CAP-BEAM DESIGN

$mp := 10$

Number of members to be checked = $members := 2$ $ns := 0..members \cdot 11 - 1$

Number of Negative moment points to check = $npa := 2$ $np := 0..npa - 1$

Enter negative moment points to check = $sp_{np} :=$

1.762
2.238

Concrete Strength (ksi) = $fc := 3$

Reinforcing Steel strength (ksi) = $fy := 60$

Factor = $\beta 1 := 0.85$

Clear cover (in) = $cover := 2.5$

Sort output from Structural Analysis

I shall determine the max/min curve for the LL cases. The DC and DW loads shall be input under the Service I and Strength I load combinations

MOMENT

$\begin{pmatrix} m1a \\ m2a \end{pmatrix} :=$

			CASE NUMBER							
point	C1	C2	C3	C4	C5	C6	C7		max	min
1.0	0	0							0	0
1.1	0	0							0	0
1.2	-71.77	-71.31							-71.31	-71.77
1.3	-208.37	-207.03							-207.03	-208.37
1.4	-344.98	-342.75							-342.75	-344.98
1.5	-481.58	-478.47							-478.47	-481.58
1.6	-618.19	-614.19							-614.19	-618.19
1.7	-754.79	-749.91							-749.91	-754.79
1.8	-891.39	-885.63							-885.63	-891.39
1.9	-1028	-1021.4							-1021.35	-1028
2.0	-1164	-1157.1							-1157.07	-1164
2.0	-583.59	-1113.1							-583.59	-1113.1
2.1	-512.47	-977.45							-512.47	-977.45
2.2	-441.35	-841.79							-441.35	-841.79
2.3	-370.22	-706.14							-370.22	-706.14
2.4	-299.1	-570.48							-299.1	-570.48
2.5	-227.98	-434.83							-227.98	-434.83
2.6	-156.85	-299.17							-156.85	-299.17
2.7	-85.73	-163.52							-85.73	-163.52
2.8	-14.61	27.86							27.86	-14.61
2.9	0	0							0	0
3.0	0	0							0	0

$m1_{ns} := m1a_{ns}$

$m2_{ns} := m2a_{ns}$

I shall determine the max/min curve for the LL cases. The DC and DW loads shall be dealt with under the Service I and Strength I load combinations

SHEAR

$\begin{pmatrix} v1a \\ v2a \end{pmatrix} :=$

			CASE NUMBER							
point	C1	C2	C3	C4	C5	C6	C7		max	min
1.0	0	0							0	0
1.1	0	0							0	0
1.2	-105.08	-104.4							-104.4	-105.08
1.3	-105.08	-104.4							-104.4	-105.08
1.4	-105.08	-104.4							-104.4	-105.08
1.5	-105.08	-104.4							-104.4	-105.08
1.6	-105.08	-104.4							-104.4	-105.08
1.7	-105.08	-104.4							-104.4	-105.08
1.8	-105.08	-104.4							-104.4	-105.08
1.9	-105.08	-104.4							-104.4	-105.08
2.0	-105.08	-104.4							-104.4	-105.08
2.0	-54.71	-104.35							-54.71	-104.35
2.1	-54.71	-104.35							-54.71	-104.35
2.2	-54.71	-104.35							-54.71	-104.35
2.3	-54.71	-104.35							-54.71	-104.35
2.4	-54.71	-104.35							-54.71	-104.35
2.5	-54.71	-104.35							-54.71	-104.35
2.6	-54.71	-104.35							-54.71	-104.35
2.7	-54.71	-104.35							-54.71	-104.35
2.8	-54.71	-104.35							-54.71	-104.35
2.9	0	0							0	0
3.0	0	0							0	0

$v1_{ns} := v1a_{ns}$

$v2_{ns} := v2a_{ns}$

Service I loads (moment) (must manually input DC, and DW)

$$\begin{pmatrix} fullm \\ SI1m \\ SI2m \\ SI3m \\ SI4m \\ SI5m \\ SI6m \end{pmatrix} :=$$

	SI1	SI3	SI4	SI5	SI5	SI6
0.0	DC LOADS	DW Loads	LL + I	0	TOTAL LOADS	
LOCATION	self wt	(comp)	M (+)	M (-)	M (+)	M (-)
1.0	0.00	0.00	0.00	0.00	0.00	0.00
1.1	-2.66	0.00	0.00	0.00	-2.66	-2.66
1.2	-94.06	-37.80	-71.31	-71.77	-203.17	-203.63
1.3	-266.12	-109.74	-207.03	-208.37	-582.89	-584.23
1.4	-443.51	-181.68	-342.75	-344.98	-967.94	-970.17
1.5	-626.23	-253.62	-478.47	-481.58	-1358.32	-1361.43
1.6	-814.27	-325.57	-614.19	-618.19	-1754.03	-1758.03
1.7	-1007.63	-397.51	-749.91	-754.79	-2155.05	-2159.93
1.8	-1206.31	-469.45	-885.63	-891.39	-2561.39	-2567.15
1.9	-1410.32	-541.39	-1021.35	-1028.00	-2973.06	-2979.71
2.0	-1619.65	-613.33	-1157.07	-1164.00	-3390.05	-3396.98
2.0	-1569.00	-591.00	-583.59	-1113.10	-2743.59	-3273.10
2.1	-1360.00	-519.00	-512.47	-977.45	-2391.47	-2856.45
2.2	-1156.00	-447.00	-441.35	-841.79	-2044.35	-2444.79
2.3	-957.00	-375.00	-370.22	-706.14	-1702.22	-2038.14
2.4	-764.00	-303.00	-299.10	-570.48	-1366.10	-1637.48
2.5	-576.00	-231.00	-227.98	-434.83	-1034.98	-1241.83
2.6	-393.00	-159.00	-156.85	-299.17	-708.85	-851.17
2.7	-216.00	-87.00	-85.73	-163.52	-388.73	-466.52
2.8	-44.00	-15.00	27.86	-14.61	-31.14	-73.61
2.9	-3.00	0.00	0.00	0.00	-3.00	-3.00
3.0	0.00	0.00	0.00	0.00	0.00	0.00

$(m1 \quad m2)$

Service I loads (shear) (must manually input DC, and DW)

