext"} \\ 0 & \text{otherwise} \end{cases} \end{cases}$$

$LLDFM = 0.669$

Table 4.6.2.2.3.a-1 - Interior beam distribution factor for shear

Range of applicibility:
3.5 <= S <= 16
20 <= L <= 240
4.5 <= ts <= 12
10000 <= kg <= 7,000,000
Nb >= 4.0

$$DFV_I := 0.2 + \frac{bs}{12} - \left(\frac{bs}{35}\right)^2$$

$DFV_I = 0.814$

Table 4.6.2.2.3b-1 - Exterior beam distribution factor for shear

Range of applicibility -1 <= de <= 5.5

$$e := \max\left(\begin{pmatrix} 0.6 + \frac{de}{10} \\ 1.0 \end{pmatrix}\right)$$

$e = 1$

$$DFV_E := DFV_I \cdot e$$

$DFV_E = 0.814$

Distribution Factor for Shear Used in Design

$$LLDFV := \begin{cases} DFV_I & \text{if } aa = \text{"int"} \\ \text{otherwise} & \begin{cases} DFV_E & \text{if } aa = \text{"ext"} \\ 0 & \text{otherwise} \end{cases} \end{cases}$$

$LLDFV = 0.814$

If the user wants to overide the distribution factors that have been calculated, simply enable the two numbers below and imput the desired factor.

Live Load distribution factor for moment $LLDFM := 0.660$ *currently disabled*

Live Load distribution factor for shear $LLDFV := 0.814$ *currently disabled*

ALLOWABLE STRESS IN CONCRETE

At release 5.9.4.1

Compression (ksi) = 5.9.4.1.1 $fic := 0.6 \cdot fci$ $fic = 3.6$

Tensiion (ksi) = 5.9.4.1.2 $fit := -0.22 \cdot \sqrt{fci}$ $fit = -0.539$

At final conditions 5.9.4.2

Case I full PS + DL + LL

Compression (ksi) = $fc1 := 0.6 \cdot fcf$ $fc1 = 4.8$

Tension (ksi) = $fft := -0.19 \cdot \sqrt{fcf}$ $fft = -0.537$

Case II PS + DL

Compression (ksi) = $fc2 := 0.45 \cdot fcf$ $fc2 = 3.6$

Tension (ksi) = $fft = -0.537$

Case III 50%PS + 50%DL + LL

Compression (ksi) = $fc3 := 0.4 \cdot fcf$ $fc3 = 3.2$

Tension (ksi) = $fft = -0.537$

Simple Span Shear and Moment

Span length (ft) = $length = 100$

Data range (ft) = $rg_{ns10} := \left| \begin{array}{l} \text{for } j \in 0..10 \\ \quad j1_j \leftarrow length \cdot \dfrac{j}{10} \\ j1_{ns10} \end{array} \right.$

Beam Weight (k/ft) = $bwt = 0.822$

Self weight Moment at tenth points (k*ft) = $Mself_{ns10} := \dfrac{bwt \cdot rg_{ns10}}{2} \cdot \left(length - rg_{ns10}\right)$

Self weight Shear (k) = $Vself_{ns10} := bwt \cdot \left(\dfrac{length}{2} - rg_{ns10}\right)$

Non composite moment (k*ft) = $Mnonc_{ns10} := \dfrac{DLnc \cdot rg_{ns10}}{2} \cdot \left(length - rg_{ns10}\right)$

Non composite shear (k) = $Vnonc_{ns10} := DLnc \cdot \left(\dfrac{length}{2} - rg_{ns10}\right)$

Shear and Moment from Diaphragm

$$ns11 := 0..\left|\begin{array}{l} ndia - 1 \text{ if } ndia \neq 0 \\ 0 \text{ otherwise} \end{array}\right.$$

Load for diaphragm (k) = $P := wdia$ $P = 1.664$

Range for variable "x" = rg_{ns10}

Definition of variable "a" = $ad_{ns11} := \frac{length}{ndia + 1}\cdot(ns11 + 1)$ $ad = \begin{pmatrix} 33.333 \\ 66.667 \end{pmatrix}$

Definition of variable "b" = $bd_{ns11} := length - ad_{ns11}$ $bd = \begin{pmatrix} 66.667 \\ 33.333 \end{pmatrix}$

Moment at point of load (k*ft) =

$$Md1_{ns10,ns11} := \left|\begin{array}{l} \frac{P\cdot bd_{ns11}\cdot rg_{ns10}}{length} \text{ if } rg_{ns10} < ad_{ns11} \\ \frac{P\cdot bd_{ns11}\cdot rg_{ns10}}{length} - P\cdot\left(rg_{ns10} - ad_{ns11}\right) \text{ otherwise} \end{array}\right.$$

$$Md_{ns10} := \sum_{ns11} Md1_{ns10,ns11}$$

Shear at point of load (k) =

$$Vd1_{ns10,ns11} := \left|\begin{array}{l} \frac{P\cdot bd_{ns11}}{length} \text{ if } rg_{ns10} < ad_{ns11} \\ \frac{P\cdot bd_{ns11}}{length} - P \text{ otherwise} \end{array}\right.$$

$$Vd_{ns10} := \sum_{ns11} Vd1_{ns10,ns11}$$

Md =

	0
0	0
1	16.64
2	33.28
3	49.92
4	55.467
5	55.467
6	55.467
7	49.92
8	33.28
9	16.64
10	0

Moment from Diaphragm (k*ft)

Vd =

	0
0	1.664
1	1.664
2	1.664
3	1.664
4	0
5	0
6	0
7	-1.664
8	-1.664
9	-1.664
10	-1.664

Shear from Diaphragm (k)

Moment and Shear, Generated by DL on the Composite Section.

This generator is capable of handling from 1 to 10 spans, and is capable of returning values for continuous sections. This is done by moment distribution. The values returned are SL.

Use a unit load "w" = 1.0 $unit := DLc$ $unit = 0.417$

disp =

	0	1	2
0	1	0	20.875
1	1.1	187.875	16.7
2	1.2	334	12.525
3	1.3	438.375	8.35
4	1.4	501	4.175
5	1.5	521.875	0
6	1.6	501	-4.175
7	1.7	438.375	-8.35
8	1.8	334	-12.525
9	1.9	187.875	-16.7
10	2	0	-20.875
11			
12			
13			
14			
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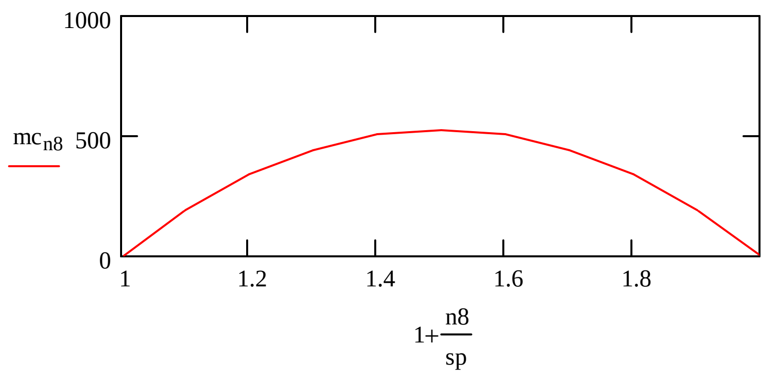
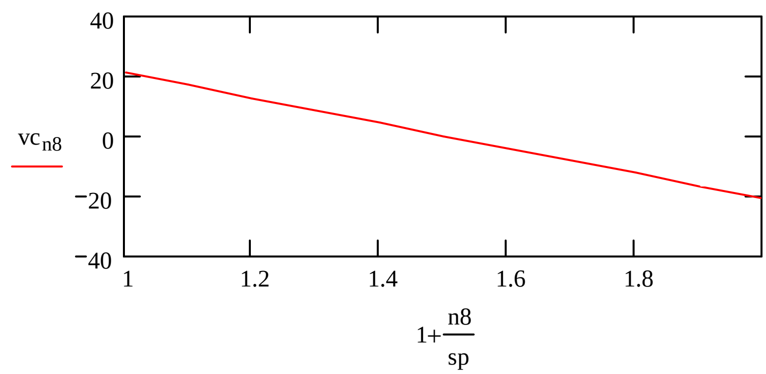
36			
37			
38			
39			

column 0 = span point
column 1 = moment
column 2 = shear

Based on continuous section, constant inertia.



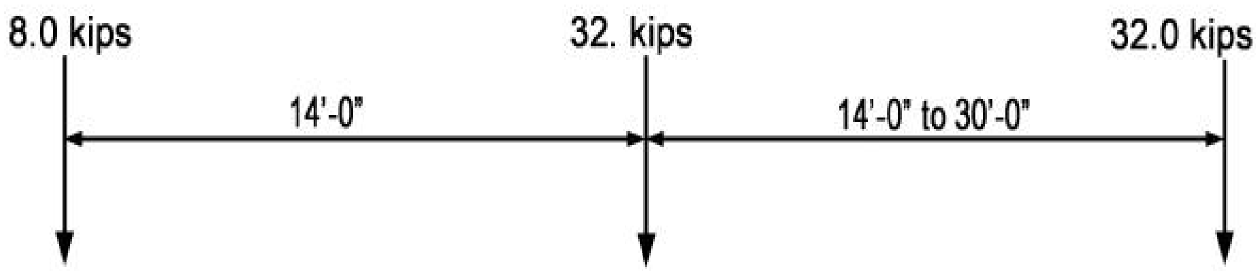


Notes on Live Load:

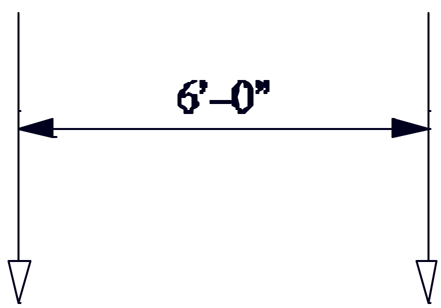
The HL-93 LL shall be used as described in 3.6.1.2 (LRFD)

The Design Lane: The design lane shall consist of a load of 0.640 k/ft uniformaly distributed in the longitudinal direction. Transversley the load shall be assumed to be 10 ft wide. DO NOT apply the dynamic load allowance (Impact) to the lane. The design lane shall accompany the design truck and tandem.

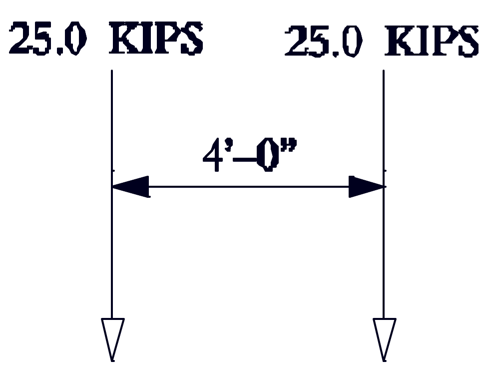
The Design Truck



Design truck axal spacing from rear



The Design Tandem: The design tandem consists of a pair of 25k axles spaced 4ft apart. Apply the dynamic load allowance to the tandem



Load Combinations

Combination 1: The effect of the design tandem combined with the effect of the design lane.

Combination 2: The effect of the design truck combined with the effect of the design lane.

Combination 3: For both the negative moment between points of contraflexure under a uniform load on all spans, and reaction at interior piers only.

90% of two design trucks spaced a minimum of 50 ft between the lead axle of truck 2 and the rear axle of truck 1.
90% the design Lane

The distance between 32 k axles shall be 14 ft.

Moment, SL, LLDF = 1.0 wheels, Impact included, input to tenth points

ldm :=

	DC LOADS (non-comp)		DW Loads	LL + I	
LOCATION	self wt	other	(comp)	M (+)	M (-)
0	0.00	0.00	0.00	0.00	0.00
0.1	369.90	487.80	187.88	1070.00	0.00
0.2	657.60	870.90	334.00	1883.00	0.00
0.3	863.10	1149.29	438.38	2473.00	0.00
0.4	986.40	1311.89	501.00	2763.00	0.00
0.5	1027.50	1364.24	521.88	2846.00	0.00
0.6	986.40	1311.89	501.00	2763.00	0.00
0.7	863.10	1149.29	438.38	2473.00	0.00
0.8	657.60	870.90	334.00	1883.00	0.00
0.9	369.90	487.80	187.88	1070.00	0.00
1	0.00	0.00	0.00	0.00	0.00

the other loads include slab, diaphragms (if there are any) and any other non-composite loads.

(Mself Mnonc mca Md)

Shear Load, SL, LLDF = 1 wheels, Impact included, input to tenth points

ldv :=

	DC LOADS (non-comp)		DW Loads	LL + I	
LOCATION	self wt	other	(comp)	V (+)	V (-)
0	41.10	54.02	20.88	120.00	0.00
0.1	32.88	43.54	16.70	105.00	-6.00
0.2	24.66	33.07	12.53	90.00	-14.00
0.3	16.44	22.60	8.35	75.00	-23.00
0.4	8.22	10.47	4.18	61.00	-35.00
0.5	0.00	0.00	0.00	48.00	-48.00
0.6	-8.22	-10.47	-4.18	35.00	-61.00
0.7	-16.44	-22.60	-8.35	23.00	-75.00
0.8	-24.66	-33.07	-12.53	13.00	-90.00
0.9	-32.88	-43.54	-16.70	6.00	-105.00
1	-41.10	-54.02	-20.88	0.00	-120.00

the other loads include slab, diaphragms (if there are any) and any other non-composite loads.

(Vself Vnonc vca Vd)

Expand area for moment and shear iterations, Also LLDF is applied here

Service I loads (moment)

$$\begin{pmatrix} full \\ SI1 \\ SI2 \\ SI3 \\ SI4 \\ SI5 \\ SI6 \\ SI7 \end{pmatrix} :=$$

	SI1	SI2	SI3	SI4	SI5	SI5	SI7
	DC LOADS (non-comp)		DW Loads	LL + I		TOTAL LOADS	
LOCATION	self wt	other (slab)	(comp)	M (+)	M (-)	M (+)	M (-)
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.05	184.95	243.90	93.94	357.89	0.00	880.68	522.79
1.1	369.90	487.80	187.88	715.78	0.00	1761.35	1045.57
1.15	513.75	679.35	260.94	987.70	0.00	2441.74	1454.04
1.2	657.60	870.90	334.00	1259.63	0.00	3122.13	1862.50
1.25	760.35	1010.09	386.19	1456.97	0.00	3613.61	2156.63
1.3	863.10	1149.29	438.38	1654.31	0.00	4105.08	2450.77
1.35	924.75	1230.59	469.69	1751.31	0.00	4376.34	2625.03
1.4	986.40	1311.89	501.00	1848.31	0.00	4647.60	2799.29
1.45	1006.95	1338.07	511.44	1876.07	0.00	4732.53	2856.45
1.5	1027.50	1364.24	521.88	1903.83	0.00	4817.45	2913.62
1.55	1006.95	1338.07	511.44	1876.07	0.00	4732.53	2856.45
1.6	986.40	1311.89	501.00	1848.31	0.00	4647.60	2799.29
1.65	924.75	1230.59	469.69	1751.31	0.00	4376.34	2625.03
1.7	863.10	1149.29	438.38	1654.31	0.00	4105.08	2450.77
1.75	760.35	1010.09	386.19	1456.97	0.00	3613.61	2156.63
1.8	657.60	870.90	334.00	1259.63	0.00	3122.13	1862.50
1.85	513.75	679.35	260.94	987.70	0.00	2441.74	1454.04
1.9	369.90	487.80	187.88	715.78	0.00	1761.35	1045.57
1.95	184.95	243.90	93.94	357.89	0.00	880.68	522.79
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00

ldm_f

Service III loads (moment)

$\begin{pmatrix} SIII1 \\ SIII2 \\ SIII3 \\ SIII4 \\ SIII5 \\ SIII6 \\ SIII7 \end{pmatrix} :=$

	SIII1	SIII2	SIII3	SIII4	SIII5	SIII5	SIII7
	DC LOADS (non-comp)		DW Loads	LL + I		TOTAL LOADS	
LOCATION	self wt	other (slab)	(comp)	M (+)	M (-)	M (+)	M (-)
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.05	184.95	243.90	93.94	286.31	0.00	809.10	522.79
1.1	369.90	487.80	187.88	572.62	0.00	1618.20	1045.57
1.15	513.75	679.35	260.94	790.16	0.00	2244.20	1454.04
1.2	657.60	870.90	334.00	1007.71	0.00	2870.20	1862.50
1.25	760.35	1010.09	386.19	1165.58	0.00	3322.21	2156.63
1.3	863.10	1149.29	438.38	1323.45	0.00	3774.22	2450.77
1.35	924.75	1230.59	469.69	1401.05	0.00	4026.08	2625.03
1.4	986.40	1311.89	501.00	1478.65	0.00	4277.94	2799.29
1.45	1006.95	1338.07	511.44	1500.86	0.00	4357.31	2856.45
1.5	1027.50	1364.24	521.88	1523.07	0.00	4436.68	2913.62
1.55	1006.95	1338.07	511.44	1500.86	0.00	4357.31	2856.45
1.6	986.40	1311.89	501.00	1478.65	0.00	4277.94	2799.29
1.65	924.75	1230.59	469.69	1401.05	0.00	4026.08	2625.03
1.7	863.10	1149.29	438.38	1323.45	0.00	3774.22	2450.77
1.75	760.35	1010.09	386.19	1165.58	0.00	3322.21	2156.63
1.8	657.60	870.90	334.00	1007.71	0.00	2870.20	1862.50
1.85	513.75	679.35	260.94	790.16	0.00	2244.20	1454.04
1.9	369.90	487.80	187.88	572.62	0.00	1618.20	1045.57
1.95	184.95	243.90	93.94	286.31	0.00	809.10	522.79
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00

full

Strength I loads (moment)