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

	SI1	SI3	SI4	SI5	SI5	SI6
0.0	DC LOADS	DW Loads	LL + I	0	TOTAL LOADS	
LOCATION	self wt	(comp)	M (+)	M (-)	M (+)	M (-)
1.0	0.00	0.00	0.00	0.00	0.00	0.00
1.1	-5.00	0.00	0.00	0.00	-5.00	-5.00
1.2	-131.00	-56.00	-104.40	-105.08	-291.40	-292.08
1.3	-135.00	-56.00	-104.40	-105.08	-295.40	-296.08
1.4	-139.00	-56.00	-104.40	-105.08	-299.40	-300.08
1.5	-143.00	-56.00	-104.40	-105.08	-303.40	-304.08
1.6	-147.00	-56.00	-104.40	-105.08	-307.40	-308.08
1.7	-151.00	-56.00	-104.40	-105.08	-311.40	-312.08
1.8	-155.00	-56.00	-104.40	-105.08	-315.40	-316.08
1.9	-159.00	-56.00	-104.40	-105.08	-319.40	-320.08
2.0	-164.00	-56.00	-104.40	-105.08	-324.40	-325.08
2.0	164.00	56.00	-54.71	-104.35	165.29	115.65
2.1	159.00	56.00	-54.71	-104.35	160.29	110.65
2.2	155.00	56.00	-54.71	-104.35	156.29	106.65
2.3	151.00	56.00	-54.71	-104.35	152.29	102.65
2.4	147.00	56.00	-54.71	-104.35	148.29	98.65
2.5	143.00	56.00	-54.71	-104.35	144.29	94.65
2.6	139.00	56.00	-54.71	-104.35	140.29	90.65
2.7	135.00	56.00	-54.71	-104.35	136.29	86.65
2.8	131.00	56.00	-54.71	-104.35	132.29	82.65
2.9	5.00	56.00	0.00	0.00	61.00	61.00
3.0	0.00	56.00	0.00	0.00	56.00	56.00

$(v1 \ \ v2)$

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.

$\begin{pmatrix} STI1m \\ STI2m \\ STI3m \\ STI4m \\ STI5m \\ STI6m \\ STI7m \end{pmatrix} :=$

	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	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.1	-2.66	0.00	0.00	0.00	0.00	0.00	0.00
1.2	-94.06	0.00	-37.80	-71.31	-71.77	-299.07	-299.87
1.3	-266.12	0.00	-109.74	-207.03	-208.37	-859.56	-861.91
1.4	-443.51	0.00	-181.68	-342.75	-344.98	-1426.72	-1430.62
1.5	-626.23	0.00	-253.62	-478.47	-481.58	-2000.54	-2005.98
1.6	-814.27	0.00	-325.57	-614.19	-618.19	-2581.03	-2588.03
1.7	-1007.63	0.00	-397.51	-749.91	-754.79	-3168.15	-3176.69
1.8	-1206.31	0.00	-469.45	-885.63	-891.39	-3761.92	-3772.00
1.9	-1410.32	0.00	-541.39	-1021.35	-1028.00	-4362.35	-4373.99
2.0	-1619.65	0.00	-613.33	-1157.07	-1164.00	-4969.43	-4981.56
2.0	-1569.00	0.00	-591.00	-583.59	-1113.10	-3869.03	-4795.68
2.1	-1360.00	0.00	-519.00	-512.47	-977.45	-3375.32	-4189.04
2.2	-1156.00	0.00	-447.00	-441.35	-841.79	-2887.86	-3588.63
2.3	-957.00	0.00	-375.00	-370.22	-706.14	-2406.64	-2994.50
2.4	-764.00	0.00	-303.00	-299.10	-570.48	-1932.93	-2407.84
2.5	-576.00	0.00	-231.00	-227.98	-434.83	-1465.47	-1827.45
2.6	-393.00	0.00	-159.00	-156.85	-299.17	-1004.24	-1253.30
2.7	-216.00	0.00	-87.00	-85.73	-163.52	-550.53	-686.66
2.8	-44.00	0.00	-15.00	27.86	-14.61	-0.60	-103.07
2.9	-3.00	0.00	0.00	0.00	0.00	0.00	0.00
3.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00

fullm

Strength I loads (shear)

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.

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

	STI1	STI2	STI3	STI4	STI5	STI6	STI7
	DC LOADS	(non-comp)	DW Loads	LL + I		TOTAL LOADS	
LOCATION	self wt	other (slab)	(comp)	M (+)	M (-)	M (+)	M (-)
1.0	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1.1	-5.00	0.00	0.00	0.00	0.00	0.00	0.00
1.2	-131.00	0.00	-56.00	-104.40	-105.08	-430.45	-431.64
1.3	-135.00	0.00	-56.00	-104.40	-105.08	-435.45	-436.64
1.4	-139.00	0.00	-56.00	-104.40	-105.08	-440.45	-441.64
1.5	-143.00	0.00	-56.00	-104.40	-105.08	-445.45	-446.64
1.6	-147.00	0.00	-56.00	-104.40	-105.08	-450.45	-451.64
1.7	-151.00	0.00	-56.00	-104.40	-105.08	-455.45	-456.64
1.8	-155.00	0.00	-56.00	-104.40	-105.08	-460.45	-461.64
1.9	-159.00	0.00	-56.00	-104.40	-105.08	-465.45	-466.64
2.0	-164.00	0.00	-56.00	-104.40	-105.08	-471.70	-472.89
2.0	164.00	0.00	56.00	-54.71	-104.35	88.26	1.39
2.1	159.00	0.00	56.00	-54.71	-104.35	83.76	-3.11
2.2	155.00	0.00	56.00	-54.71	-104.35	80.16	-6.71
2.3	151.00	0.00	56.00	-54.71	-104.35	76.56	-10.31
2.4	147.00	0.00	56.00	-54.71	-104.35	72.96	-13.91
2.5	143.00	0.00	56.00	-54.71	-104.35	69.36	-17.51
2.6	139.00	0.00	56.00	-54.71	-104.35	65.76	-21.11
2.7	135.00	0.00	56.00	-54.71	-104.35	62.16	-24.71
2.8	131.00	0.00	56.00	-54.71	-104.35	58.56	-28.31
2.9	5.00	0.00	56.00	0.00	0.00	0.00	0.00
3.0	0.00	0.00	56.00	0.00	0.00	0.00	0.00

fullv

Final Service Load Data to Use

SL :=

range	V+	V-	M+	M-
1	0	0	0	0
1.1	-5	-5	-2.66	-2.66
1.2	-291.4	-292.08	-203.17	-203.63
1.3	-295.4	-296.08	-582.89	-584.23
1.4	-299.4	-300.08	-967.94	-970.17
1.5	-303.4	-304.08	-1358.32	-1361.43
1.6	-307.4	-308.08	-1754.03	-1758.03
1.7	-311.4	-312.08	-2155.05	-2159.93
1.8	-315.4	-316.08	-2561.39	-2567.15
1.9	-319.4	-320.08	-2973.06	-2979.71
2	-324.4	-325.08	-3390.05	-3396.98
2	165.29	115.65	-2743.59	-3273.1
2.1	160.29	110.65	-2391.47	-2856.45
2.2	156.29	106.65	-2044.35	-2444.79
2.3	152.29	102.65	-1702.22	-2038.14
2.4	148.29	98.65	-1366.1	-1637.48
2.5	144.29	94.65	-1034.98	-1241.83
2.6	140.29	90.65	-708.85	-851.17
2.7	136.29	86.65	-388.73	-466.52
2.8	132.29	82.65	-31.14	-73.61
2.9	61	61	-3	-3
3	56	56	0	0

$(\mathrm{SI5v}\quad \mathrm{SI6v}\quad \mathrm{SI5m}\quad \mathrm{SI6m})$

$$V\_SL_{ns} := \max\left(\begin{pmatrix} \left|SL_{ns,1}\right| \\ \left|SL_{ns,2}\right| \end{pmatrix}\right)$$

$$M\_SLp_{ns} := \max\left(\begin{pmatrix} SL_{ns,3} \\ SL_{ns,4} \end{pmatrix}\right)$$

$$M\_SLn_{ns} := \min\left(\begin{pmatrix} SL_{ns,3} \\ SL_{ns,4} \end{pmatrix}\right)$$

Final Load Factor Data to Use

LD :=

range	V+	V-	M+	M-
1	0	0	0	0
1.1	0	0	0	0
1.2	-430.45	-431.64	-299.068	-299.873
1.3	-435.45	-436.64	-859.563	-861.908
1.4	-440.45	-441.64	-1426.72	-1430.62
1.5	-445.45	-446.64	-2000.54	-2005.98
1.6	-450.45	-451.64	-2581.03	-2588.03
1.7	-455.45	-456.64	-3168.15	-3176.69
1.8	-460.45	-461.64	-3761.92	-3772
1.9	-465.45	-466.64	-4362.35	-4373.99
2	-471.7	-472.89	-4969.43	-4981.56
2	88.2575	1.3875	-3869.03	-4795.68
2.1	83.7575	-3.1125	-3375.32	-4189.04
2.2	80.1575	-6.7125	-2887.86	-3588.63
2.3	76.5575	-10.3125	-2406.64	-2994.5
2.4	72.9575	-13.9125	-1932.93	-2407.84
2.5	69.3575	-17.5125	-1465.47	-1827.45
2.6	65.7575	-21.1125	-1004.24	-1253.3
2.7	62.1575	-24.7125	-550.528	-686.66
2.8	58.5575	-28.3125	-0.595	-103.068
2.9	0	0	0	0
3	0	0	0	0

$(STI6v \quad STI7v \quad STI6m \quad STI7m)$

$$V\_LD_{ns} := \max\left(\begin{pmatrix} \left|LD_{ns,1}\right| \\ \left|LD_{ns,2}\right| \end{pmatrix}\right)$$

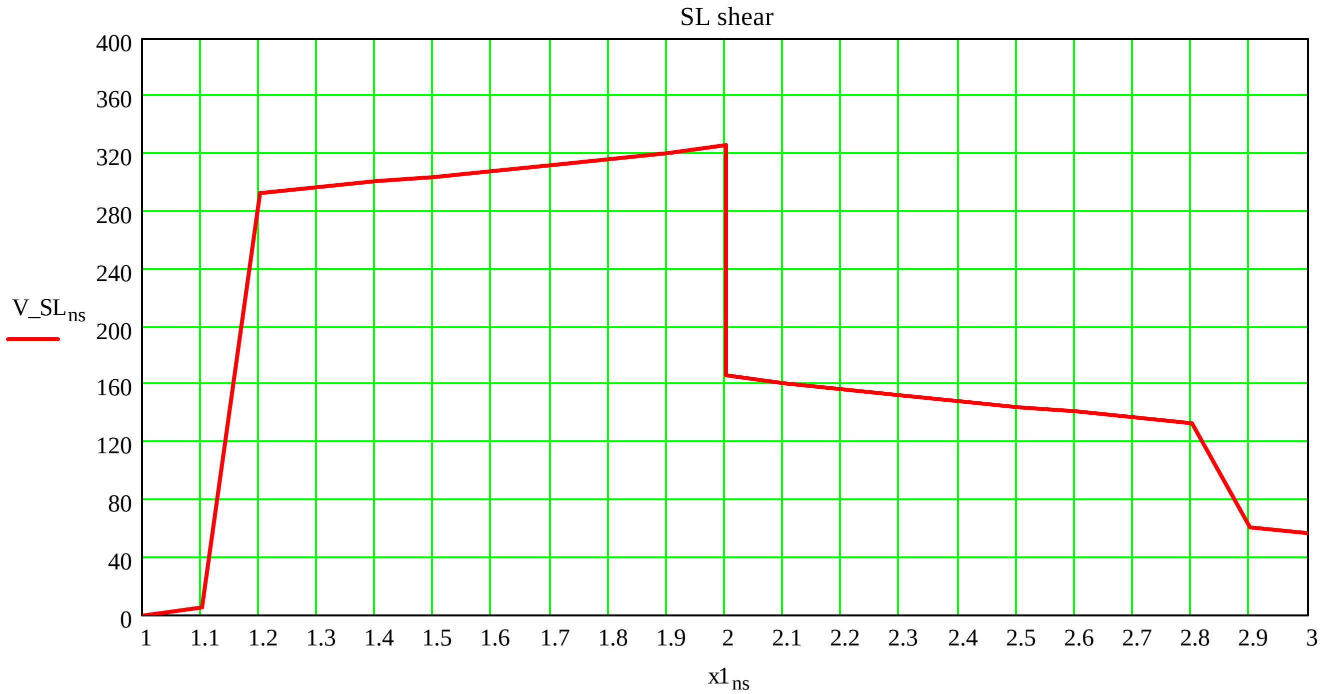
$$M\_LDp_{ns} := \max\left(\begin{pmatrix} LD_{ns,3} \\ LD_{ns,4} \end{pmatrix}\right)$$