Maximum 1.25*DW + 1.5*DW + 1.75*(LL + IM)
Minimum 0.9*DC + 0.65*DW + 1.75*(LL + IM)

The loads shown in the DL columns reflect the values from Service I. The appropriate load combination (max or min) is shown in the total loads columns. The minimum load factors for dead load are used when dead load and future wearing survace stresses are of opposite sign to that of the live load.

$$\begin{pmatrix} STI1 \\ STI2 \\ STI3 \\ STI4 \\ STI5 \\ STI6 \\ STI7 \end{pmatrix} :=$$

	STI1	STI2	STI3	STI4	STI5	STI6	STI7
	DC LOADS (non-comp)		DW Loads	LL + I		TOTAL LOADS	
LOCATION	self wt	other (slab)	(comp)	M (+)	M (-)	M (+)	M (-)
1	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.05	184.95	243.90	93.94	357.89	0.00	1303.27	0.00
1.1	369.90	487.80	187.88	715.78	0.00	2606.55	0.00
1.15	513.75	679.35	260.94	987.70	0.00	3611.26	0.00
1.2	657.60	870.90	334.00	1259.63	0.00	4615.98	0.00
1.25	760.35	1010.09	386.19	1456.97	0.00	5342.04	0.00
1.3	863.10	1149.29	438.38	1654.31	0.00	6068.10	0.00
1.35	924.75	1230.59	469.69	1751.31	0.00	6463.50	0.00
1.4	986.40	1311.89	501.00	1848.31	0.00	6858.91	0.00
1.45	1006.95	1338.07	511.44	1876.07	0.00	6981.55	0.00
1.5	1027.50	1364.24	521.88	1903.83	0.00	7104.20	0.00
1.55	1006.95	1338.07	511.44	1876.07	0.00	6981.55	0.00
1.6	986.40	1311.89	501.00	1848.31	0.00	6858.91	0.00
1.65	924.75	1230.59	469.69	1751.31	0.00	6463.50	0.00
1.7	863.10	1149.29	438.38	1654.31	0.00	6068.10	0.00
1.75	760.35	1010.09	386.19	1456.97	0.00	5342.04	0.00
1.8	657.60	870.90	334.00	1259.63	0.00	4615.98	0.00
1.85	513.75	679.35	260.94	987.70	0.00	3611.26	0.00
1.9	369.90	487.80	187.88	715.78	0.00	2606.55	0.00
1.95	184.95	243.90	93.94	357.89	0.00	1303.27	0.00
2	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00

full

Service I loads (shear)

$$\begin{pmatrix} fullv \\ SI1v \\ SI2v \\ SI3v \\ SI4v \\ SI5v \\ SI6v \\ SI7v \end{pmatrix} :=$$

	SI1	SI2	SI3	SI4	SI5	SI5	SI7
	DC LOADS (non-comp)		DW Loads	LL + I		TOTAL LOADS	
LOCATION	self wt	other (slab)	(comp)	V (+)	V (-)	V (+)	V (-)
1	41.10	54.02	20.88	97.73	0.00	213.72	115.99
1.05	36.99	48.78	18.79	91.62	-2.44	196.18	102.11
1.1	32.88	43.54	16.70	85.51	-4.89	178.64	88.24
1.15	28.77	38.31	14.61	79.41	-8.14	161.10	73.55
1.2	24.66	33.07	12.53	73.30	-11.40	143.56	58.86
1.25	20.55	27.84	10.44	67.19	-15.07	126.02	43.76
1.3	16.44	22.60	8.35	61.08	-18.73	108.48	28.66
1.35	12.33	16.54	6.26	55.38	-23.62	90.51	11.51
1.4	8.22	10.47	4.18	49.68	-28.50	72.54	-5.64
1.45	4.11	5.24	2.09	44.39	-33.80	55.82	-22.37
1.5	0.00	0.00	0.00	39.09	-39.09	39.09	-39.09
1.55	-4.11	-5.24	-2.09	33.80	-44.39	22.37	-55.82
1.6	-8.22	-10.47	-4.18	28.50	-49.68	5.64	-72.54
1.65	-12.33	-16.54	-6.26	23.62	-55.38	-11.51	-90.51
1.7	-16.44	-22.60	-8.35	18.73	-61.08	-28.66	-108.48
1.75	-20.55	-27.84	-10.44	14.66	-67.19	-44.17	-126.02
1.8	-24.66	-33.07	-12.53	10.59	-73.30	-59.67	-143.56
1.85	-28.77	-38.31	-14.61	7.74	-79.41	-73.96	-161.10
1.9	-32.88	-43.54	-16.70	4.89	-85.51	-88.24	-178.64
1.95	-36.99	-48.78	-18.79	2.44	-91.62	-102.11	-196.18
2	-41.10	-54.02	-20.88	0.00	-97.73	-115.99	-213.72

ldv_f

Service III loads (shear)

$$\begin{pmatrix} SIII1v \\ SIII2v \\ SIII3v \\ SIII4v \\ SIII5v \\ SIII6v \\ SIII7v \end{pmatrix} :=$$

	SIII1	SIII2	SIII3	SIII4	SIII5	SIII5	SIII7	
	DC LOADS (non-comp)		DW Loads	LL + I		TOTAL LOADS		
LOCATION	self wt	other (slab)	(comp)	V (+)	V (-)	V (+)	V (-)	
1	41.10	54.02	20.88	78.18	0.00	194.17	115.99	
1.05	36.99	48.78	18.79	73.30	-1.95	177.86	102.60	
1.1	32.88	43.54	16.70	68.41	-3.91	161.54	89.22	
1.15	28.77	38.31	14.61	63.52	-6.52	145.22	75.18	
1.2	24.66	33.07	12.53	58.64	-9.12	128.90	61.14	
1.25	20.55	27.84	10.44	53.75	-12.05	112.58	46.77	
1.3	16.44	22.60	8.35	48.87	-14.99	96.26	32.41	
1.35	12.33	16.54	6.26	44.30	-18.89	79.43	16.24	
1.4	8.22	10.47	4.18	39.74	-22.80	62.61	0.06	
1.45	4.11	5.24	2.09	35.51	-27.04	46.94	-15.61	
1.5	0.00	0.00	0.00	31.27	-31.27	31.27	-31.27	
1.55	-4.11	-5.24	-2.09	27.04	-35.51	15.61	-46.94	
1.6	-8.22	-10.47	-4.18	22.80	-39.74	-0.06	-62.61	
1.65	-12.33	-16.54	-6.26	18.89	-44.30	-16.24	-79.43	
1.7	-16.44	-22.60	-8.35	14.99	-48.87	-32.41	-96.26	
1.75	-20.55	-27.84	-10.44	11.73	-53.75	-47.10	-112.58	
1.8	-24.66	-33.07	-12.53	8.47	-58.64	-61.79	-128.90	
1.85	-28.77	-38.31	-14.61	6.19	-63.52	-75.50	-145.22	
1.9	-32.88	-43.54	-16.70	3.91	-68.41	-89.22	-161.54	
1.95	-36.99	-48.78	-18.79	1.95	-73.30	-102.60	-177.86	
2	-41.10	-54.02	-20.88	0.00	-78.18	-115.99	-194.17	
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00	

fullv

Strength I loads (shear)

Maximum 1.25*DC + 1.5*DW + 1.75*(LL + IM)
Minimum 0.9*DC + 0.65*DW + 1.75*(LL + IM)

The loads shown in the DL columns reflect the values from Service I. The appropriate load combination (max or min) is shown in the total loads columns. The minimum load factors for dead load are used when dead load and future wearing surface stresses are of opposite sign to that of the live load.

$$\begin{pmatrix} STI1v \\ STI2v \\ STI3v \\ STI4v \\ STI5v \\ STI6v \\ STI7v \end{pmatrix} :=$$

	STI1	STI2	STI3	STI4	STI5	STI6	STI7
	DC LOADS (non-comp)		DW Loads	LL + I		TOTAL LOADS	
LOCATION	self wt	other (slab)	(comp)	V (+)	V (-)	V (+)	V (-)
1	41.10	54.02	20.88	97.73	0.00	321.23	0.00
1.05	36.99	48.78	18.79	91.62	-2.44	295.73	85.13
1.1	32.88	43.54	16.70	85.51	-4.89	270.23	71.09
1.15	28.77	38.31	14.61	79.41	-8.14	244.73	55.62
1.2	24.66	33.07	12.53	73.30	-11.40	219.23	40.15
1.25	20.55	27.84	10.44	67.19	-15.07	193.73	23.97
1.3	16.44	22.60	8.35	61.08	-18.73	168.22	7.79
1.35	12.33	16.54	6.26	55.38	-23.62	142.39	-11.28
1.4	8.22	10.47	4.18	49.68	-28.50	116.56	-30.35
1.45	4.11	5.24	2.09	44.39	-33.80	92.49	-49.38
1.5	0.00	0.00	0.00	39.09	-39.09	68.41	-68.41
1.55	-4.11	-5.24	-2.09	33.80	-44.39	49.38	-92.49
1.6	-8.22	-10.47	-4.18	28.50	-49.68	30.35	-116.56
1.65	-12.33	-16.54	-6.26	23.62	-55.38	11.28	-142.39
1.7	-16.44	-22.60	-8.35	18.73	-61.08	-7.79	-168.22
1.75	-20.55	-27.84	-10.44	14.66	-67.19	-24.68	-193.73
1.8	-24.66	-33.07	-12.53	10.59	-73.30	-41.57	-219.23
1.85	-28.77	-38.31	-14.61	7.74	-79.41	-56.33	-244.73
1.9	-32.88	-43.54	-16.70	4.89	-85.51	-71.09	-270.23
1.95	-36.99	-48.78	-18.79	2.44	-91.62	-85.13	-295.73
2	-41.10	-54.02	-20.88	0.00	-97.73	0.00	-321.23
0	0.00	0.00	0.00	0.00	0.00	0.00	0.00

fullv

Estimate number of Required Strands

Final stress in bottom (no pre-stress) (ksi) =
(I will use the moments at mid point)

$$fa := \max\left[\frac{(SI1 + SI2)\cdot 12}{Sb} + \frac{(SI3 + SI4)\cdot 12}{Sbc}\right] \qquad fa = 4.326$$

Required stress from pre-stress (ksi) =

$$f\_reqd := fa - 0.19\cdot\sqrt{fcf} \qquad f\_reqd = 3.789$$

Approximate force per strand (k) =
(estimate 42 ksi loss)

$$F\_est := Strand\_area\cdot(0.75\cdot Strand\_strength - 42) \qquad F\_est = 34.828$$

Approximate number of strands required =

$$N1 := \frac{f\_reqd}{F\_est\cdot\left(\frac{1}{Area} + \frac{yb - 4}{Sb}\right)} \qquad N1 = 33.642$$

The area below gives the proper data for strand pattern for the proper beam.

End Strand pattern

Input Strand pattern at end (only fill in the columns in red)
If the user wants to cut strands in the middle (break bond in middle) input a "y" in the column middle break and enter the number of strands broken, and the distance from center on each side.

Will harped strands be used ("y" or "n") = harped := "n"

$$\begin{pmatrix} mb \\ br \\ act \\ sn \\ stt \end{pmatrix} :=$$

max	Row		actual	middle	BREAK NO. 1		BREAK NO. 2		BREAK NO. 3		harp.
per row	Height		strands	break	strands	dist	strands	dist	strands	dist	
12	2		12		4	15					
12	4		12		2	15					
12	6		10								
10	8										
8	10										
6	12										
4	14										
2	16										
2	18										
2	20										
2	22										
2	35										
2	37										
2	39										
2	41										
2	43										
2	45										
2	47										
2	49		2	y	2	25					
2	51		2	y	2	25					

(x_s x_s1)

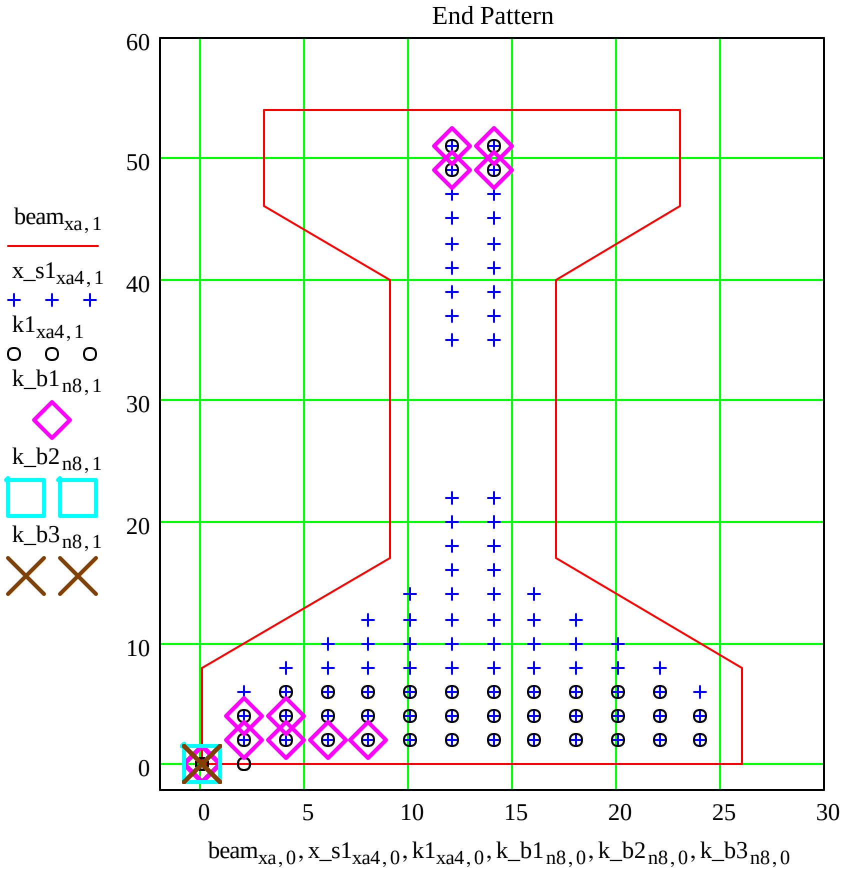
Middle Strand pattern

Input Strand pattern at middle (only fill in the columns in red), do not input middle break here.

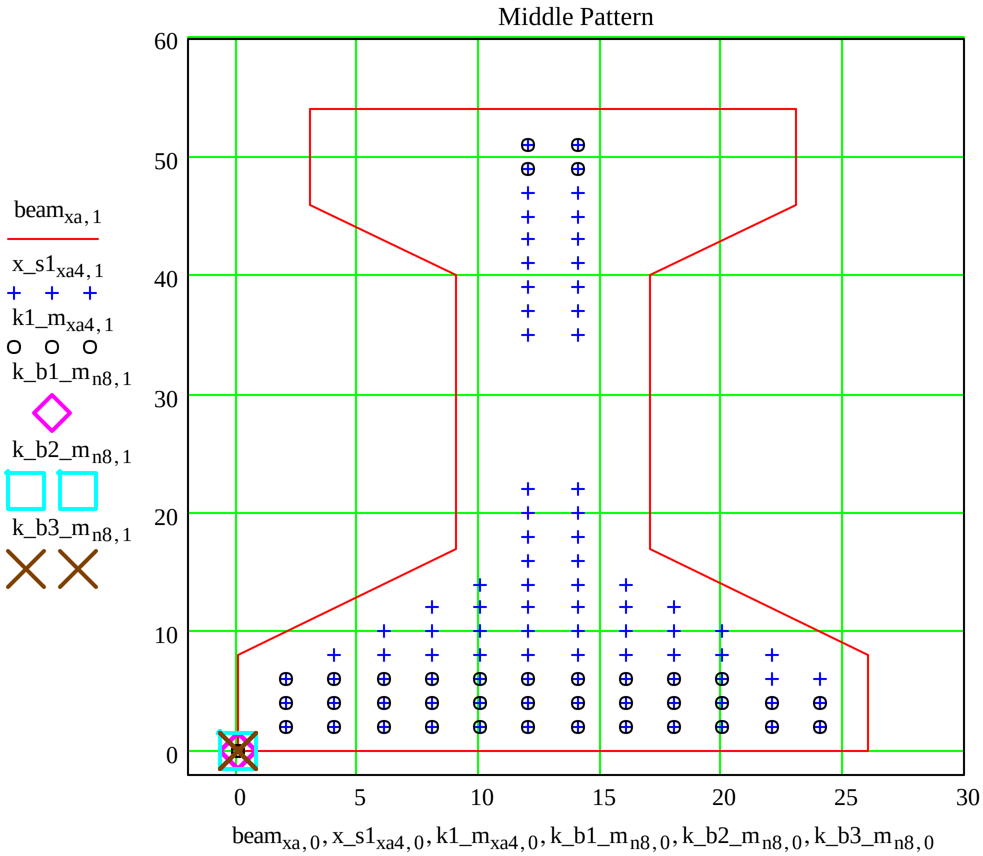
$$\begin{pmatrix} x\_u1\_m \\ br\_m \\ act\_m \\ sn\_m \end{pmatrix} :=$$

max strands per row	Row Height		actual strands								Harped "y" or "n"
12	2		12								n
12	4		12								n
12	6		10								n
10	8										
8	10										
6	12										
4	14										
2	16										
2	18										
2	20										
2	22										
2	35										
2	37										
2	39										
2	41										
2	43										
2	45										
2	47										
2	49		2								n
2	51		2								n

$(x\_s \;\; x\_s1)$

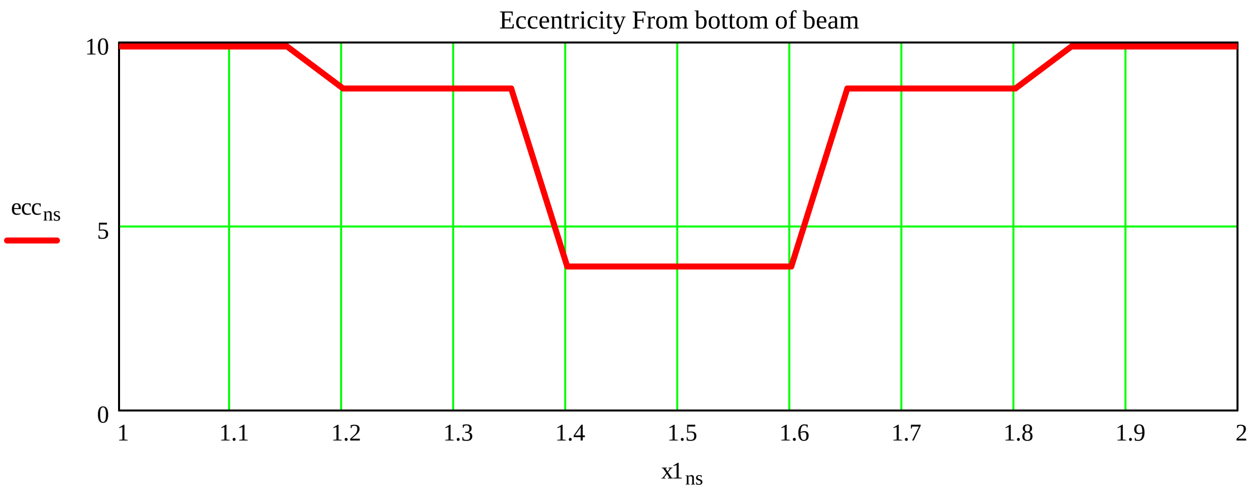


Do not worry about if the "x" coordinate of a strand or a strand break is not correct. The critical thing is that the "y" coordinate is correct.



Do not worry about if the "x" coordinate of a strand or a strand break is not correct. The critical thing is that the "y" coordinate is correct.

Calculate Eccentricity



The graph indicates values from the bottom of beam

$ecc_{mp} = 3.882$

Eccentricity for non-composite section

$enc_{ns} := yb - ecc_{ns}$

$enc_{mp} = 20.848$

$disp10_{ns,0} := x1_{ns}$ $disp10_{ns,1} := ecc_{ns}$ $disp10_{ns,2} := enc_{ns}$

disp10 =

	0	1	2
0	1	9.875	14.855
1	1.05	9.875	14.855
2	1.1	9.875	14.855
3	1.15	9.875	14.855
4	1.2	8.737	15.993
5	1.25	8.737	15.993
6	1.3	8.737	15.993
7	1.35	8.737	15.993
8	1.4	3.882	20.848
9	1.45	3.882	20.848
10	1.5	3.882	20.848
11	1.55	3.882	20.848
12	1.6	3.882	20.848
13	1.65	8.737	15.993
14	1.7	8.737	15.993
15	1.75	8.737	15.993
16	1.8	8.737	15.993
17	1.85	9.875	14.855
18	1.9	9.875	14.855
19	1.95	9.875	14.855
20	2	9.875	14.855

column 0 = span point
column 1 = eccentricity from bottom of beam
column 2 = eccentricity of non-comp section

Prestress Losses LRFD 5.9.5.1

The total loses shall be the sum of elastis shortening (ES) + shrinkage (SR) + Creep (CR) + Relaxation (R2)

Elastic Shortening LRFD 5.9.5.2.3a

Initial modulus of elasticity for concrete $Eci := 33000 \cdot 0.15^{1.5} \cdot \sqrt{fci}$ $Eci = 4695.982$

Modulus of elasticity for the strands $Ep := 28500$

Number of strands at middle $Ns\_middle := ye4_{ceil\left(\frac{sp}{2}\right)}$ $Ns\_middle = 34$

Area of pre-stress strands $Aps\_1 := Ns\_middle \cdot Strand\_area$ $Aps\_1 = 7.378$

Total force in strands $Fs := Strand\_strength \cdot Aps\_1 \cdot 0.75$ $Fs = 1494.045$

Moment from beam alone (k*ft) $M\_1 := max(SI1)$ $M\_1 = 1027.5$

Area of concrete (in^2) = $Ac := Area$ $Ac = 789$

Eccentricity of strands for N.A. non-composite $e\_2 := yb - ecc_{ceil\left(\frac{sp}{2}\right)}$ $e\_2 = 20.848$

fcgp (stress @ c.g. strands from prestress and beam weight only)

at bottom $fbi := \frac{Fs}{Ac} + \frac{Fs \cdot e\_2}{Sb} - \frac{M\_1 \cdot 12}{Sb}$ $fbi = 3.678$

at top $fti := \frac{Fs}{Ac} - \frac{Fs \cdot e\_2}{St} + \frac{M\_1 \cdot 12}{St}$ $fti = -0.219$

$fcgp := fbi + \frac{(fti - fbi) \cdot (yb - e\_2 - 0)}{h}$ $fcgp = 3.398$

Elastic shortenting (ksi) = $ES := \frac{Ep}{Eci} \cdot fcgp$ $ES = 20.624$

Shrinkage LRFD 5.9.5.4.2

Pretensioned members shrinkage = 17-0.15*H

H can be abtained from figure 5.4.2.3.3-1

$H := 70$

$SR := 17 - 0.15 \cdot H$ $\quad SR = 6.5$

Creep of concrete LRFD 5.9.5.4.3

For pre-tensioned members creep = 12*fcgp - 7*fcdp >= 0

fcdp = stress at c.g. strands from all permanent loads, except the loads used in fcgp

at bottom $\quad fb := \frac{-SI2_7 \cdot 12}{Sb} - \frac{(SI3_7 + SI4_7) \cdot 12}{Sbc}$ $\quad fb = -2.869$

at top $\quad ft := \frac{SI2_7 \cdot 12}{St} + \frac{(SI3_7 + SI4_7) \cdot 12}{Stb}$ $\quad ft = 2.149$

$fcdp := fb + \frac{(ft - fb) \cdot (yb - e\_2 - 0)}{h}$ $\quad fcdp = -2.508$

Creep (ksi) = $\quad CR := 12 \cdot fcgp - 7 \cdot |fcdp|$ $\quad CR = 23.219$

Relaxation at Transfer (fpr1) LRFD 5.9.5.4.4b

Time of transfer (18 hours) = 0.75 days $\quad time := 0.75$

Yield strength of tendons (ksi) = $\quad fpy := Strand\_strength$ $\quad fpy = 270$

Initial stress in tendon (ksi) = $\quad fpj := 0.75 \cdot fpy$

Relaxation at transfer = $\quad R1 := \frac{\log(24 \cdot time)}{40} \cdot \left(\frac{fpj}{fpy} - 0.55\right) \cdot fpj$ $\quad R1 = 1.271$

Relaxation after Transfer LRFD 5.9.5.4.4c

$\Delta fpR2$ = 0.30 * (the value from formula 5.9.5.4.4c-1) for Low Lax strands

$R2 := 0.3\cdot[20 - 0.4\cdot ES - 0.2\cdot(SR + CR)]$ $R2 = 1.742$

Total Initial Loses

$\Delta fi := R1 + ES$ $\Delta fi = 21.895$

Total Final Loses

$\Delta ft := ES + SR + CR + R2$ $\Delta ft = 52.085$

Total Initial stress in the strands with Initial Losses $fi := 0.75\cdot Strand\_strength - \Delta fi$ $fi = 180.605$

Total Final stress in the strands with Final Loses $ff := 0.75\cdot Strand\_strength - \Delta ft$ $ff = 150.415$

Design variables

Eccentricity for non-compite section $enc_{ns} := yb - ecc_{ns}$ $enc_{mp} = 20.848$

Effective strand area (including bond break) in^2 = $Aps_{ns} := ye4_{ns} \cdot Strand\_area$ $Aps_{mp} = 7.378$

Number of effective strends = $ye4_{mp} = 34$

Initial Strand force (k) = $Fi_{ns} := Aps_{ns} \cdot fi$ $Fi_{mp} = 1332.506$

Final Strand force (k) = $Ff_{ns} := Aps_{ns} \cdot ff$ $Ff_{mp} = 1109.764$

$disp := 0$ $disp_{ns,1} := enc_{ns}$ $disp_{ns,3} := ye4_{ns}$ $disp_{ns,5} := SI1_{ns}$ $disp_{ns,7} := Ff_{ns}$

$disp_{ns,0} := x1_{ns}$ $disp_{ns,2} := 0$ $disp_{ns,4} := Aps_{ns}$ $disp_{ns,6} := Fi_{ns}$

column 0 = span point
column 1 = eccentricity of strands on non-compostie section
column 2 = blank
column 3 = number of effective strands
column 4 = Area of prestressing strands at a given point
column 5 = Service I moment
column 6 = initial force
column 7 = final force

$disp =$

	0	1	2	3	4	5	6	7
0	1	14.855	0	32	6.944	0	1254.123	1044.483
1	1.05	14.855	0	32	6.944	184.95	1254.123	1044.483
2	1.1	14.855	0	32	6.944	369.9	1254.123	1044.483
3	1.15	14.855	0	32	6.944	513.75	1254.123	1044.483
4	1.2	15.993	0	38	8.246	657.6	1489.271	1240.324
5	1.25	15.993	0	38	8.246	760.35	1489.271	1240.324
6	1.3	15.993	0	38	8.246	863.1	1489.271	1240.324
7	1.35	15.993	0	38	8.246	924.75	1489.271	1240.324
8	1.4	20.848	0	34	7.378	986.4	1332.506	1109.764
9	1.45	20.848	0	34	7.378	1006.95	1332.506	1109.764
10	1.5	20.848	0	34	7.378	1027.5	1332.506	1109.764
11	1.55	20.848	0	34	7.378	1006.95	1332.506	1109.764
12	1.6	20.848	0	34	7.378	986.4	1332.506	1109.764
13	1.65	15.993	0	38	8.246	924.75	1489.271	1240.324
14	1.7	15.993	0	38	8.246	863.1	1489.271	1240.324
15	1.75	15.993	0	38	8.246	760.35	1489.271	1240.324
16	1.8	15.993	0	38	8.246	657.6	1489.271	1240.324

16	1.8	15.993	0	38	8.246	657.6	1489.271	1240.324
17	1.85	14.855	0	32	6.944	513.75	1254.123	1044.483
18	1.9	14.855	0	32	6.944	369.9	1254.123	1044.483
19	1.95	14.855	0	32	6.944	184.95	1254.123	1044.483
20	2	14.855	0	32	6.944	0	1254.123	1044.483

Stress at initial conditions

This includes the beam weight and the pre-stress force only.

Initial strand force (lb) = $Fi_{ns} := Aps_{ns} \cdot fi$ $\quad Fi_{mp} = 1332.506$

Initial stress top (psi) = $top\_i_{ns} := \dfrac{Fi_{ns}}{Ac} - \dfrac{Fi_{ns} \cdot enc_{ns}}{St} + \dfrac{SI1_{ns} \cdot 12}{St}$ $\quad top\_i_{mp} = -0.046$

Initial stress in bottom (psi) = $bot\_i_{ns} := \dfrac{Fi_{ns}}{Ac} + \dfrac{Fi_{ns} \cdot enc_{ns}}{Sb} - \dfrac{SI1_{ns} \cdot 12}{Sb}$ $\quad bot\_i_{mp} = 3.154$

Pass fail condition = $check\_i_{ns,0} := x1_{ns}$

$$check\_i_{ns,3} := \begin{cases} \text{"top fail"} & \text{if } (top\_i_{ns} < 0) \cdot (top\_i_{ns} < fit) \\ \text{otherwise} & \begin{cases} \text{"top fail"} & \text{if } (top\_i_{ns} > 0) \cdot (top\_i_{ns} > fic) \\ \text{"top OK"} & \text{otherwise} \end{cases} \end{cases}$$

$$check\_i_{ns,6} := \begin{cases} \text{"bot fail"} & \text{if } (bot\_i_{ns} < 0) \cdot (bot\_i_{ns} < fit) \\ \text{otherwise} & \begin{cases} \text{"bot fail"} & \text{if } (bot\_i_{ns} > 0) \cdot (bot\_i_{ns} > fic) \\ \text{"bot OK"} & \text{otherwise} \end{cases} \end{cases}$$

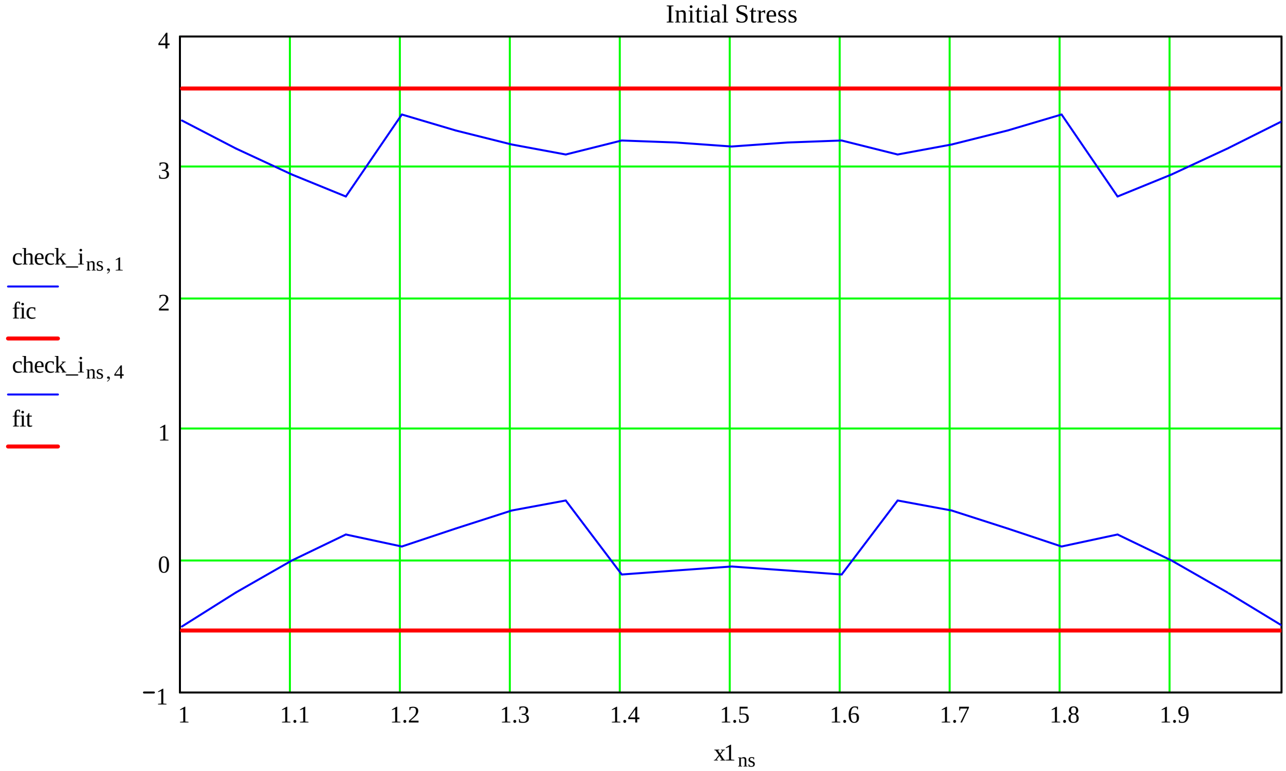
$check\_i_{ns,1} := top\_i_{ns}$ $\qquad check\_i_{ns,4} := bot\_i_{ns}$

$$check\_i_{ns,2} := \begin{cases} fit & \text{if } top\_i_{ns} < 0 \\ fic & \text{otherwise} \end{cases} \qquad check\_i_{ns,5} := \begin{cases} fit & \text{if } bot\_i_{ns} < 0 \\ fic & \text{otherwise} \end{cases}$$

check_i =

	0	1	2	3	4	5	6
0	1	-0.502	-0.539	"top OK"	3.357	3.6	"bot OK"
1	1.05	-0.253	-0.539	"top OK"	3.146	3.6	"bot OK"
2	1.1	-0.004	-0.539	"top OK"	2.936	3.6	"bot OK"
3	1.15	0.19	3.6	"top OK"	2.772	3.6	"bot OK"
4	1.2	0.1	3.6	"top OK"	3.398	3.6	"bot OK"
5	1.25	0.238	3.6	"top OK"	3.281	3.6	"bot OK"
6	1.3	0.376	3.6	"top OK"	3.164	3.6	"bot OK"
7	1.35	0.459	3.6	"top OK"	3.094	3.6	"bot OK"
8	1.4	-0.101	-0.539	"top OK"	3.201	3.6	"bot OK"
9	1.45	-0.073	-0.539	"top OK"	3.178	3.6	"bot OK"
10	1.5	-0.046	-0.539	"top OK"	3.154	3.6	"bot OK"
11	1.55	-0.073	-0.539	"top OK"	3.178	3.6	"bot OK"
12	1.6	-0.101	-0.539	"top OK"	3.201	3.6	"bot OK"
13	1.65	0.459	3.6	"top OK"	3.094	3.6	"bot OK"
14	1.7	0.376	3.6	"top OK"	3.164	3.6	"bot OK"
15	1.75	0.238	3.6	"top OK"	3.281	3.6	"bot OK"
16	1.8	0.1	3.6	"top OK"	3.398	3.6	"bot OK"
17	1.85	0.19	3.6	"top OK"	2.772	3.6	"bot OK"
18	1.9	-0.004	-0.539	"top OK"	2.936	3.6	"bot OK"
19	1.95	-0.253	-0.539	"top OK"	3.146	3.6	"bot OK"
20	2	-0.502	-0.539	"top OK"	3.357	3.6	"bot OK"

column 0 = span point
column 1 = top stress
column 2 = top allowable
column 3 = top check
column 4 = bottom stress
column 5 = bottom allowable
column 6 = bottom check