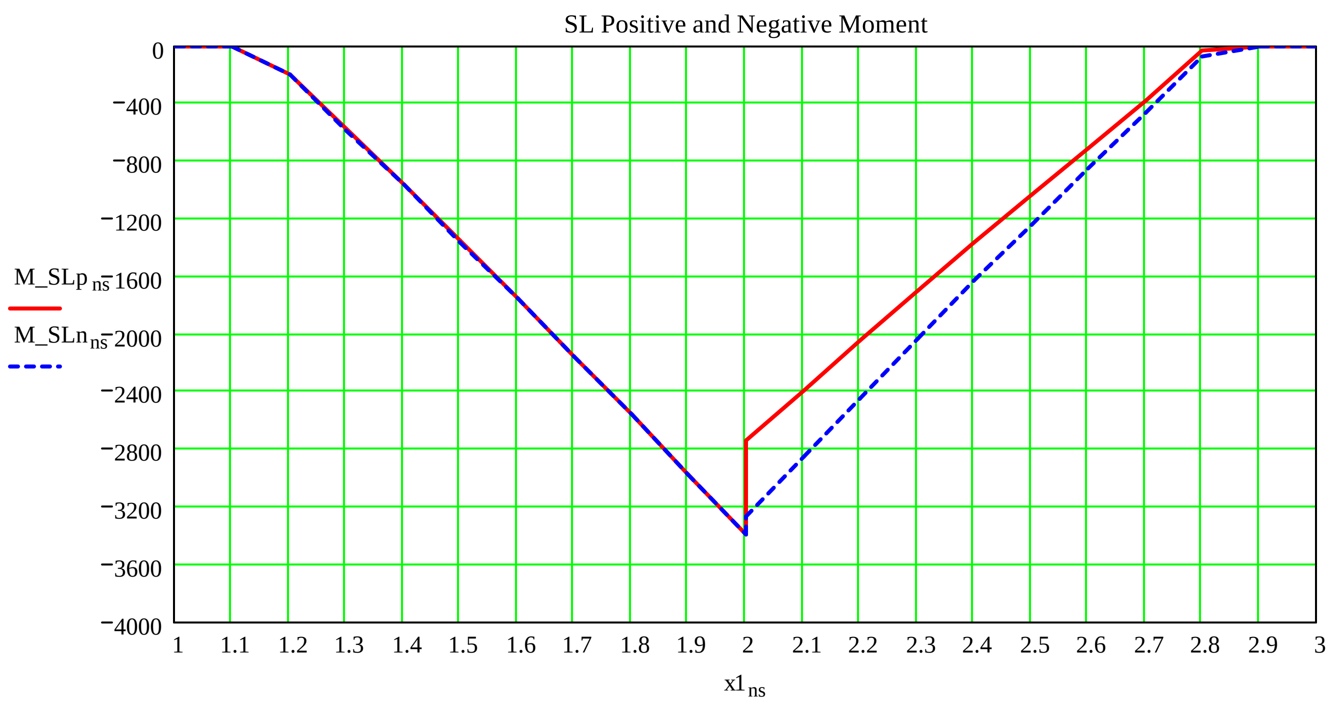
$$M\_LDn_{ns} := \min\left(\begin{pmatrix} LD_{ns,3} \\ LD_{ns,4} \end{pmatrix}\right)$$