Positive moment envelope at Service I

Final stress top (psi) = $SI\_pt_{ns} := \frac{Ff_{ns}}{Ac} - \frac{Ff_{ns} \cdot enc_{ns}}{St} + \frac{(SI1_{ns} + SI2_{ns}) \cdot 12}{St} + \frac{(SI3_{ns} + SI4_{ns}) \cdot 12}{Stb}$ $SI\_pt_{mp} = 2.568$

Final stress in bottom (psi) =

$$SI\_pb_{ns} := \frac{Ff_{ns}}{Ac} + \frac{Ff_{ns} \cdot enc_{ns}}{Sb} - \frac{(SI1_{ns} + SI2_{ns}) \cdot 12}{Sb} - \frac{(SI3_{ns} + SI4_{ns}) \cdot 12}{Sbc} \qquad SI\_pb_{mp} = -0.725$$

Pass fail condition =

From LRFD 5.9.4.2.1 under final conditions it is only necessary to check tension under service III load combinations.
If the flag "see SIII" is shown in the tables below see the section with service III loads for tension checks.

$$check\_1_{ns,0} := x1_{ns}$$

$$check\_1_{ns,3} := \begin{cases} \text{"see SIII"} & \text{if } \left(SI\_pt_{ns} < 0\right) \\ \text{otherwise} & \\ \quad \begin{cases} \text{"top fail"} & \text{if } \left(SI\_pt_{ns} > 0\right)\cdot\left(SI\_pt_{ns} > fc1\right) \\ \text{"top OK"} & \text{otherwise} \end{cases} & \end{cases}$$

$$check\_1_{ns,6} := \begin{cases} \text{"see SIII"} & \text{if } \left(SI\_pb_{ns} < 0\right) \\ \text{otherwise} & \\ \quad \begin{cases} \text{"bot fail"} & \text{if } \left(SI\_pb_{ns} > 0\right)\cdot\left(SI\_pb_{ns} > fc1\right) \\ \text{"bot OK"} & \text{otherwise} \end{cases} & \end{cases}$$

$$check\_1_{ns,1} := SI\_pt_{ns} \qquad check\_1_{ns,4} := SI\_pb_{ns}$$

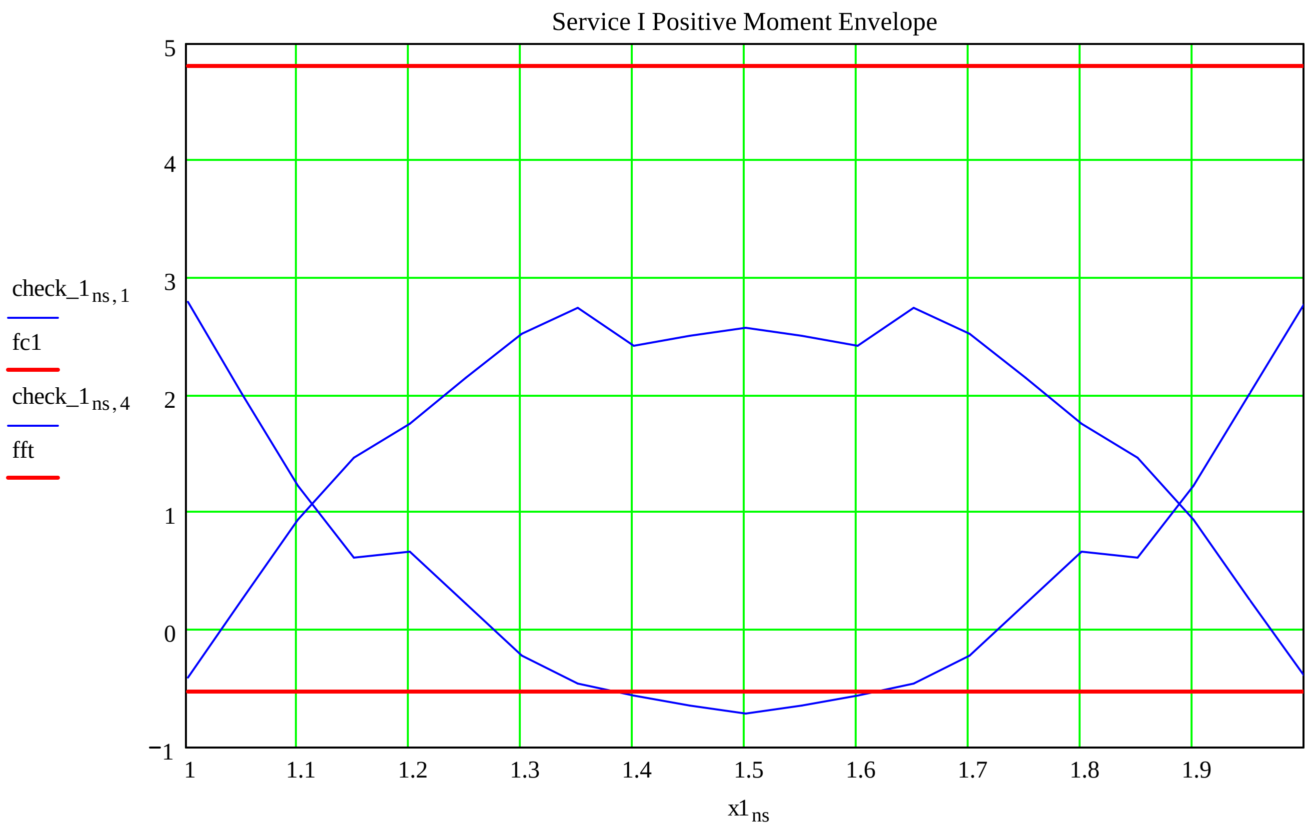
$$check\_1_{ns,2} := \begin{cases} fft & \text{if } SI\_pt_{ns} < 0 \\ fc1 & \text{otherwise} \end{cases} \qquad check\_1_{ns,5} := \begin{cases} fft & \text{if } SI\_pb_{ns} < 0 \\ fc1 & \text{otherwise} \end{cases}$$

check_1 =

	0	1	2	3	4	5	6
0	1	-0.418	-0.537	"see SIII"	2.795	4.8	"bot OK"
1	1.05	0.26	4.8	"top OK"	2.009	4.8	"bot OK"
2	1.1	0.937	4.8	"top OK"	1.222	4.8	"bot OK"
3	1.15	1.466	4.8	"top OK"	0.612	4.8	"bot OK"
4	1.2	1.757	4.8	"top OK"	0.66	4.8	"bot OK"
5	1.25	2.138	4.8	"top OK"	0.22	4.8	"bot OK"
6	1.3	2.519	4.8	"top OK"	-0.221	-0.537	"see SIII"
7	1.35	2.74	4.8	"top OK"	-0.468	-0.537	"see SIII"
8	1.4	2.425	4.8	"top OK"	-0.568	-0.537	"see SIII"
9	1.45	2.497	4.8	"top OK"	-0.647	-0.537	"see SIII"
10	1.5	2.568	4.8	"top OK"	-0.725	-0.537	"see SIII"
11	1.55	2.497	4.8	"top OK"	-0.647	-0.537	"see SIII"
12	1.6	2.425	4.8	"top OK"	-0.568	-0.537	"see SIII"
13	1.65	2.74	4.8	"top OK"	-0.468	-0.537	"see SIII"
14	1.7	2.519	4.8	"top OK"	-0.221	-0.537	"see SIII"
15	1.75	2.138	4.8	"top OK"	0.22	4.8	"bot OK"
16	1.8	1.757	4.8	"top OK"	0.66	4.8	"bot OK"
17	1.85	1.466	4.8	"top OK"	0.612	4.8	"bot OK"
18	1.9	0.937	4.8	"top OK"	1.222	4.8	"bot OK"
19	1.95	0.26	4.8	"top OK"	2.009	4.8	"bot OK"
20	2	0.418	0.537	"see SIII"	2.795	4.8	"bot OK"

column 0 = span point
column 1 = top stress
column 2 = top allowable
column 3 = top check
column 4 = bottom stress
column 5 = bottom allowable
column 6 = bottom check

20	2	-0.418	-0.537	see SIII	2.795	4.8	but OK



Negative moment envelope at Service I

Final stress top (psi) = $SI\_nt_{ns} := \dfrac{Ff_{ns}}{Ac} - \dfrac{Ff_{ns} \cdot enc_{ns}}{St} + \dfrac{(SI1_{ns} + SI2_{ns}) \cdot 12}{St} + \dfrac{(SI3_{ns} + SI5_{ns}) \cdot 12}{Stb}$ $SI\_nt_{mp} = 2.147$

Final stress in bottom (psi) =

$$SI\_nb_{ns} := \frac{Ff_{ns}}{Ac} + \frac{Ff_{ns} \cdot enc_{ns}}{Sb} - \frac{(SI1_{ns} + SI2_{ns}) \cdot 12}{Sb} - \frac{(SI3_{ns} + SI5_{ns}) \cdot 12}{Sbc} \qquad SI\_nb_{mp} = 0.534$$

Pass fail condition =

From LRFD 5.9.4.2.1 under final conditions it is only necessary to check tension under service III load combinations. If the flag "see SIII" is shown in the tables below see the section with service III loads for tension checks.

$$check\_1_{ns,0} := x1_{ns}$$

$$check\_1_{ns,3} := \left| \begin{array}{l} \text{"see SIII"} \quad \text{if } (SI\_nt_{ns} < 0) \\ \text{otherwise} \\ \quad \left| \begin{array}{l} \text{"top fail"} \quad \text{if } (SI\_nt_{ns} > 0) \cdot (SI\_nt_{ns} > fc1) \\ \text{"top OK"} \quad \text{otherwise} \end{array} \right. \end{array} \right.$$

$$check\_1_{ns,6} := \left| \begin{array}{l} \text{"see SIII"} \quad \text{if } (SI\_nb_{ns} < 0) \\ \text{otherwise} \\ \quad \left| \begin{array}{l} \text{"bot fail"} \quad \text{if } (SI\_nb_{ns} > 0) \cdot (SI\_nb_{ns} > fc1) \\ \text{"bot OK"} \quad \text{otherwise} \end{array} \right. \end{array} \right.$$

$$check\_1_{ns,1} := SI\_nt_{ns}$$

$$check\_1_{ns,4} := SI\_nb_{ns}$$

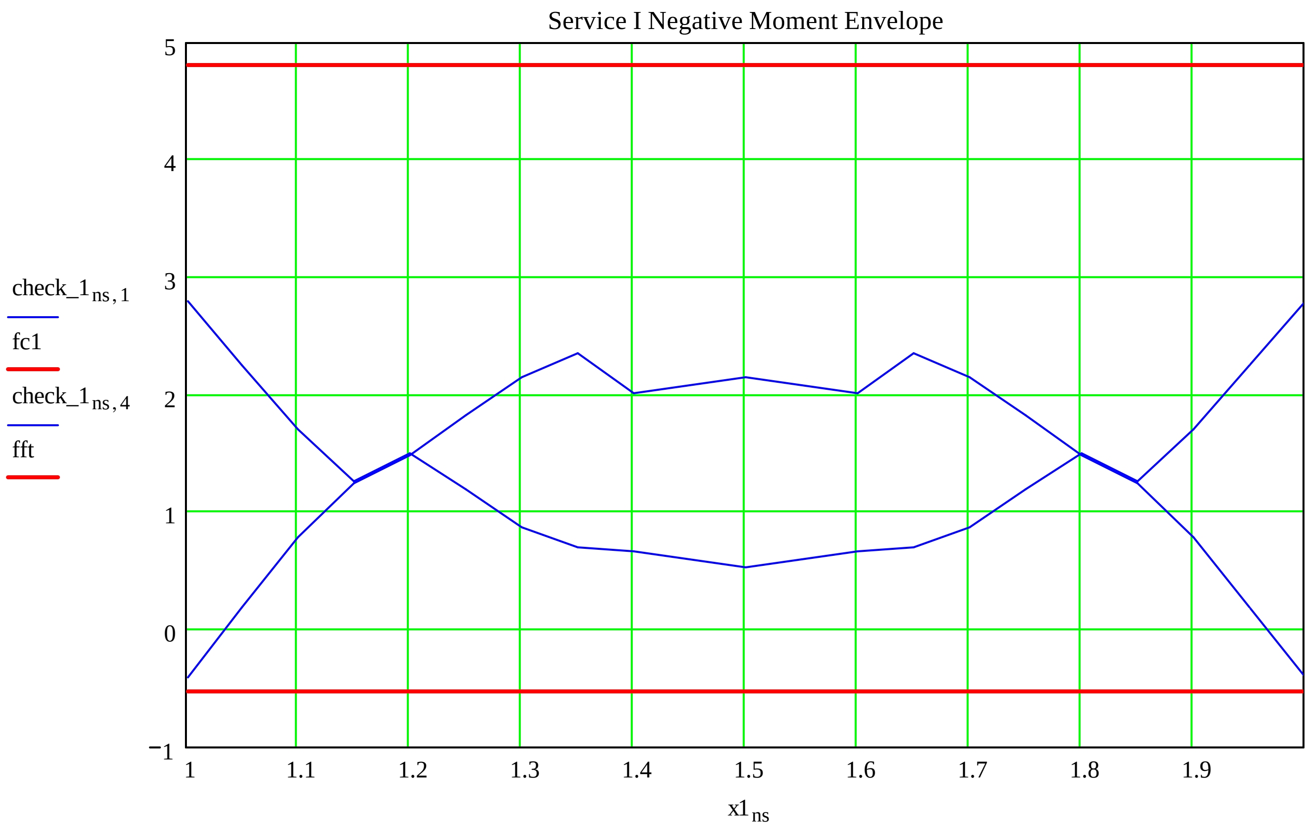
$$check\_1_{ns,2} := \left| \begin{array}{l} fft \quad \text{if } SI\_nt_{ns} < 0 \\ fc1 \quad \text{otherwise} \end{array} \right.$$

$$check\_1_{ns,5} := \left| \begin{array}{l} fft \quad \text{if } SI\_nb_{ns} < 0 \\ fc1 \quad \text{otherwise} \end{array} \right.$$

$check\_1 =$

	0	1	2	3	4	5	6
0	1	-0.418	-0.537	"see SIII"	2.795	4.8	"bot OK"
1	1.05	0.18	4.8	"top OK"	2.245	4.8	"bot OK"
2	1.1	0.779	4.8	"top OK"	1.695	4.8	"bot OK"
3	1.15	1.247	4.8	"top OK"	1.265	4.8	"bot OK"
4	1.2	1.478	4.8	"top OK"	1.493	4.8	"bot OK"
5	1.25	1.816	4.8	"top OK"	1.183	4.8	"bot OK"
6	1.3	2.153	4.8	"top OK"	0.873	4.8	"bot OK"
7	1.35	2.353	4.8	"top OK"	0.69	4.8	"bot OK"
8	1.4	2.016	4.8	"top OK"	0.654	4.8	"bot OK"
9	1.45	2.081	4.8	"top OK"	0.594	4.8	"bot OK"
10	1.5	2.147	4.8	"top OK"	0.534	4.8	"bot OK"
11	1.55	2.081	4.8	"top OK"	0.594	4.8	"bot OK"
12	1.6	2.016	4.8	"top OK"	0.654	4.8	"bot OK"
13	1.65	2.353	4.8	"top OK"	0.69	4.8	"bot OK"
14	1.7	2.153	4.8	"top OK"	0.873	4.8	"bot OK"
15	1.75	1.816	4.8	"top OK"	1.183	4.8	"bot OK"
16	1.8	1.478	4.8	"top OK"	1.493	4.8	"bot OK"
17	1.85	1.247	4.8	"top OK"	1.265	4.8	"bot OK"
18	1.9	0.779	4.8	"top OK"	1.695	4.8	"bot OK"
19	1.95	0.18	4.8	"top OK"	2.245	4.8	"bot OK"
20	2	-0.418	-0.537	"see SIII"	2.795	4.8	"bot OK"

column 0 = span point
column 1 = top stress
column 2 = top allowable
column 3 = top check
column 4 = bottom stress
column 5 = bottom allowable
column 6 = bottom check



Positive moment envelope at Service III

Final stress top (psi) = $$SIII\_pt_{ns} := \frac{Ff_{ns}}{Ac} - \frac{Ff_{ns} \cdot enc_{ns}}{St} + \frac{\left(SIII1_{ns} + SIII2_{ns}\right) \cdot 12}{St} + \frac{\left(SIII3_{ns} + SIII4_{ns}\right) \cdot 12}{Stb} \qquad SIII\_pt_{mp} = 2.484$$