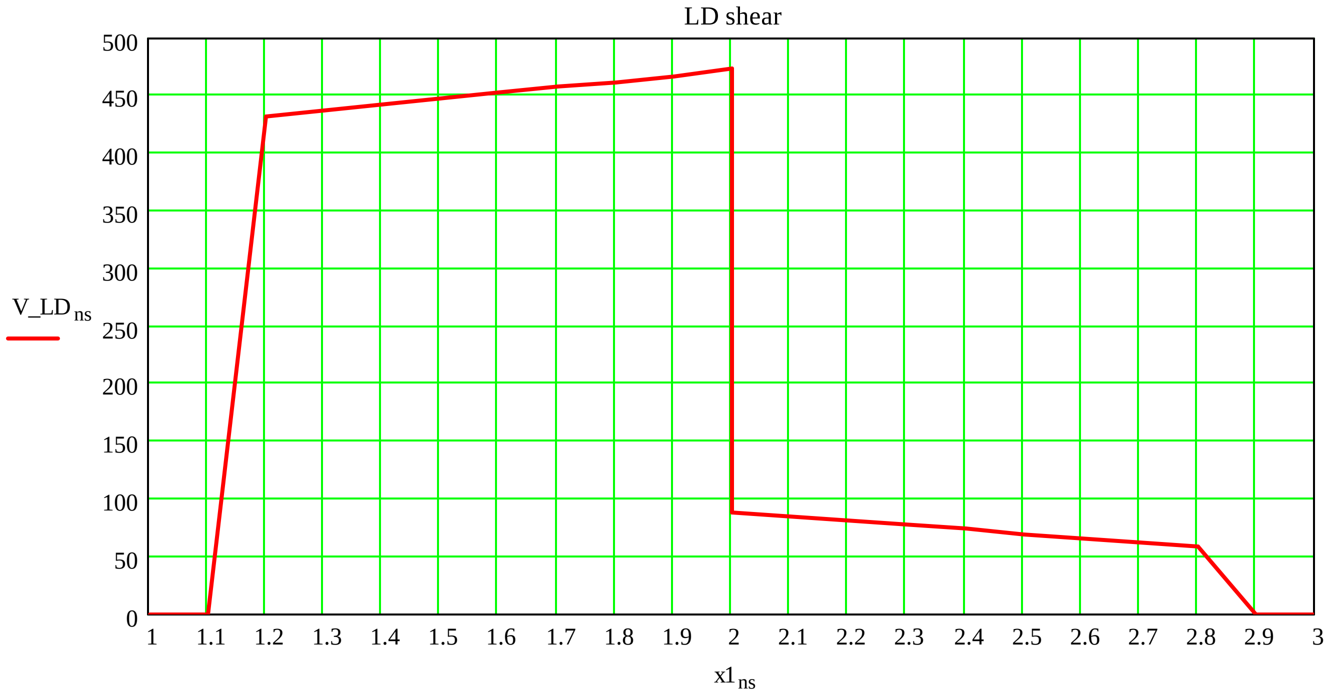
$$SL_{members \cdot 11, 0} := 0$$

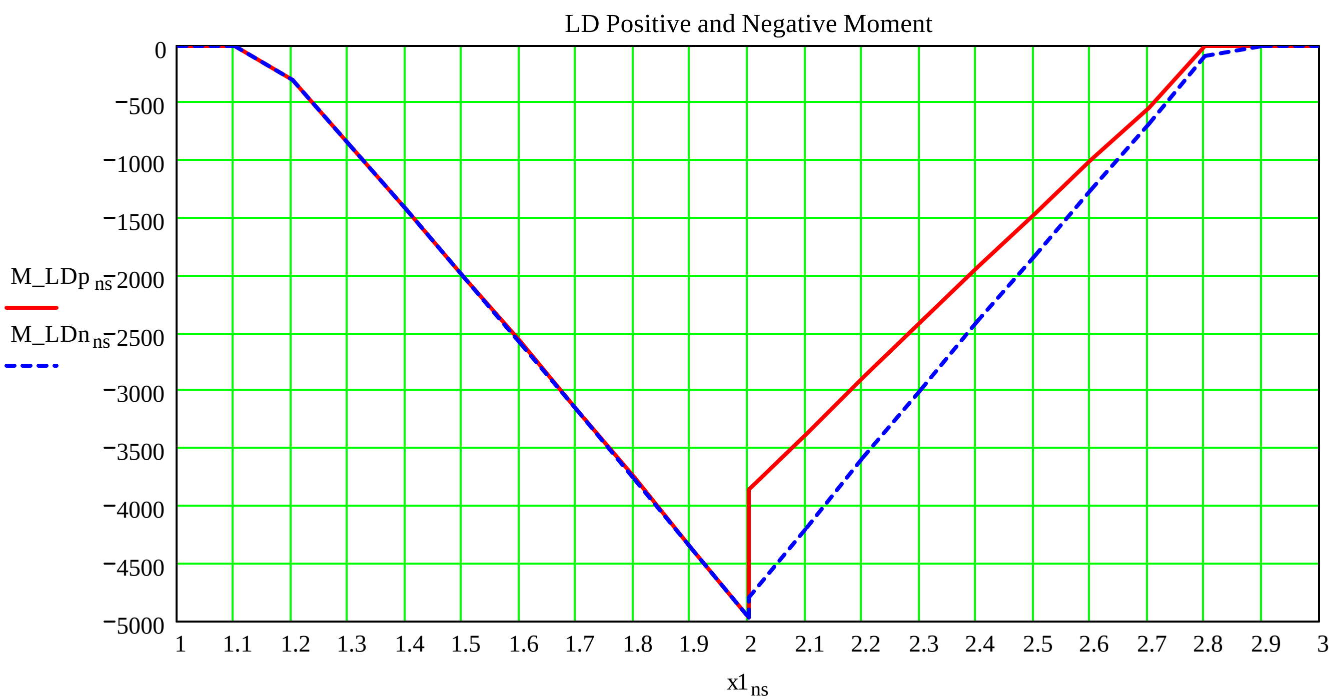
Shear and Moment Graphs

$$x1_{ns} := SL_{ns,0}$$






LD shear
500
450
400
350
300
250
200
150
100
50
0
V_LD ns
1 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 1.9 2 2.1 2.2 2.3 2.4 2.5 2.6 2.7 2.8 2.9 3
x1 ns


LD Positive and Negative Moment
0
−500
−1000
−1500
−2000
−2500
−3000
−3500
−4000
−4500
−5000
M_LDp ns
M_LDn ns
1 1.1 1.2 1.3 1.4 1.5 1.6 1.7 1.8 1.9 2 2.1 2.2 2.3 2.4 2.5 2.6 2.7 2.8 2.9 3
x1 ns

Determine the first matrix point for the iterated value (the second will simply by the first +1)

$$av_{np} := \left| \begin{array}{l} \text{for } j \in 0..npa-1 \\ \quad \text{for } j1 \in 0..members \cdot 11 - 1 \\ \quad\quad \left| \begin{array}{l} (j2a_j \leftarrow j1)\cdot(\text{break}) \;\; \text{if } \left(SL_{j1,0} < sp_j\right)\cdot\left(SL_{j1+1,0} > sp_j\right) \\ j2a_j \leftarrow \text{"hi"} \;\; \text{otherwise} \end{array} \right. \\ j2a_{np} \end{array} \right.$$

$$av_{npa} := 0$$

Give each span point a theoretical matrix span point

$$a2_{np} := SL_{av_{np},0} \qquad a2_{npa} := 0$$

$$a1_{np} := av_{np} + \frac{\left(av_{np+1} - av_{np}\right)\cdot\left(sp_{np} - a2_{np}\right)}{a2_{np+1} - a2_{np}}$$

Calculate actual iterated value for SL shear

$$Vsl_{np} := V\_SL_{\left(av_{np}\right)} + \frac{\left[V\_SL_{av_{np}+1} - V\_SL_{\left(av_{np}\right)}\right]\cdot\left(a1_{np} - av_{np}\right)}{av_{np+1} - av_{np}} \qquad Vsl = \begin{pmatrix} 312.576 \\ 156.359 \end{pmatrix}$$

Calculate actual iterated value for LD shear

$$Vld_{np} := V\_LD_{\left(av_{np}\right)} + \frac{\left[V\_LD_{av_{np}+1} - V\_LD_{\left(av_{np}\right)}\right]\cdot\left(a1_{np} - av_{np}\right)}{av_{np+1} - av_{np}} \qquad Vld = \begin{pmatrix} 457.26 \\ 80.22 \end{pmatrix}$$

Calculate actual iterated value for negative SL moment

$$Msl_{np} := M\_SLn_{\left(av_{np}\right)} + \frac{\left[M\_SLn_{av_{np}+1} - M\_SLn_{\left(av_{np}\right)}\right]\cdot\left(a1_{np} - av_{np}\right)}{av_{np+1} - av_{np}} \qquad Msl = \begin{pmatrix} -2210.425 \\ -2451.814 \end{pmatrix}$$

Calculate actual iterated value for negative LD moment

$$Mld_{np} := M\_LDn_{\left(av_{np}\right)} + \frac{\left[M\_LDn_{av_{np}+1} - M\_LDn_{\left(av_{np}\right)}\right]\cdot\left(a1_{np} - av_{np}\right)}{av_{np+1} - av_{np}} \qquad Mld = \begin{pmatrix} -3250.503 \\ -3598.895 \end{pmatrix}$$

Design Shear

Design SL shear (k) = $VSL := \max\left(Vsl^{\langle 0 \rangle}\right)$ $VSL = 312.576$

Design LD shear (k) = $VLD := \max\left(Vld^{\langle 0 \rangle}\right)$ $VLD = 457.26$

Design Moment

SL negative moment (k*ft) = $MSLn := \left|\min\left(Msl^{\langle 0 \rangle}\right)\right|$ $MSLn = 2451.814$

LD negative moment (k*ft) = $MLDn := \left|\min\left(Mld^{\langle 0 \rangle}\right)\right|$ $MLDn = 3598.895$

SL positive moment (k*ft) = $MSLp := \max\left(M\_SLp^{\langle 0 \rangle}\right)$ $MSLp = 0$

LD positive moment (k*ft) = $MLDp := \max\left(M\_LDp^{\langle 0 \rangle}\right)$ $MLDp = 0$

Strength Design Positive Moment

Section Depth for positive moment (in) = $b1 := 72$

Section Width for positive moment (in) = $b2 := 42$

Concrete Strength (psi) = $fc = 3$

Reinforcing Steel strength (psi) = $fy = 60$

Number of rows of steel = $rowsp := 1$

Effective depth

$$dp := b1 - 3 - \frac{(rowsp - 1)\cdot 3}{2} \qquad dp = 69$$

Section Modulus (in^3) $$Stc := \frac{b2\cdot b1^3}{12}\cdot\frac{1}{b1\cdot 0.5} \qquad Stc = 36288$$

Design positive moment (k*ft) = $MLDp = 0$

LRFD 5.7.3.3.2 minimum reinforcement

Unless otherwise 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 lesser of

1.2* the cracking strength
1.33 times the factored moment required by Strength I

What I shall do is calculate required steel based on the cracking moment. Then I shall calculate the required steel based on strength. If any of the reqired are less than 1.2*Mcr I shall increase the required by 4/3.