Final stress in bottom (psi) =

$$SIII\_pb_{ns} := \frac{Ff_{ns}}{Ac} + \frac{Ff_{ns} \cdot enc_{ns}}{Sb} - \frac{\left(SIII1_{ns} + SIII2_{ns}\right) \cdot 12}{Sb} - \frac{\left(SIII3_{ns} + SIII4_{ns}\right) \cdot 12}{Sbc} \qquad SIII\_pb_{mp} = -0.474$$

Pass fail condition =

From LRFD 5.9.4.2.1 under final conditions it is only necessary to check tension under service III load combinations. If the flag "see SIII" is shown in the tables below see the section with service III loads for tension checks.

$$check\_1_{ns,0} := x1_{ns}$$

$$check\_1_{ns,3} := \begin{vmatrix} \text{"top fail"} \quad \text{if } (SIII\_pt_{ns} < 0)\cdot(SIII\_pt_{ns} < fft) \\ \text{otherwise} \\ \quad \begin{vmatrix} \text{"top fail"} \quad \text{if } (SIII\_pt_{ns} > 0)\cdot(SIII\_pt_{ns} > fc3) \\ \text{"top OK"} \quad \text{otherwise} \end{vmatrix} \end{vmatrix}$$

$$check\_1_{ns,6} := \begin{vmatrix} \text{"bot fail"} \quad \text{if } (SIII\_pb_{ns} < 0)\cdot(SIII\_pb_{ns} < fft) \\ \text{otherwise} \\ \quad \begin{vmatrix} \text{"bot fail"} \quad \text{if } (SIII\_pb_{ns} > 0)\cdot(SIII\_pb_{ns} > fc3) \\ \text{"bot OK"} \quad \text{otherwise} \end{vmatrix} \end{vmatrix}$$

$$check\_1_{ns,1} := SIII\_pt_{ns}$$

$$check\_1_{ns,4} := SIII\_pb_{ns}$$

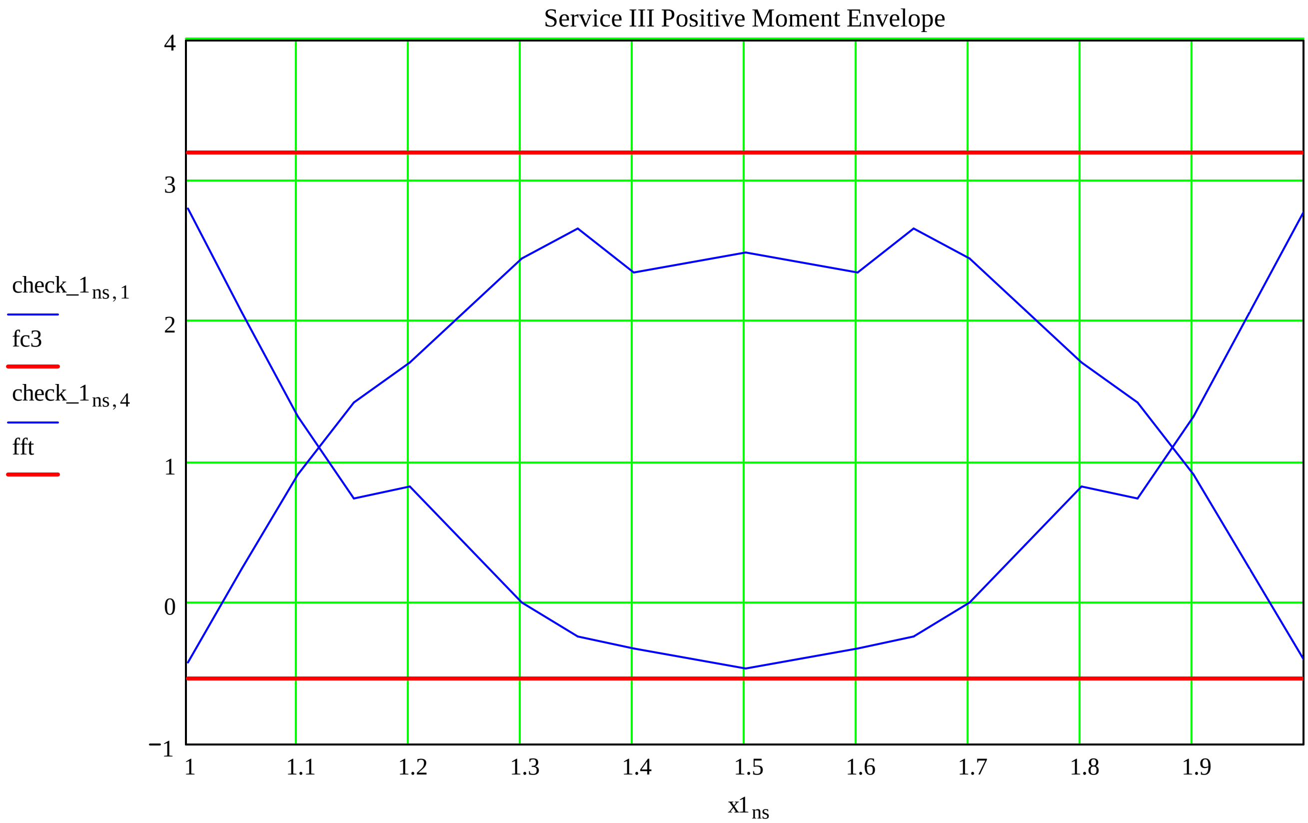
$$check\_1_{ns,2} := \begin{vmatrix} fft \quad \text{if } SIII\_pt_{ns} < 0 \\ fc3 \quad \text{otherwise} \end{vmatrix}$$

$$check\_1_{ns,5} := \begin{vmatrix} fft \quad \text{if } SIII\_pb_{ns} < 0 \\ fc3 \quad \text{otherwise} \end{vmatrix}$$

check_1 =

	0	1	2	3	4	5	6
0	1	-0.418	-0.537	"top OK"	2.795	3.2	"bot OK"
1	1.05	0.244	3.2	"top OK"	2.056	3.2	"bot OK"
2	1.1	0.906	3.2	"top OK"	1.316	3.2	"bot OK"
3	1.15	1.422	3.2	"top OK"	0.742	3.2	"bot OK"
4	1.2	1.701	3.2	"top OK"	0.827	3.2	"bot OK"
5	1.25	2.073	3.2	"top OK"	0.412	3.2	"bot OK"
6	1.3	2.446	3.2	"top OK"	-0.002	-0.537	"bot OK"
7	1.35	2.663	3.2	"top OK"	-0.237	-0.537	"bot OK"
8	1.4	2.343	3.2	"top OK"	-0.324	-0.537	"bot OK"
9	1.45	2.414	3.2	"top OK"	-0.399	-0.537	"bot OK"
10	1.5	2.484	3.2	"top OK"	-0.474	-0.537	"bot OK"
11	1.55	2.414	3.2	"top OK"	-0.399	-0.537	"bot OK"
12	1.6	2.343	3.2	"top OK"	-0.324	-0.537	"bot OK"
13	1.65	2.663	3.2	"top OK"	-0.237	-0.537	"bot OK"
14	1.7	2.446	3.2	"top OK"	-0.002	-0.537	"bot OK"
15	1.75	2.073	3.2	"top OK"	0.412	3.2	"bot OK"
16	1.8	1.701	3.2	"top OK"	0.827	3.2	"bot OK"
17	1.85	1.422	3.2	"top OK"	0.742	3.2	"bot OK"
18	1.9	0.906	3.2	"top OK"	1.316	3.2	"bot OK"
19	1.95	0.244	3.2	"top OK"	2.056	3.2	"bot OK"
20	2	-0.418	-0.537	"top OK"	2.795	3.2	"bot OK"

column 0 = span point
column 1 = top stress
column 2 = top allowable
column 3 = top check
column 4 = bottom stress
column 5 = bottom allowable
column 6 = bottom check



Negative moment envelope at Service III

Final stress top (psi) = $SIII\_nt_{ns} := \frac{Ff_{ns}}{Ac} - \frac{Ff_{ns} \cdot enc_{ns}}{St} + \frac{(SIII1_{ns} + SIII2_{ns}) \cdot 12}{St} + \frac{(SIII3_{ns} + SIII5_{ns}) \cdot 12}{Stb}$ $SIII\_nt_{mp} = 2.147$

Final stress in bottom (psi) =

$$SIII\_nb_{ns} := \frac{Ff_{ns}}{Ac} + \frac{Ff_{ns} \cdot enc_{ns}}{Sb} - \frac{(SIII1_{ns} + SIII2_{ns}) \cdot 12}{Sb} - \frac{(SIII3_{ns} + SIII5_{ns}) \cdot 12}{Sbc} \qquad SIII\_nb_{mp} = 0.534$$

Pass fail condition =

From LRFD 5.9.4.2.1 under final conditions it is only necessary to check tension under service III load combinations. If the flag "see SIII" is shown in the tables below see the section with service III loads for tension checks.

$$check\_1_{ns,0} := x1_{ns}$$

$$check\_1_{ns,3} := \begin{array}{|l} \text{"top fail"} \quad \text{if } (SIII\_nt_{ns} < 0)\cdot(SIII\_nt_{ns} < fft) \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"top fail"} \quad \text{if } (SIII\_nt_{ns} > 0)\cdot(SIII\_nt_{ns} > fc3) \\ \text{"top OK"} \quad \text{otherwise} \end{array} \end{array}$$

$$check\_1_{ns,6} := \begin{array}{|l} \text{"see SIII"} \quad \text{if } (SI\_nb_{ns} < 0) \\ \text{otherwise} \\ \quad \begin{array}{|l} \text{"bot fail"} \quad \text{if } (SI\_nb_{ns} > 0)\cdot(SI\_nb_{ns} > fc3) \\ \text{"bot OK"} \quad \text{otherwise} \end{array} \end{array}$$

$$check\_1_{ns,1} := SIII\_nt_{ns}$$

$$check\_1_{ns,4} := SI\_nb_{ns}$$

$$check\_1_{ns,2} := \begin{array}{|l} fft \quad \text{if } SIII\_nt_{ns} < 0 \\ fc3 \quad \text{otherwise} \end{array}$$

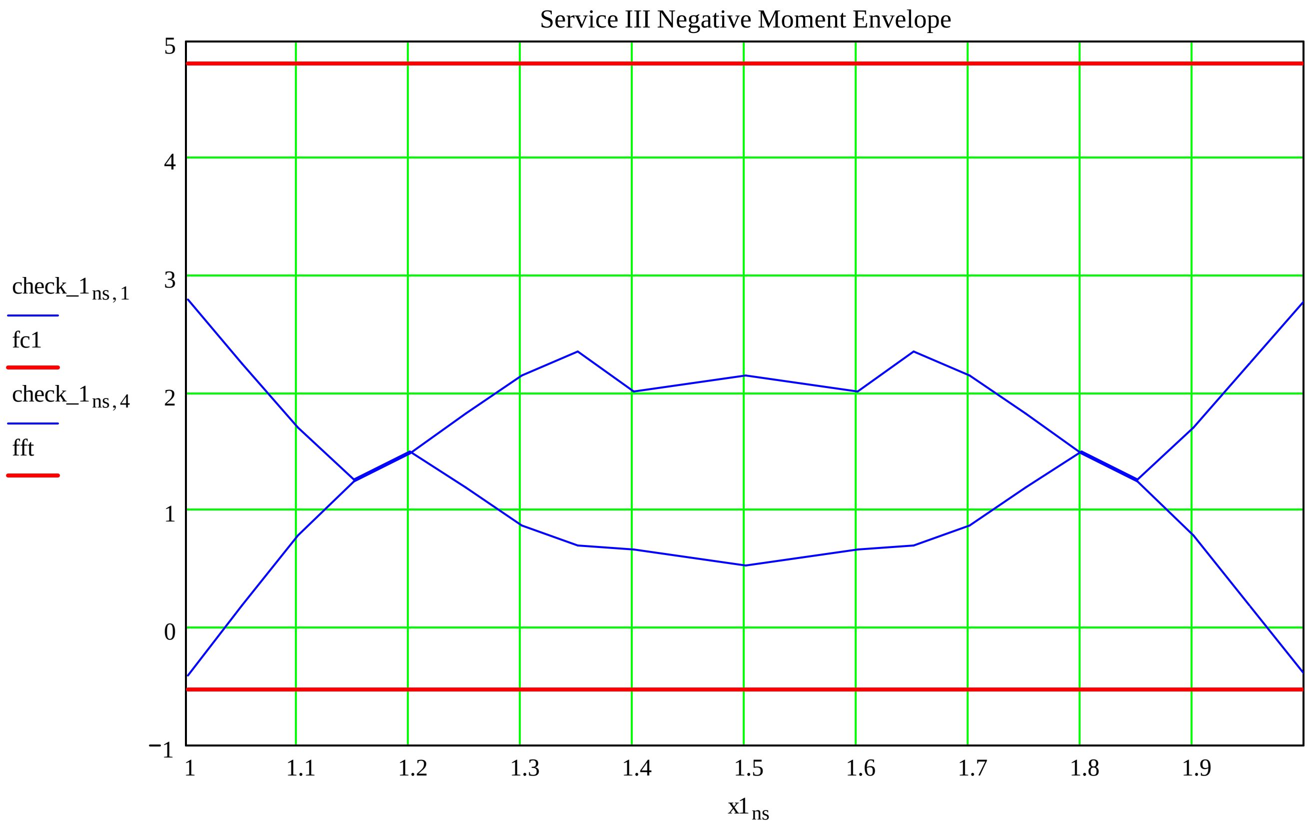
$$check\_1_{ns,5} := \begin{array}{|l} fft \quad \text{if } SIII\_nb_{ns} < 0 \\ fc3 \quad \text{otherwise} \end{array}$$

check_1 =

	0	1	2	3	4	5	6
0	1	-0.418	-0.537	"top OK"	2.795	3.2	"bot OK"
1	1.05	0.18	3.2	"top OK"	2.245	3.2	"bot OK"
2	1.1	0.779	3.2	"top OK"	1.695	3.2	"bot OK"
3	1.15	1.247	3.2	"top OK"	1.265	3.2	"bot OK"
4	1.2	1.478	3.2	"top OK"	1.493	3.2	"bot OK"
5	1.25	1.816	3.2	"top OK"	1.183	3.2	"bot OK"
6	1.3	2.153	3.2	"top OK"	0.873	3.2	"bot OK"
7	1.35	2.353	3.2	"top OK"	0.69	3.2	"bot OK"
8	1.4	2.016	3.2	"top OK"	0.654	3.2	"bot OK"
9	1.45	2.081	3.2	"top OK"	0.594	3.2	"bot OK"
10	1.5	2.147	3.2	"top OK"	0.534	3.2	"bot OK"
11	1.55	2.081	3.2	"top OK"	0.594	3.2	"bot OK"
12	1.6	2.016	3.2	"top OK"	0.654	3.2	"bot OK"
13	1.65	2.353	3.2	"top OK"	0.69	3.2	"bot OK"
14	1.7	2.153	3.2	"top OK"	0.873	3.2	"bot OK"
15	1.75	1.816	3.2	"top OK"	1.183	3.2	"bot OK"
16	1.8	1.478	3.2	"top OK"	1.493	3.2	"bot OK"
17	1.85	1.247	3.2	"top OK"	1.265	3.2	"bot OK"
18	1.9	0.779	3.2	"top OK"	1.695	3.2	"bot OK"
19	1.95	0.18	3.2	"top OK"	2.245	3.2	"bot OK"
20	2	-0.418	-0.537	"top OK"	2.795	3.2	"bot OK"

column 0 = span point
column 1 = top stress
column 2 = top allowable
column 3 = top check
column 4 = bottom stress
column 5 = bottom allowable
column 6 = bottom check

20	2	-0.418	-0.537	"top OK"	2.795	3.2	"bot OK"



Flexural Strength - LRFD 5.7.3.1.1 stress in prestressing tendons

I will consider the bonded case only. Use Strength I.

$$fps = fpu \cdot \left(1 - \frac{k \cdot c}{dp}\right)$$

$$k = 2 \cdot \left(1.04 - \frac{fpy}{fpu}\right)$$ for values of k see Table C5.7.3.1.1-1

$$c = \frac{Aps \cdot fpu + As \cdot fy - A's \cdot f'y - 0.85 \cdot \beta 1 \cdot fc \cdot (b - bw) \cdot hf}{0.85 \cdot fc \cdot \beta 1 \cdot bw + k \cdot Aps \cdot \frac{fpu}{dp}}$$ for "T" section behavior (ts<c)

$$c = \frac{Aps \cdot fpu + As \cdot fy - A's \cdot f'y}{0.85 \cdot fc \cdot \beta 1 \cdot b + k \cdot Aps \cdot \frac{fpu}{dp}}$$ for rectangular section behavior (ts>c)

fpu = ultimate strength of tendions

fps = actual stress in strand
c = compression depth
dp = distance from extreme compression fiber to centroid of tendions
Aps = area of prestressing strands
fy = yield of tension reinforcing
As = area of tension reinforcing
A's = area of compression reinforcing
f'y = yield of compression reinforcing
$\beta 1 = 5.7.2.2$
b = width of compression flange (un-transformed)
bw = width of web of beam
hf = depth of compression flange

Calculations

$k := 0.28$ $A's := 0$ $\beta 1 := 0.85$ $fpu := Strand\_strength$ $fpu = 270$

$Aps_{mp} = 7.378$ $f'y := 60$ $bw = 6$ $As := 0$ $fy := 60$

$dp_{ns} := h + ts - ecc_{ns}$ $dp_{mp} = 58.368$ $hf := ts$

$$c_{ns} := \begin{array}{|l} j1_{ns} \leftarrow \dfrac{Aps_{ns} \cdot fpu + As \cdot fy - A's \cdot f'y}{0.85 \cdot fc \cdot \beta1 \cdot ETFW + k \cdot Aps_{ns} \cdot \dfrac{fpu}{dp_{ns}}} \\ j2_{ns} \leftarrow \dfrac{Aps_{ns} + As \cdot fy - A's \cdot f'y - 0.85 \cdot \beta1 \cdot fc \cdot (ETFW - bw) \cdot hf}{0.85 \cdot fc \cdot \beta1 \cdot bw + k \cdot Aps_{ns} \cdot \dfrac{fpu}{dp_{ns}}} \\ j1_{ns} \;\; \text{if} \;\; j1_{ns} < ts \\ j2_{ns} \;\; \text{otherwise} \end{array}$$

$c_{mp} = 6.941$

$$fps_{ns} := fpu \cdot \left(1 - k \cdot \frac{c_{ns}}{dp_{ns}}\right)$$

$fps_{mp} = 261.01$

depth of compressive stress block $a_{ns} := c_{ns} \cdot \beta1$ $a_{mp} = 5.9$

Allowable flexural strength (k*ft) = $Mn_{ns} := Aps_{ns} \cdot fps_{ns} \cdot \left(dp_{ns} - \frac{a_{ns}}{2}\right) \cdot \frac{1}{12}$ $Mn_{mp} = 8893.29$

Check pass or fail condition

$$check\_m_{ns} := \begin{array}{|l} \text{"OK"} \;\; \text{if} \;\; Mn_{ns} > STI6_{ns} \\ \text{"FAIL"} \;\; \text{otherwise} \end{array}$$

Prepare for output display

$disp\_m_{ns,0} := x1_{ns}$ $disp\_m_{ns,3} := c_{ns}$ $disp\_m_{ns,6} := check\_m_{ns}$

$disp\_m_{ns,1} := Aps_{ns}$ $disp\_m_{ns,4} := STI6_{ns}$

$disp\_m_{ns,2} := dp_{ns}$ $disp\_m_{ns,5} := Mn_{ns}$

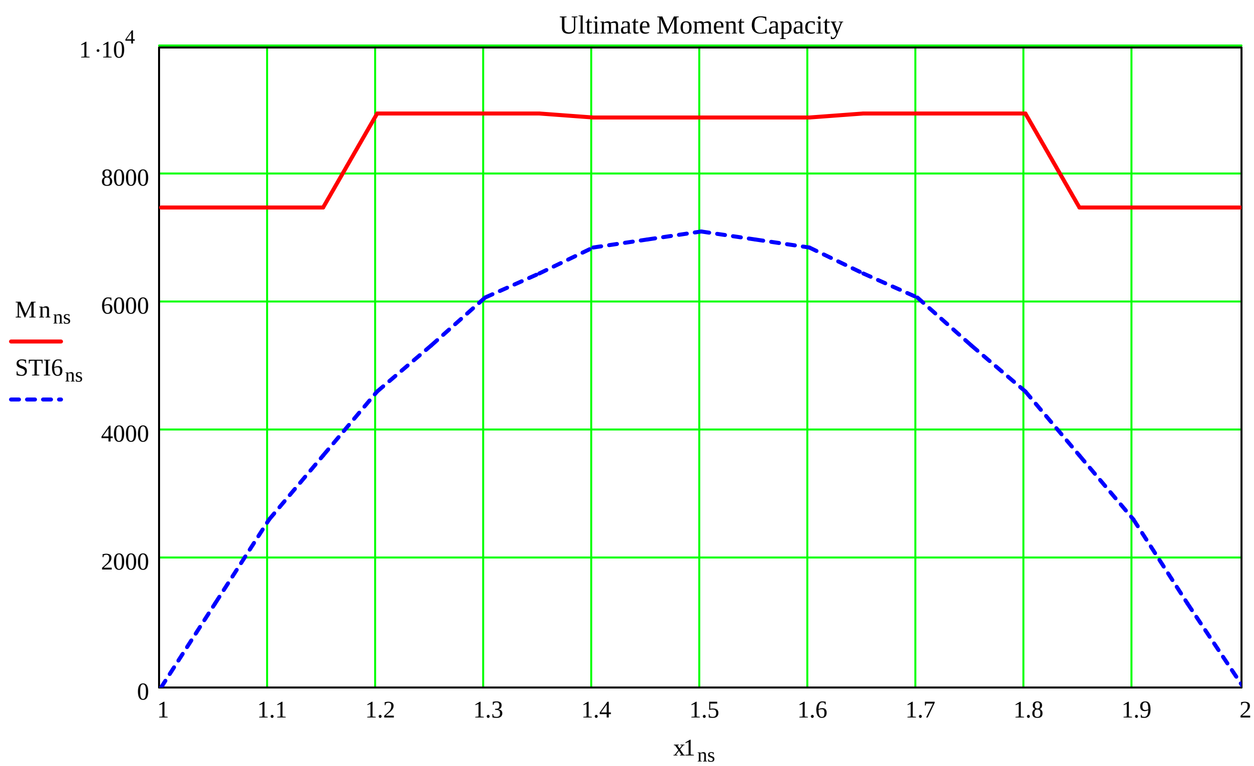
Untimate Moment Capacity Output

	0	1	2	3	4	5	6
0	1	6.944	52.375	6.522	0	7479.76	"OK"
1	1.05	6.944	52.375	6.522	1303.273	7479.76	"OK"
2	1.1	6.944	52.375	6.522	2606.546	7479.76	"OK"
3	1.15	6.944	52.375	6.522	3611.262	7479.76	"OK"
4	1.2	8.246	53.513	7.701	4615.979	8945.663	"OK"
5	1.25	8.246	53.513	7.701	5342.04	8945.663	"OK"
6	1.3	8.246	53.513	7.701	6068.101	8945.663	"OK"
7	1.35	8.246	53.513	7.701	6463.504	8945.663	"OK"

column 0 = span point
column 1 = Aps
column 2 = dp
column 3 = c
column 4 = applied loads (Strength I)
column 5 = capacity Mn
column 6 = pass or fail

disp_m =

8	1.4	7.378	58.368	6.941	6858.906	8893.29	"OK"
9	1.45	7.378	58.368	6.941	6981.552	8893.29	"OK"
10	1.5	7.378	58.368	6.941	7104.197	8893.29	"OK"
11	1.55	7.378	58.368	6.941	6981.552	8893.29	"OK"
12	1.6	7.378	58.368	6.941	6858.906	8893.29	"OK"
13	1.65	8.246	53.513	7.701	6463.504	8945.663	"OK"
14	1.7	8.246	53.513	7.701	6068.101	8945.663	"OK"
15	1.75	8.246	53.513	7.701	5342.04	8945.663	"OK"
16	1.8	8.246	53.513	7.701	4615.979	8945.663	"OK"
17	1.85	6.944	52.375	6.522	3611.262	7479.76	"OK"
18	1.9	6.944	52.375	6.522	2606.546	7479.76	"OK"
19	1.95	6.944	52.375	6.522	1303.273	7479.76	"OK"
20	2	6.944	52.375	6.522	0	7479.76	"OK"



Maximum reinforcing LRFD 5.7.3.3.1

$$\frac{c}{de} \le 0.42$$

where c = the distance fro the extreme compression fiber to the N.A.

$$de = \frac{Aps \cdot fps \cdot dp + As \cdot fy \cdot ds}{Aps \cdot fps + As \cdot fy}$$

sinze As = 0 then de = dp

$de_{ns} := dp_{ns}$

$ratio_{ns} := \frac{c_{ns}}{dp_{ns}}$

$check_{ns} := \begin{cases} \text{"OK"} & \text{if } ratio_{ns} < 0.42 \\ \text{"NG"} & \text{otherwise} \end{cases}$

$disp := 0$

$disp_{ns,0} := x1_{ns}$ $disp_{ns,2} := 0.42$

$disp_{ns,1} := ratio_{ns}$ $disp_{ns,3} := check_{ns}$

disp =

	0	1	2	3
0	1	0.125	0.42	"OK"
1	1.05	0.125	0.42	"OK"
2	1.1	0.125	0.42	"OK"
3	1.15	0.125	0.42	"OK"
4	1.2	0.144	0.42	"OK"
5	1.25	0.144	0.42	"OK"
6	1.3	0.144	0.42	"OK"
7	1.35	0.144	0.42	"OK"
8	1.4	0.119	0.42	"OK"
9	1.45	0.119	0.42	"OK"
10	1.5	0.119	0.42	"OK"
11	1.55	0.119	0.42	"OK"
12	1.6	0.119	0.42	"OK"
13	1.65	0.144	0.42	"OK"
14	1.7	0.144	0.42	"OK"
15	1.75	0.144	0.42	"OK"

column 0 = span point
column 1 = actual ratio
column 2 = allowable ratio
column 3 = check

Minimum Steel Requirement LRFD 5.7.3.3.2

Unless othewise specified, at any section of a flexural component, the amount of prestressed and nonprestressed tensile reinforcement shall be adequate to develop a factored flexural resistance, Mr, at least equal to the lesse of

1. 1.2 times the cracking strength determined on the basis of elastic stress distribution and the modulus of rupture, fr, of the concrete as specified in Article 5.4.2.6

2. 1.33 times the factored moment required by the applicable strength load combinations specified in Table 3.4.1-1. I shall basically ignore this one because it makes no sense. You see the allowable in the section is by default greater that the Strength I combination, so I shall use this as my minimum. If they wanted the allowable to be larger, to the

1.33 value, they would have increased it by 1.33 already.

The LRFD code does not specify a method for calculation Mcr, I shall therefore use the method specified in the AASHTO standard spec.

AASHTO 9.18.2.1; The total amount of prestressed and non-prestressed reinforcement shall be adequate to develop an ultimate moment at the critical section at least 1.2*Mcr

$$\phi \cdot Mn \geq 1.2 \cdot Mcr$$

$$Mcr = Sc \cdot (fr + fpe) - Mdnc \cdot \left(\frac{Sbc}{Sb} - 1 \right)$$

Sbc = composite section modulus at extreme tension fiber
Sb = non-composite section modulus at extreme tension fiber
fr = Modulus of rupture of concrete **LRFD 5.4.2.6**
fpe = compressive stress in concrete due to effective prestress forces only (after allowance for all loses) at extreme fiber of section where tensile stress is caused by externally applied loads.

Composite section modulus (in^3) = $Sbc = 18147.259$

Non compoiste section modulus (in^3) = $Sb = 10543.065$

Modulus of rupture (psi) = $fr := 0.24 \cdot \sqrt{fcf}$ $\quad fr = 0.679$

fpe (psi) = $fpe_{ns} := \frac{Ff_{ns}}{Ac} + \frac{Ff_{ns} \cdot enc_{ns}}{Sb}$ $\quad fpe_{mp} = 3.601$

Non-composite DL moment (lb*in) = $Mdnc_{ns} := \left(SI1_{ns} + SI2_{ns} \right) \cdot 12$ $\quad Mdnc_{mp} = 28700.912$

Cracking moment (k*ft) = $Mcr_{ns} := \left[Sbc \cdot \left(fr + fpe_{ns} \right) - Mdnc_{ns} \cdot \left(\frac{Sbc}{Sb} - 1 \right) \right] \cdot \frac{1}{12}$ $\quad Mcr_{mp} = 4747.161$

Check Minimum $\quad check\_min_{ns} := \begin{cases} \text{"Min OK"} & \text{if } Mn_{ns} > 1.2 \cdot Mcr_{ns} \\ \text{"Min NG"} & \text{otherwise} \end{cases}$ $\quad check\_min_{mp} = \text{"Min OK"}$

$disp := 0$

$disp_{ns,0} := x1_{ns}$ $\quad disp_{ns,1} := Mn_{ns}$ $\quad disp_{ns,2} := Mcr_{ns}$ $\quad disp_{ns,3} := check\_min_{ns}$

	0	1	2	3
0	1	7479.76	5254.068	"Min OK"

column 0 = span point

disp =

0	1	7479.76	5254.068	"Min OK"
1	1.05	7479.76	4944.76	"Min OK"
2	1.1	7479.76	4635.451	"Min OK"
3	1.15	7479.76	4393.545	"Min OK"
4	1.2	8945.663	5146.784	"Min OK"
5	1.25	8945.663	4972.279	"Min OK"
6	1.3	8945.663	4797.774	"Min OK"
7	1.35	8945.663	4694.671	"Min OK"
8	1.4	8893.29	4814.562	"Min OK"
9	1.45	8893.29	4780.862	"Min OK"
10	1.5	8893.29	4747.161	"Min OK"
11	1.55	8893.29	4780.862	"Min OK"
12	1.6	8893.29	4814.562	"Min OK"
13	1.65	8945.663	4694.671	"Min OK"
14	1.7	8945.663	4797.774	"Min OK"
15	1.75	8945.663	4972.279	"Min OK"
16	1.8	8945.663	5146.784	"Min OK"
17	1.85	7479.76	4393.545	"Min OK"
18	1.9	7479.76	4635.451	"Min OK"
19	1.95	7479.76	4944.76	"Min OK"
20	2	7479.76	5254.068	"Min OK"

column 1 = capacity of section
column 2 = cracking moment
column 3 = minimum check

Check Deflections

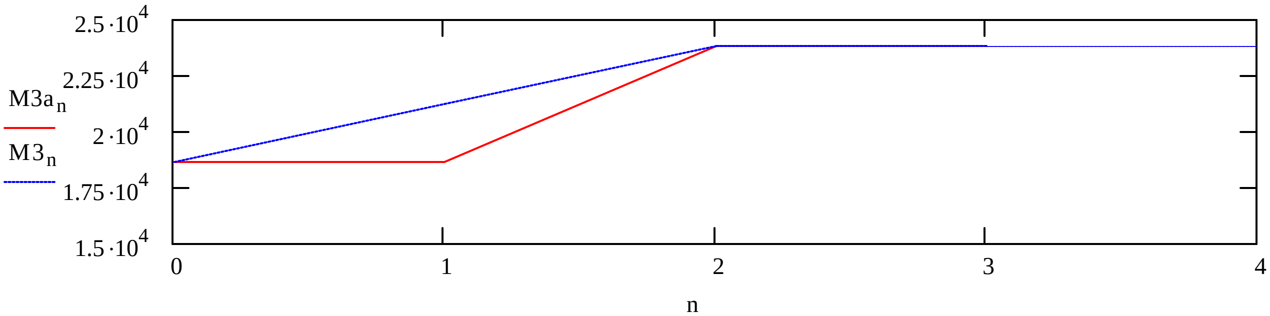
$n := 0..4$

Deflection due to pre-stress force (in) I will use the M/(E*I) method

Length of each section = $int := \frac{length}{10}$ $int = 10$

Preliminary moment curve (all points) $M1_{ns} := enc_{ns} \cdot Fi_{ns}$

Define actual moment curve = $M3a := \begin{pmatrix} M1_0 \\ M1_3 \\ M1_4 \\ M1_5 \\ M1_6 \end{pmatrix}$ $M3a = \begin{pmatrix} 18629.998 \\ 18629.998 \\ 23818.149 \\ 23818.149 \\ 23818.149 \end{pmatrix}$ $M3 := \begin{bmatrix} (M3a_0 + M3a_1)\cdot 0.5 \\ (M3a_1 + M3a_2)\cdot 0.5 \\ (M3a_2 + M3a_3)\cdot 0.5 \\ (M3a_3 + M3a_4)\cdot 0.5 \\ M3a_4 \end{bmatrix}$



Define range for each point on moment curve $xr := \begin{pmatrix} 4.5\cdot int \\ 3.5\cdot int \\ 2.5\cdot int \\ 1.5\cdot int \\ 0.5\cdot int \end{pmatrix}\cdot 12$ $xr = \begin{pmatrix} 540 \\ 420 \\ 300 \\ 180 \\ 60 \end{pmatrix}$

Area along each block $M4_n := M3_n\cdot int\cdot 12$ $M4 = \begin{pmatrix} 2235599.809 \\ 2546888.835 \\ 2858177.861 \\ 2858177.861 \\ 2858177.861 \end{pmatrix}$

Determine reaction area $M5 := \sum_n M4_n$ $M5 = 1.336\times 10^7$

Area of each block * range $M6_n := M4_n\cdot xr_n$

Final Pre-Stress Deflection $defl\_p := \dfrac{(M5\cdot length\cdot 12\cdot 0.5) - \sum_n M6_n}{Eci\cdot Inc}$ $defl\_p = 3.425$

Method two for pre-stress camber
using a little different take on the M/EI method

$lc := length$ $xr := 0$

Length of each section = $int := \dfrac{lc}{sp}\cdot 12$ $int = 60$

Range (in) =

$ns1 := 0..\, sp$

$$xr_{ns1} := \left| \begin{array}{l} \text{for } j \in 0..\, sp \\ \quad j1_j \leftarrow int \cdot j \\ j1_{ns1} \end{array} \right.$$

Preliminary moment curve (all points)