LRFD 5.4.2.6 the modulus of rupture (ksi) = $fr := 0.24\cdot\sqrt{fc}$ $fr = 0.416$

Cracking moment (k*ft) = $Mcr := \frac{fr\cdot Stc}{12}$ $Mcr = 1257.053$

Effective section depth (in) = $dp = 69$

$Acr := 0$ $As := 0$ $Ar := 0$ $Art := 0$

Required steel based on 1.2*Mcr (in^2) = $Acr := root\left[\left|12\cdot Mcr\right| - 0.9\cdot Acr\cdot fy\cdot\left(dp - \frac{Acr\cdot fy}{1.7\cdot fc\cdot b2}\right), Acr\right]$

$Acr = 4.117$

CALCULATIONS BASED ON RECTANGULAR SECTION

Required steel (pure strength)(in^2) = $Asp1 := root\left[\left|12\cdot MLDp\right| - 0.9\cdot Ar\cdot fy\cdot\left(dp - \frac{Ar\cdot fy}{1.7\cdot fc\cdot b2}\right), Ar\right]$ $Asp1 = 0$

Required Steed (in^2) = $Asp := \begin{cases} \frac{4}{3}\cdot Asp1 & \text{if } Asp1 < Acr \\ Asp1 & \text{otherwise} \end{cases}$ $Asp = 0$

Distance between the N.A. and the compression flange (in) = $c1 := \frac{Asp\cdot fy}{0.85\cdot fc\cdot \beta1\cdot b2}$ $c1 = 0$

BAR PATTERN REQUIRMENTS $n8 := 0..6$

$$bars := \begin{pmatrix} 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 11 \end{pmatrix} \quad Asone := \begin{pmatrix} 0.31 \\ 0.44 \\ 0.60 \\ 0.79 \\ 1.0 \\ 1.27 \\ 1.56 \end{pmatrix} \quad bd := \begin{pmatrix} 0.625 \\ 0.75 \\ 0.875 \\ 1.0 \\ 1.128 \\ 1.27 \\ 1.41 \end{pmatrix}$$

number of bars total minimum

$$\mathrm{floor}\left(\frac{Asp}{Asone}\right) + 1 = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix}$$

Number of bars per row, and spacing (natural)

$$nbarsp_{n8} := \mathrm{ceil}\left(\frac{\mathrm{floor}\left(\frac{Asp}{Asone_{n8}}\right) + 1}{rowsp}\right) \qquad sbarsp_{n8} := \frac{b2 - 6}{nbarsp_{n8} - 1}$$

$$nbarsp = \begin{pmatrix} 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \\ 1 \end{pmatrix} \qquad sbarsp = ▮$$

choose bar size $bsp := 10$ $cp := bsp - 5$ $cp = 5$

chose number of bars $np := 12$

number of rows $rowsp = 1$

Spacing of bars in each row

$$sbarsp := \frac{b2 - 6}{\frac{np}{rowsp} - 1} \qquad sbarsp = 3.273$$

$Asp := Asone_{cp} \cdot np$ $Asp = 15.24$

$nbarsp := np$ $nbarsp = 12$

$sbarsp := sbarsp$ $sbarsp = 3.273$

$bd := bd_{cp}$ $bd = 1.27$

LRFD 5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

The provisions specified herein shall apply to the reinforcement of all concrete components, except that of deck slabs designed in accordance with Article 9.7.2, in which tension in the cross-section exceeds 80 percent of the modulus of rupture, specified in article 5.4.2.6, at applicable service limit state load combination specified in Table 3.4.1-1.

Since this is the main negative moment reinforcement parallel to traffic I shall apply the provisions of this article.

Components shall be so proportioned that the tensile stress in the mild steel reinforcement at the service limit state, fsa does not exceed:

$$fsa = \frac{Z}{(dc \cdot A)^{\frac{1}{3}}} \le 0.6 \cdot fy \qquad 5.7.3.4\text{-}1$$

dc = depth of concrete measured from extreme tension fiber to center of bar or wire located closest thereto; for calculation purposes, the thickness of clear coner used to compute dc shall not be taken to be greater than 2 in.

A = area fo concrete having the same centroid as the principal tensile reinforcement and bounded by the surfaces of hte cross-section and a straight line parallel to the neutral axis, divided by teh number of bars or wires; for calculation purposes the thickness of clear concrete cover used to compute A shall not be taken to be greater than 2.0 in.

Z = crack width parameter - use 170 k/in for moderate exposure.

Number of bars = $nbarsp = 12$

Area of steel (in^2) = $Asp = 15.24$

SL moment to be used (use 0 if the moment is positive) $MSLp = 0$

$F := 0$

Stress in steel for rectangular section

SL Stress in steel (psi) = $fsp := root\left[12 \cdot MSLp - Asp \cdot F \cdot \left(dp - \frac{Asp \cdot F}{1.7 \cdot fc \cdot b2}\right), F\right]$ $fsp = 0$

Allowable Stress

Cracking factor = $z := 170$

Total Section Height (in) = $\text{height} := b1$ $\text{height} = 72$

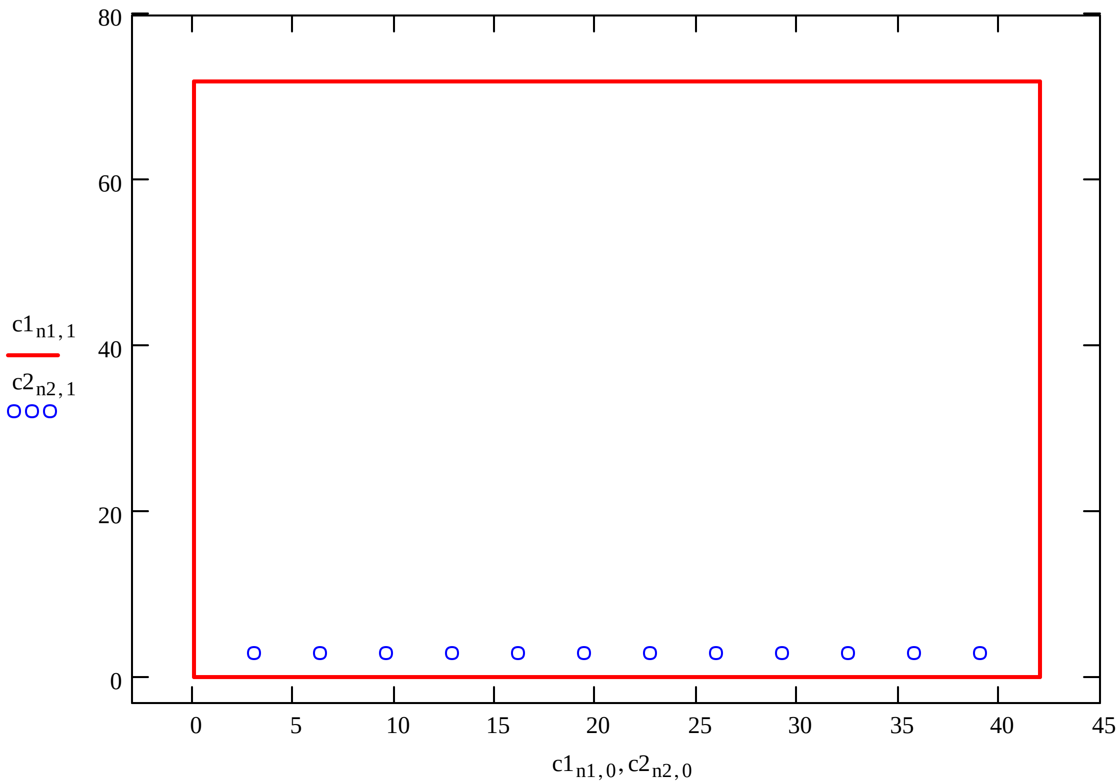
Distance from extreme tension fiber to CL. reinforcing= $dc := \min\left(\left(\frac{\text{cover}}{2}\right)\right) + 0.75 + \frac{bd}{2}$ $dc = 3.385$