$M1_{ns} := enc_{ns} \cdot Fi_{ns}$ $M1_{sp+1} := M1_{sp}$

Block area (moment*length) (kip*in) =

$M2_{ns} := M1_{ns} \cdot int$

Reaction =

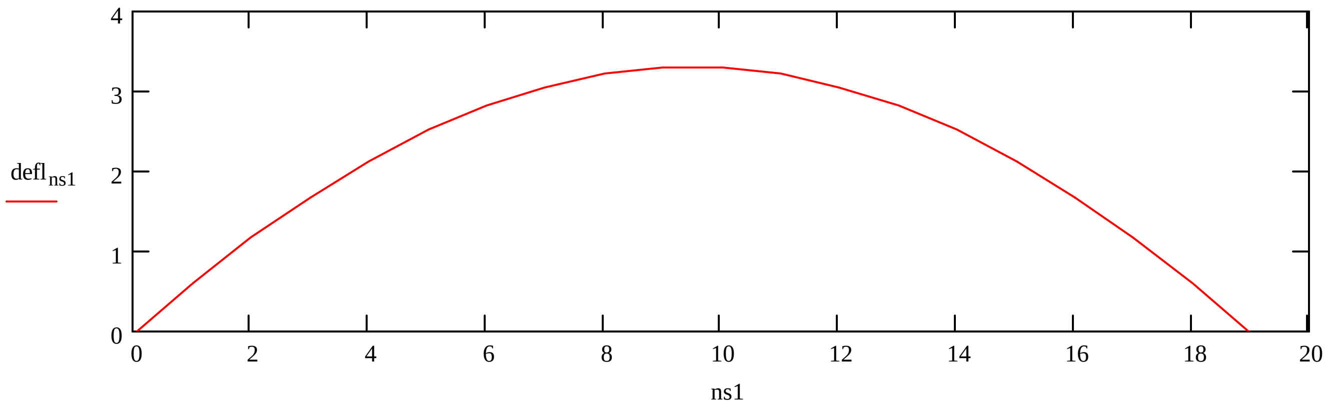
$M3 := \left(\sum_{ns} M2_{ns} \right) \cdot \frac{1}{2}$ $M3 = 1.435 \times 10^7$

Sum at points

$$M5_{ns1} := \left| \begin{array}{l} \text{for } j \in 0..\, sp \\ \quad \left| \begin{array}{l} \text{for } j3 \in 0..\, j \\ \quad \left| \begin{array}{l} j1 \leftarrow int \cdot j \\ j2_{j3} \leftarrow j1 - int \cdot j3 - \frac{int}{2} + int \\ j4_{j3} \leftarrow M2_{j3} \cdot j2_{j3} \end{array} \right. \\ j5_j \leftarrow \sum_{j6=0}^{j} j4_{j6} \end{array} \right. \\ j5_{ns1} \end{array} \right.$$

Deflection at points

$$defl_{ns1} := \frac{M3 \cdot xr_{ns1} - M5_{ns1}}{Eci \cdot Inc}$$



$disp := 0$ $\quad disp_{ns,0} := x1_{ns}$ $\quad disp_{ns,1} := defl_{ns}$

disp =

	0	1
0	1	-0.027
1	1.05	0.594
2	1.1	1.16
3	1.15	1.672
4	1.2	2.121
5	1.25	2.501
6	1.3	2.81
7	1.35	3.049
8	1.4	3.212
9	1.45	3.294
10	1.5	3.294
11	1.55	3.212
12	1.6	3.049
13	1.65	2.81
14	1.7	2.501
15	1.75	2.121
16	1.8	1.672
17	1.85	1.16
18	1.9	0.594
19	1.95	-0.027
20	2	-0.703

column 0 = span point
column 1 = deflection (in)

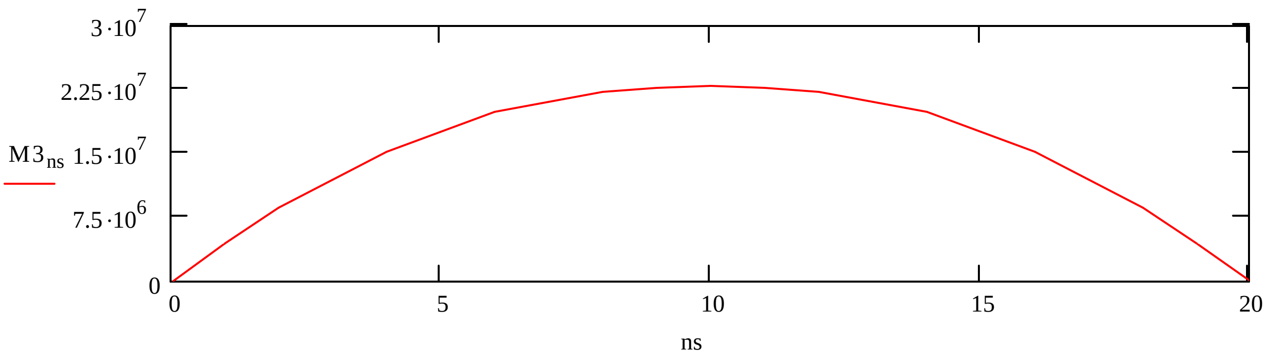
Deflections under Live Load (SL, max Positive),

These will be based on simple span Live Load Moments
I will use the M/(E*I) method

Length of section for calculations (ft) = $lc := length$ $lc = 100$

Length of each section = $int := \frac{length}{sp} \cdot 12$ $int = 60$

Define actual moment curve = $M3_{ns} := disp3aa_{ns} \cdot LLDFM \cdot 12000$



Define range for each point on moment curve

$$xr1_{ns} := \left| \begin{array}{l} \text{for } j \in 0..sp \\ \quad j1_j \leftarrow \frac{lc \cdot 12}{2} - int \cdot j - \frac{int}{2} \\ j1_{ns} \end{array} \right.$$

Area along each block $M4_{ns} := M3_{ns} \cdot length \cdot 12 \cdot \frac{1}{sp}$

Determine reaction area $M5 := \left(\sum_{ns} M4_{ns} \right) \cdot \frac{1}{2}$

Area of each block * range $M6_{ns} := M4_{ns} \cdot xr1_{ns}$

Final Deflection

$$defl\_p := \frac{M5 \cdot lc \cdot 12 \cdot 0.5 - \sum_{j=0}^{sp \cdot 0.5} M6_j}{Ec \cdot 1000 \cdot Ic}$$

$defl\_p = 0.948$

Positive value indicates an downward deflection

Max allowable deflection $max\_d := \frac{length \cdot 12}{800}$ $max\_d = 1.5$

Deflections due to non-composite Dead Loads (DC)

I will calculate the deflection due to beam weight based on the initial strength (and modulus) of the beam. The slab weight shall be applied to the final concrete strength.

$n1 := 0..10$

$w := \frac{bwt}{12}$ $\quad w = 0.069$ $\quad ws := \frac{DLnc}{12}$

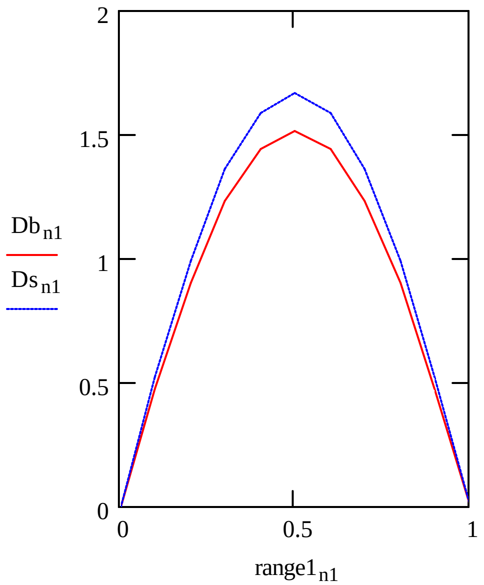
$ws = 0.087$

range for tenth points (in) = $range1 := \begin{pmatrix} 0 \\ 0.1 \\ 0.2 \\ 0.3 \\ 0.4 \\ 0.5 \\ 0.6 \\ 0.7 \\ 0.8 \\ 0.9 \\ 1.0 \end{pmatrix}$ $\quad x_{n1} := range1_{n1} \cdot length \cdot 12$

Deflection from self weight at tenth points (k*in) = $Db_{n1} := \frac{w \cdot x_{n1}}{24 \cdot Inc \cdot Eci} \cdot \left[(length \cdot 12)^3 - 2 \cdot length \cdot 12 \cdot \left(x_{n1}\right)^2 + \left(x_{n1}\right)^3 \right]$

Deflection from Non-Composite at tenth points (k*in) = $Ds_{n1} := \frac{ws \cdot x_{n1}}{24 \cdot Inc \cdot Ec} \cdot \left[(length \cdot 12)^3 - 2 \cdot length \cdot 12 \cdot \left(x_{n1}\right)^2 + \left(x_{n1}\right)^3 \right]$

column 0 = span point
column 1 = self wt
column 2 = non-comp Defl
column 3 = rail deflection
column 4 = Total for Oklahoma curve



disp =

	0	1	2
0	0	0	0
1	0.1	0.474	0.523
2	0.2	0.897	0.99
3	0.3	1.228	1.355
4	0.4	1.439	1.587
5	0.5	1.511	1.666
6	0.6	1.439	1.587
7	0.7	1.228	1.355
8	0.8	0.897	0.99
9	0.9	0.474	0.523
10	1	0	0

Shear Design, pre-stress method

LRFD 5.8.2.4 Except for slabs footings and culverts, transverse reinforcement shall be provided where

$$Vu > 0.5 \cdot \phi \cdot (Vc + Vp)$$

Vu = factored shear force
Vc = nominal resistance of concrete
Vp = component of the prestressing force in direction of the shear force

LRFD 5.8.2.5: Minimum transverse reinforcing

$$Av = 0.0316 \cdot \sqrt{fcf} \cdot \frac{bv \cdot S}{fy}$$

Av = area of transverse reinforcing within distance S
bv = width of web adjusted for the presence of ducts as specified in 5.8.2.9
S = spacing of transverse reinforcing
fy = yeild strength of transverse reinforcing
fcf = final concrete strength

LRFD 5.8.2.7: Maximum spacing of transverse reinforcing.

If Vu<0.125*fcf then: Smax = 0.8*dv<= 24 in

If Vu>=0.125*fcf then: Smax = 0.4*dv<= 12 in

Vu = the shear stress calculated in accordance with 5.8.2.9
dv = effective shear depth as defined in 5.8.2.9

LRFD 5.8.2.9: Shear stress in concrete

$$V = \frac{Vu - \phi \cdot Vp}{\phi \cdot bv \cdot dv}$$

bv = effective web width

dv = effective shear depth $\quad dv = \dfrac{Mn}{As \cdot fy + Aps \cdot fps}$

ϕ = resistance factor for shear 5.5.4.2

LRFD 5.8.3.3: The nominal shear resistance Vn shall be determined as the lesser of

$$Vn = Vc + Vs + Vp$$

$$Vn = 0.25 \cdot fcf \cdot bv \cdot dv + Vp$$

for which

$$Vc = 0.0316 \cdot \beta \cdot \sqrt{fcf} \cdot bv \cdot dv$$

$Vs = \dfrac{Av \cdot fy \cdot dv \cdot \cot(\theta)}{S}$ this is as per commentary EQ C5.8.3.3.1

β = Factor as defined in article 5.8.3.4
θ = angle of inclination of diagonal compressive stresses as determined in 5.8.3.4
α = angle of inclination of transverse reinforcement to longitudinal axis

LRFD5.8.3.4.2: Use for the calculation of β and θ

$$\varepsilon t = \frac{\dfrac{Mu_{ns}}{dv_{ns}} + Vu_{ns} - Vp_{ns} - Aps_{ns} \cdot fpo}{Ep \cdot Aps_{ns}}$$

$$\varepsilon x = \frac{\varepsilon t}{2}$$

Ac = area of concrete on the flexural tension side of the member (see fig 1)
Aps = area of prestressing steel on the flexural tension side of the member (see fig 1)
As = area of nonprestressed steel on the flexural tension side of the member at the section (fig 1)
fpo = for the usual levels of prestressing 0.7*fpu will be appropriate
Mu = factored moment taken as positive, but not taken less than Vu*dv
Vu = factored shear force taken as positive

$mp := 3$

Width of web (in) = $bv := bw$ $bv = 6$

Distance from top slab to CL. prestressing (in) = $de_{ns} := h + ts - ecc_{ns}$ $de_{mp} = 52.375$

Effective shear depth (in) =

$$dv_{ns} := \left| \begin{array}{l} j_{ns} \leftarrow \left| \begin{array}{ll} 0.9 \cdot (de_{ns}) & \text{if } 0.9 \cdot de_{ns} > 0.72 \cdot (h + ts) \\ 0.72 \cdot (h + ts) & \text{otherwise} \end{array} \right. \\ j1_{ns} \leftarrow de_{ns} - a_{ns} \cdot 0.5 \\ j_{ns} \ \text{if } j_{ns} > j1_{ns} \\ j1_{ns} \ \text{otherwise} \end{array} \right.$$

$dv_{mp} = 49.603$

Define factored shear (k) = $Vu_{ns} := \max\left(\begin{pmatrix} |STI6v_{ns}| \\ |STI7v_{ns}| \end{pmatrix}\right)$ $Vu_{mp} = 244.729$

Define factored moment (k*in) = $Mu1_{ns} := \max\left(\begin{pmatrix} |STI6_{ns}| \\ |STI7_{ns}| \end{pmatrix}\right) \cdot 12$ $Mu1_{mp} = 43335.148$

Mu shall not be less than Vu*dv (k*in) = $Mu_{ns} := \max\left(\begin{pmatrix} Mu1_{ns} \\ Vu_{ns} \cdot dv_{ns} \end{pmatrix}\right)$ $Mu_{mp} = 43335.148$

Resistance factor for concrete (5.5.4.2) = $\phi := 0.9$

Calculate force "fpo" (ksi) = $fpo := 0.7 \cdot Strand\_strength$ $\quad fpo = 189$

Use stress for vertical force (k) = $Fp_{ns} := (fpo - \Delta ft) \cdot Aps_{ns}$ $\quad Fp_{mp} = 950.739$

Component of the prestressing force in direction of the shear force Vp (k) =

$$angle_{ns} := \begin{cases} 0 & \text{if } harped = \text{"n"} \\ \operatorname{atan}\left(\left|\dfrac{enc_0 - enc_{ceil(sp \cdot 0.5)}}{length \cdot 12 \cdot depress}\right|\right) \cdot \dfrac{180}{\pi} & \text{otherwise} \end{cases}$$

$$Vp_{ns} := Ff_{ns} \cdot \sin\left(angle_{ns} \cdot \frac{\pi}{180}\right) \qquad Vp_{mp} = 0$$

Shear stress on the concrete (ksi) =

$$V_{ns} := \frac{Vu_{ns} - \phi \cdot Vp_{ns}}{\phi \cdot bv \cdot dv_{ns}} \qquad V_{mp} = 0.914$$

Ratio of V/fcf =

$$r_{ns} := \frac{V_{ns}}{fcf} \qquad r_{mp} = 0.114$$

Maximum spacing of transverse reinforcing (in) =

$$Smax_{ns} := \begin{cases} \min\left(\begin{pmatrix} 0.8 \cdot dv_{ns} \\ 24.0 \end{pmatrix}\right) & \text{if } V_{ns} < 0.125 \cdot fcf \\ \min\left(\begin{pmatrix} 0.4 \cdot dv_{ns} \\ 12.0 \end{pmatrix}\right) & \text{otherwise} \end{cases} \qquad Smax_{mp} = 24$$

$disp := 0$

$disp_{ns,0} := x1_{ns}$ $\quad disp_{ns,2} := de_{ns}$ $\quad disp_{ns,4} := Vu_{ns}$ $\quad disp_{ns,6} := Vp_{ns}$ $\quad disp_{ns,8} := r_{ns}$

$disp_{ns,1} := bv$ $\quad disp_{ns,3} := dv_{ns}$ $\quad disp_{ns,5} := Mu_{ns}$ $\quad disp_{ns,7} := V_{ns}$ $\quad disp_{ns,9} := Smax_{ns}$

column 0 = span point
column 1 = "bv" web width
column 2 = "de"
column 3 = "dv"
column 4 = Vu, Strength I shear
column 5 = Mu, Strength I moment
column 6 = Vp, vertical component of shear
column 7 = applied shear stress
column 8 = ration of shear stress to concrete strength
column 9 = Maximum spacing of transverse reinforcing

disp =

	0	1	2	3	4	5	6	7	8	9
0	1	6	52.375	49.603	321.235	15934.24	0	1.199	0.15	12
1	1.05	6	52.375	49.603	295.733	15639.275	0	1.104	0.138	12
2	1.1	6	52.375	49.603	270.231	31278.551	0	1.009	0.126	12
3	1.15	6	52.375	49.603	244.729	43335.148	0	0.914	0.114	24
4	1.2	6	53.513	50.24	219.227	55391.746	0	0.808	0.101	24
5	1.25	6	53.513	50.24	193.725	64104.481	0	0.714	0.089	24
6	1.3	6	53.513	50.24	168.223	72817.217	0	0.62	0.078	24
7	1.35	6	53.513	50.24	142.394	77562.043	0	0.525	0.066	24
8	1.4	6	58.368	55.418	116.565	82306.87	0	0.39	0.049	24
9	1.45	6	58.368	55.418	92.488	83778.618	0	0.309	0.039	24
10	1.5	6	58.368	55.418	68.411	85250.366	0	0.229	0.029	24
11	1.55	6	58.368	55.418	92.488	83778.618	0	0.309	0.039	24
12	1.6	6	58.368	55.418	116.565	82306.87	0	0.39	0.049	24
13	1.65	6	53.513	50.24	142.394	77562.043	0	0.525	0.066	24
14	1.7	6	53.513	50.24	168.223	72817.217	0	0.62	0.078	24
15	1.75	6	53.513	50.24	193.725	64104.481	0	0.714	0.089	24
16	1.8	6	53.513	50.24	219.227	55391.746	0	0.808	0.101	24
17	1.85	6	52.375	49.603	244.729	43335.148	0	0.914	0.114	24
18	1.9	6	52.375	49.603	270.231	31278.551	0	1.009	0.126	12
19	1.95	6	52.375	49.603	295.733	15639.275	0	1.104	0.138	12
20	2	6	52.375	49.603	321.235	15934.24	0	1.199	0.15	12

Compute the strain in the reinforcement

Modulus of prestressing strands (ksi) = $Ep = 28500$

$$\varepsilon t_{ns} := \frac{\frac{Mu_{ns}}{dv_{ns}} + Vu_{ns} - Vp_{ns} - Aps_{ns} \cdot fpo}{Ep \cdot Aps_{ns}}$$ $\varepsilon t_{mp} = -0.0009805$

$$\varepsilon x_{ns} := \min\left(\begin{pmatrix} 0.002 \\ \frac{\varepsilon t_{ns}}{2} \end{pmatrix}\right)$$ $\varepsilon x_{mp} = -0.0004903$

Enter Table 5.8.3.4.2-1 and indicate values of θ and β

expand area for value determination

$disp := 0$

$disp_{ns,0} := x1_{ns}$ $disp_{ns,1} := \varepsilon x_{ns}$ $disp_{ns,2} := r_{ns}$ $disp_{ns,3} := \theta_{ns}$ $disp_{ns,4} := \beta_{ns}$

disp =

	0	1	2	3	4
0	1	-0.00169	0.14991	21.6	2.88
1	1.05	-0.00177	0.13801	21.6	2.88
2	1.1	-0.00104	0.12611	21.6	2.88
3	1.15	-0.00049	0.11421	19.9	3.18
4	1.2	-0.0005	0.10101	19.9	3.18
5	1.25	-0.00019	0.08926	20.4	3.38
6	1.3	0.00013	0.07751	27.1	2.75
7	1.35	0.00027	0.06561	30.5	2.59
8	1.4	0.00049	0.04869	30.5	2.59
9	1.45	0.0005	0.03863	30.5	2.59
10	1.5	0.0005	0.02858	33.7	2.38
11	1.55	0.0005	0.03863	30.5	2.59
12	1.6	0.00049	0.04869	30.5	2.59
13	1.65	0.00027	0.06561	30.5	2.59
14	1.7	0.00013	0.07751	27.1	2.75
15	1.75	-0.00019	0.08926	20.4	3.38
16	1.8	-0.0005	0.10101	19.9	3.18

column 0 = span point
column 1 = strain
column 2 = ratio of V/fcf
column 3 = angle θ
column 4 = factor β

17	1.85	-0.00049	0.11421	19.9	3.18
18	1.9	-0.00104	0.12611	21.6	2.88
19	1.95	-0.00177	0.13801	21.6	2.88
20	2	-0.00169	0.14991	21.6	2.88

Range definition $ns1 := 0..2$

Area of double no. 5 bars (in^2)= $Av_{ns,0} := 0.4$

Area of double no. 6 bars (in^2) = $Av_{ns,1} := 0.62$

Area of double no. 7 bars (in^2) = $Av_{ns,2} := 0.88$

Nominal resistance of concrete (k) = $Vc_{ns} := 0.0316 \cdot \beta_{ns} \cdot \sqrt{fcf} \cdot bv \cdot dv_{ns}$ $Vc_{mp} = 84.59$

Calculate required nominal steel strength (k) =

$$Vs_{ns} := \left| \begin{array}{l} j_{ns} \leftarrow \dfrac{Vu_{ns}}{\phi} - Vc_{ns} - Vp_{ns} \\ 0 \ \text{ if } \ Vu_{ns} < 0.5 \cdot \phi \cdot \left(Vc_{ns} + Vp_{ns}\right) \\ \text{otherwise} \\ \quad \left| \begin{array}{l} 0 \ \text{ if } \ j_{ns} < 0 \\ j_{ns} \ \text{ otherwise} \end{array} \right. \end{array} \right.$$

$Vs_{mp} = 187.331$

Required spacing of stirrups (in) =
(not considering max)

$$S1_{ns,ns1} := \left| \begin{array}{l} Smax_{ns} \ \text{ if } \ Vs_{ns} = 0 \\ j_{ns} \leftarrow \dfrac{Av_{ns,ns1} \cdot fy \cdot dv_{ns} \cdot \cot\left(\theta_{ns} \cdot \dfrac{\pi}{180}\right)}{Vs_{ns}} \ \text{ otherwise} \end{array} \right.$$

$disp := 0 \qquad disp_{ns,0} := x1_{ns} \qquad disp_{ns,1} := Vc_{ns} \qquad disp_{ns,2} := Vs_{ns}$

disp =

	0	1	2
0	1	76.61	280.318
1	1.05	76.61	251.982
2	1.1	76.61	223.647
3	1.15	84.59	187.331
4	1.2	85.676	157.91
5	1.25	91.065	124.186
6	1.3	74.091	112.824
7	1.35	69.78	88.435
8	1.4	76.972	52.545
9	1.45	76.972	25.793
10	1.5	70.731	5.282
11	1.55	76.972	25.793
12	1.6	76.972	52.545
13	1.65	69.78	88.435
14	1.7	74.091	112.824
15	1.75	91.065	124.186

column 0 = span point
column 1 = "Vc", nominal concrete strength
column 2 = "Vs", required steel strength

$disp := 0$

actual required spacing

S1 =

	0	1	2
0	10.726	16.626	23.598
1	11.933	18.495	26.252
2	13.444	20.839	29.578
3	17.555	27.211	38.622
4	21.094	32.695	46.406
5	26.108	40.467	57.437
6	20.884	32.371	45.946
7	23.147	35.877	50.922
8	42.972	66.606	94.538
9	87.541	135.689	192.591
10	377.572	585.237	830.659
11	87.541	135.689	192.591
12	42.972	66.606	94.538
13	23.147	35.877	50.922
14	20.884	32.371	45.946

maximum allowable spacing

Smax =

	0
0	12
1	12
2	12
3	24
4	24
5	24
6	24
7	24
8	24
9	24
10	24
11	24
12	24
13	24
14	24

15	26.108	40.467	57.437

15	24

Spacing of stirrups considering the max allowable

Actual spacing to use (in) =

$$S_{ns,ns1} := \begin{cases} Smax_{ns} & \text{if } Vs_{ns} = 0 \\ \text{otherwise} & \\ \quad j_{ns} \leftarrow \dfrac{Av_{ns,ns1} \cdot fy \cdot dv_{ns} \cdot \cot\left(\theta_{ns} \cdot \dfrac{\pi}{180}\right)}{Vs_{ns}} & \\ \quad Smax_{ns} & \text{if } j_{ns} > Smax_{ns} \\ \quad j_{ns} & \text{otherwise} \end{cases}$$

$S_{mp,ns1} =$

17.555
24
24

$S =$

	0	1	2
0	10.726	12	12
1	11.933	12	12
2	12	12	12
3	17.555	24	24
4	21.094	24	24
5	24	24	24
6	20.884	24	24
7	23.147	24	24
8	24	24	24
9	24	24	24
10	24	24	24
11	24	24	24
12	24	24	24
13	23.147	24	24
14	20.884	24	24
15	24	24	24
16	21.094	24	24
17	17.555	24	24
18	12	12	12
19	11.933	12	12
20	10.726	12	12
21			
22			

$x1 =$

	0
0	1
1	1.05
2	1.1
3	1.15
4	1.2
5	1.25
6	1.3
7	1.35
8	1.4
9	1.45
10	1.5
11	1.55
12	1.6
13	1.65
14	1.7
15	1.75
16	1.8
17	1.85
18	1.9
19	1.95
20	2
21	
22	

column 0 = required spacing for no. 4 stirrup
column 1 = required spacing for no. 5 stirrup
column 2 = required spacing for no. 6 stirrup

Check actual against minimum

Actual area of steel used per foot (in^2) = $Avact_{ns,ns1} := \frac{12}{S_{ns,ns1}} \cdot Av_{ns,ns1}$

Minimum transverse reinforcing (in^2) = $Avmin_{ns,ns1} := 0.0316\sqrt{fcf} \cdot \frac{bv \cdot Av_{ns,ns1}}{fy}$

$Avmin_{mp,ns1} = \begin{pmatrix} 0.004 \\ 0.006 \\ 0.008 \end{pmatrix}$

$check := 0$

Check minimum $check_{ns,ns1} := \begin{cases} \text{"OK"} & \text{if } Avact_{ns,ns1} > Avmin_{ns,ns1} \\ \text{"NG"} & \text{otherwise} \end{cases}$

actual area of steel used

Avact =

	0	1	2
0	0.447	0.62	0.88
1	0.402	0.62	0.88
2	0.4	0.62	0.88
3	0.273	0.31	0.44
4	0.228	0.31	0.44
5	0.2	0.31	0.44
6	0.23	0.31	0.44
7	0.207	0.31	0.44
8	0.2	0.31	0.44
9	0.2	0.31	0.44
10	0.2	0.31	0.44
11	0.2	0.31	0.44
12	0.2	0.31	0.44
13	0.207	0.31	0.44
14	0.23	0.31	0.44
15	0.2	0.31	0.44

required minimum area of steel

Avmin =

	0	1	2
0	0.004	0.006	0.008
1	0.004	0.006	0.008
2	0.004	0.006	0.008
3	0.004	0.006	0.008
4	0.004	0.006	0.008
5	0.004	0.006	0.008
6	0.004	0.006	0.008
7	0.004	0.006	0.008
8	0.004	0.006	0.008
9	0.004	0.006	0.008
10	0.004	0.006	0.008
11	0.004	0.006	0.008
12	0.004	0.006	0.008
13	0.004	0.006	0.008
14	0.004	0.006	0.008
15	0.004	0.006	0.008

check conditions

check =

	0	1	2
0	"OK"	"OK"	"OK"
1	"OK"	"OK"	"OK"
2	"OK"	"OK"	"OK"
3	"OK"	"OK"	"OK"
4	"OK"	"OK"	"OK"
5	"OK"	"OK"	"OK"
6	"OK"	"OK"	"OK"
7	"OK"	"OK"	"OK"
8	"OK"	"OK"	"OK"
9	"OK"	"OK"	"OK"
10	"OK"	"OK"	"OK"
11	"OK"	"OK"	"OK"
12	"OK"	"OK"	"OK"
13	"OK"	"OK"	"OK"
14	"OK"	"OK"	"OK"
15	"OK"	"OK"	"OK"

LRFD 5.10.10.1 Factored Bursting Resistance

The bursting resistance of pretenioned anchorage zones provided by vertical reinforcement in the ends of pretensioned beams at the service limit state shall be taken as.

Pr=fs*As

fs = stress in steel but not taken greater than 20 ksi
As = total area of vertical reinforcement located within the distance h/4 from the end of the beam
h = overall depth of precast member

The resistance shall not be less than 4% of the prestressing force at transfer. The end vertical reinforcement shall be as close to the end of the beam as practicable.

$Fpi := Fi_0$ $\qquad Fpi = 1254.123$

The bursting resistance shall not be less than $\qquad Pr := 0.04 \cdot Fpi \qquad Pr = 50.165$

The total required steel within h/4 (in^2) = $\qquad Asv := \frac{Pr}{20} \qquad Asv = 2.508$

Spacing of stirrups $\qquad Spb_{ns1} := \frac{Av_{0,ns1}}{Asv} \cdot \frac{h}{4}$

$$Spb = \begin{pmatrix} 2.153 \\ 3.337 \\ 4.736 \end{pmatrix}$$

no. 4
no. 5
no. 6

LRFD 5.8.4.1 INTERFACE SHEAR (horizontal shear)

5.8.4.1 GENERAL: Interface shear shall be considered across a given plane at

1. An existing or potential deck.
2. An interface between dissimilar materials.
3. An interface between two concretes cast at different times.

The nominal shear resistance of the interface plane shall be taken as: $Vn = c \cdot Acv + \mu \cdot (Avf \cdot fy + Pc)$

The nominal shear resistance used in the design shall not exceed: $Vn \le 0.2 \cdot fc \cdot Acv$ or $Vn \le 0.8 \cdot Acv$

Vn = nominal shear resistance
Acv = area of concrete engaged in shear transfer
Avf: area of shear reinforcement crossing the shear plane
fy = yeild strength of reinforcement
c = cohesion factor specified in 5.8.4.2
μ = friction factor specified in 5.8.4.2
Pc = permanent net compressive force normal to the shear plane: if force is tnesile, Pc = 0.0
fc = specified 28 day compressive strength of the weaker concrete

Reinforcement for interface shear between concretes of slab and beams or girders may consist of single bars, multiple leg stirrups, or the vertical legs of welded wire fabric. The cross-sectional area, Avf of the reinforcement per unit length of the beam or girder should not be less than either that required by Equation 1 or

$$Avf \ge \frac{0.05 \cdot bv}{fy}$$

where bv = width of the interface

The minimum reinforcement requirement of Avf may be waived if Vn/Acv is less than 0.100 ksi.

From C5.8.4.1-1 the applied horizontal shear force may be taken as $Vh = \frac{Vu}{de}$

Vh = horizontal shear per unit length of the girder
Vu = the factored vertical shear
de = the distance between the centroid of the steel in the tension side of the beam to the center of the compression blocks in the deck.

$c := 0$

From LRFD 5.8.4.2 $c := 0.100$

$\mu := 1.0$

Area of concrete engaged in shear transfer (in^2/ft) = $Acv := fw$ $Acv = 20$

Area of shear reinforcement crossing the shear plane (in^2/ft) =
(I shall use the smaller of the stirrup sized difened earlier)

$Avf_{ns} := \frac{Avact_{ns,0}}{2}$ $Avf_{mp} = 0.137$

Permanent net compressive force normal to the shear plane (k) = $Pc := 0$

Compressive strength of the weaker concrete at 28 days (ksi) = $fc = 4$

Yield strength of reinforcing (ksi) = $fy = 60$

The nominal shear resistance

$Vn1_{ns} := c \cdot Acv + \mu \cdot (Avf_{ns} \cdot fy + Pc)$ $Vn1_{mp} = 10.203$

$Vn2_{ns} := 0.2 \cdot fc \cdot Acv$ $Vn2_{mp} = 16$

$Vn3_{ns} := 0.8 \cdot Acv$ $Vn3_{mp} = 16$

$Vn_{ns} := \max\left(\left(\begin{pmatrix} Vn1_{ns} \\ Vn2_{ns} \\ Vn3_{ns} \end{pmatrix}\right)\right)$ $Vn_{mp} = 16$

Applied horizontal shear (k) = $Vh_{ns} := \frac{Vu_{ns}}{dv_{ns}}$ $Vh_{mp} = 4.934$

Width of interface (in) = $bv := fw$

Mimimum amount of reinforcement per foot (in^2) = $Avfmin := \frac{0.05 \cdot bv}{fy}$ $Avfmin = 0.017$

Check actual horizontal shear against the capacity $check1_{ns} := \begin{cases} \text{"OK"} & \text{if } Vn_{ns} > Vh_{ns} \\ \text{"NG"} & \text{otherwise} \end{cases}$

Check minimum reinforcing against the actual $check2_{ns} := \begin{cases} \text{"OK"} & \text{if } Avf_{ns} > Avfmin \\ \text{"NG"} & \text{otherwise} \end{cases}$

$disp := 0$

$disp_{ns,0} := x1_{ns}$ $disp_{ns,1} := Avf_{ns}$ $disp_{ns,2} := Vn_{ns}$ $disp_{ns,3} := Vh_{ns}$ $disp_{ns,4} := Avfmin$ $disp_{ns,5} := check1_{ns}$

$disp_{ns,6} := check2_{ns}$

disp =

	0	1	2	3	4	5	6
0	1	0.224	16	6.476	0.017	"OK"	"OK"
1	1.05	0.201	16	5.962	0.017	"OK"	"OK"
2	1.1	0.2	16	5.448	0.017	"OK"	"OK"
3	1.15	0.137	16	4.934	0.017	"OK"	"OK"
4	1.2	0.114	16	4.364	0.017	"OK"	"OK"
5	1.25	0.1	16	3.856	0.017	"OK"	"OK"
6	1.3	0.115	16	3.348	0.017	"OK"	"OK"
7	1.35	0.104	16	2.834	0.017	"OK"	"OK"
8	1.4	0.1	16	2.103	0.017	"OK"	"OK"
9	1.45	0.1	16	1.669	0.017	"OK"	"OK"
10	1.5	0.1	16	1.234	0.017	"OK"	"OK"
11	1.55	0.1	16	1.669	0.017	"OK"	"OK"
12	1.6	0.1	16	2.103	0.017	"OK"	"OK"
13	1.65	0.104	16	2.834	0.017	"OK"	"OK"
14	1.7	0.115	16	3.348	0.017	"OK"	"OK"
15	1.75	0.1	16	3.856	0.017	"OK"	"OK"
16	1.8	0.114	16	4.364	0.017	"OK"	"OK"
17	1.85	0.137	16	4.934	0.017	"OK"	"OK"
18	1.9	0.2	16	5.448	0.017	"OK"	"OK"
19	1.95	0.201	16	5.962	0.017	"OK"	"OK"
20	2	0.224	16	6.476	0.017	"OK"	"OK"
21							
22							

column 0 = span point
column 1 = actual reinforcing
column 2 = allowable shear
column 3 = applied shear
column 4 = minimum steel
column 5 = capacity check
column 6 = minimum check

22							