Effective tension flange area $Ap := \frac{2 \cdot dc \cdot b2}{\text{nbarsp}}$ $Ap = 23.695$

Allowable Stress $fsap := \min\left[\left[\begin{matrix} \dfrac{z}{(dc \cdot Ap)^{\frac{1}{3}}} \\ 0.6 \cdot fy \end{matrix}\right]\right]$ $fsap = 36$

Check steel for cracking moment $ckp := \begin{cases} \text{"fail"} & \text{if } fsap < fsp \\ \text{"pass"} & \text{otherwise} \end{cases}$ $ckp = \text{"pass"}$

Diagram of Positive moment section



Size of bars used = $bsp = 10$

Total number of bars = $nbarsp = 12$

Spacing of bars (in) = $sbarsp = 3.273$

Total Section Depth (in) = $b1 = 72$

Total Section width (in) = $b2 = 42$

Design Negative Moment

Section Depth for negative moment (in) = $c1 := 72$

Section Width for negative moment (in) = $c2 := 42$

Concrete Strength (psi) = $fc = 3$

Reinforcing Steel strength (psi) = $fy = 60$

Number of rows of steel = $rowsn := 1$

Effective depth

$$dn := c1 - 3 - \frac{(rowsn - 1)\cdot 3}{2} \qquad dn = 69$$

Section Modulus (in^3) $$Stc := \frac{c2\cdot c1^{3}}{12}\cdot\frac{1}{c1\cdot 0.5} \qquad Stc = 36288$$

Design negative moment (k*ft) = $MLDn = 3598.895$

LRFD 5.7.3.3.2 minimum reinforcement

Unless otherwise 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 lesser of

1.2* the cracking strength
1.33 times the factored moment required by Strength I

What I shall do is calculate required steel based on the cracking moment. Then I shall calculate the required steel based on strength. If any of the reqired are less than 1.2*Mcr I shall increase the required by 4/3.

LRFD 5.4.2.6 the modulus of rupture (ksi) = $fr := 0.24\cdot\sqrt{fc}$ $fr = 0.416$

Cracking moment (k*ft) = $Mcr := \dfrac{fr\cdot Stc}{12}$ $Mcr = 1257.053$

Effective section depth (in) = $dp = 69$

$Acr := 0$ $As := 0$ $Ar := 0$ $Art := 0$

Required steel based on 1.2*Mcr (in^2) = $Acr := root\left[\left|12\cdot Mcr\right| - 0.9\cdot Acr\cdot fy\cdot\left(dn - \dfrac{Acr\cdot fy}{1.7\cdot fc\cdot c2}\right), Acr\right]$

$Acr = 4.117$

CALCULATIONS BASED ON RECTANGULAR SECTION

Required steel (pure strength)(in^2) = $Asn1 := root\left[\left|12\cdot MLDn\right| - 0.9\cdot Ar\cdot fy\cdot\left(dn - \dfrac{Ar\cdot fy}{1.7\cdot fc\cdot c2}\right), Ar\right]$ $Asn1 = 12.194$

Required Steed (in^2) = $Asn := \begin{cases} \dfrac{4}{3}\cdot Asn1 & \text{if } Asn1 < Acr \\ Asn1 & \text{otherwise} \end{cases}$ $Asn = 12.194$

Distance between the N.A. and the compression flange (in) = $c1a := \dfrac{Asn\cdot fy}{0.85\cdot fc\cdot \beta 1\cdot c2}$ $c1a = 8.037$

BAR PATTERN REQUIRMENTS

$n8 := 0..6$

$$bars := \begin{pmatrix} 5 \\ 6 \\ 7 \\ 8 \\ 9 \\ 10 \\ 11 \end{pmatrix} \quad Asone := \begin{pmatrix} 0.31 \\ 0.44 \\ 0.60 \\ 0.79 \\ 1.0 \\ 1.27 \\ 1.56 \end{pmatrix} \quad bd := \begin{pmatrix} 0.625 \\ 0.75 \\ 0.875 \\ 1.0 \\ 1.128 \\ 1.27 \\ 1.41 \end{pmatrix}$$

number of bars total minimum

$$\mathrm{floor}\left(\frac{Asn}{Asone}\right) + 1 = \begin{pmatrix} 40 \\ 28 \\ 21 \\ 16 \\ 13 \\ 10 \\ 8 \end{pmatrix}$$

Number of bars per row, and spacing (natural)

$$nbarsn_{n8} := \mathrm{ceil}\left(\frac{\mathrm{floor}\left(\frac{Asn}{Asone_{n8}}\right) + 1}{rowsn}\right) \qquad sbarsn_{n8} := \frac{c2 - 6}{nbarsn_{n8} - 1}$$

$$nbarsn = \begin{pmatrix} 40 \\ 28 \\ 21 \\ 16 \\ 13 \\ 10 \\ 8 \end{pmatrix} \qquad sbarsn = \begin{pmatrix} 0.923 \\ 1.333 \\ 1.8 \\ 2.4 \\ 3 \\ 4 \\ 5.143 \end{pmatrix}$$

choose bar size $bsn := 11$ $\quad cp := bsn - 5 \quad cp = 6$

chose number of bars $nn := 9$

number of rows $rowsn = 1$

Spacing of bars in each row

$$sbarsn := \frac{c2 - 6}{\frac{nn}{rowsn} - 1} \qquad sbarsn = 4.5$$

$Asn := Asone_{cp} \cdot nn \qquad Asn = 14.04$

$nbarsn := nn \qquad nbarsn = 9$

$sbarsn := sbarsn \qquad sbarsn = 4.5$

$bd := bd_{cp} \qquad bd = 1.41$

LRFD 5.7.3.4 CONTROL OF CRACKING BY DISTRIBUTION OF REINFORCEMENT

The provisions specified herein shall apply to the reinforcement of all concrete components, except that of deck slabs designed in accordance with Article 9.7.2, in which tension in the cross-section exceeds 80 percent of the modulus of rupture, specified in article 5.4.2.6, at applicable service limit state load combination specified in Table 3.4.1-1.

Components shall be so proportioned that the tensile stress in the mild steel reinforcement at the service limit state, fsa does not exceed:

$$fsa = \frac{Z}{(dc \cdot A)^{\frac{1}{3}}} \le 0.6 \cdot fy \qquad 5.7.3.4\text{-}1$$

dc = depth of concrete measured from extreme tension fiber to center of bar or wire located closest there to; for calculation purposes, the thickness of clear cover used to compute dc shall not be taken to be greater than 2 in.

A = area fo concrete having the same centroid as the principal tensile reinforcement and bounded by the surfaces of the cross-section and a straight line parallel to the neutral axis, divided by teh number of bars or wires; for calculation purposes the thickness of clear concrete cover used to compute A shall not be taken to be greater than 2.0 in.

Z = crack width parameter - use 170 k/in for moderate exposure.

Number of bars = $nbarsn = 9$

Area of steel (in^2) = $Asn = 14.04$

SL moment to be used (use 0 if the moment is positive) $MSLn = 2451.814$

$F := 0$

Stress in steel for rectangular section

SL Stress in steel (psi) = $fsn := root\left[12 \cdot MSLn - 0.9Asn \cdot F \cdot \left(dn - \frac{Asn \cdot F}{1.7 \cdot fc \cdot c2}\right), F\right]$ $fsn = 34.902$

Allowable Stress

Cracking factor = $z := 170$

Total Section Height (in) = $height := c1$ $height = 72$

Distance from extreme tension fiber to CL. reinforcing= $dc := \min\left(\left(\dfrac{3 - bd\cdot 0.5}{2}\right)\right) + \dfrac{bd}{2}$ $dc = 2.705$

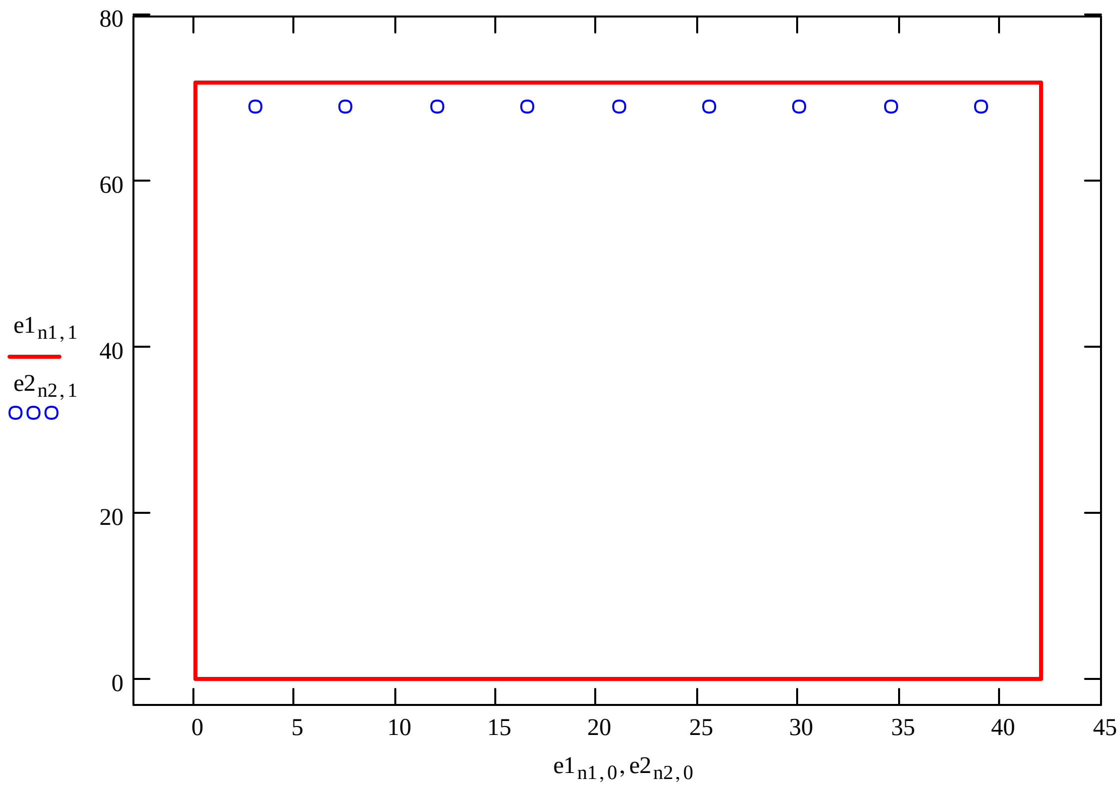
Effective tension flange area $An := \dfrac{2\cdot dc\cdot c2}{nbarsn}$ $An = 25.247$

Allowable Stress $fsan := \min\left[\left[\begin{matrix} \dfrac{z}{(dc\cdot An)^{\frac{1}{3}}} \\ 0.6\cdot fy \end{matrix}\right]\right]$ $fsan = 36$

Actual Stress (ksi) = $fsn = 34.902$

Check steel for cracking moment $ckn := \begin{cases} \text{"fail"} & \text{if } fsan < fsn \\ \text{"pass"} & \text{otherwise} \end{cases}$ $ckn = \text{"pass"}$

Diagram of Negative moment section



Size of bars used = $bsn = 11$

Total number of bars = $nbarsn = 9$

Spacing of bars (in) = $sbarsn = 4.5$

Total Section Depth (in) = $c1 = 72$

Total Section width (in) = $c2 = 42$

Shear Design

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 $\quad dv = \frac{Mn}{As\cdot fy + Aps\cdot fps}$

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

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

$$Vs = \frac{Av\cdot fy\cdot dv\cdot\cot(\theta)}{S}$$ 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

Width of section (in) = $bv := 42$

Effective depth (in) = $de := \min\left(\begin{pmatrix} dn \\ dp \end{pmatrix}\right)$ $de = 69$

Effective tension steel (in^2) = $As := Asn$ $As = 14.04$

Effective shear depth (in) = $dv := de - \dfrac{As \cdot fy}{1.7 \cdot fc \cdot bv}$ $dv = 65.067$

Define factored shear (k) = $Vu := VLD$ $Vu = 457.26$

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

Shear stress on the concrete (ksi) = $V := \dfrac{Vu}{\phi \cdot bv \cdot dv}$ $V = 0.186$

Maximum spacing of transverse reinforcing (in) = $Smax := \begin{vmatrix} \min\left(\begin{pmatrix} 0.8 \cdot dv \\ 24.0 \end{pmatrix}\right) & \text{if } V < 0.125 \cdot fc \\ \min\left(\begin{pmatrix} 0.4 \cdot dv \\ 12.0 \end{pmatrix}\right) & \text{otherwise} \end{vmatrix}$ $Smax = 24$

LRFD 5.8.3.4 Determination of b **and** q

For non-prestressed concrete sections not subjected to axial tension and containing at least the minimum amount of transverse reinforcement specified in LRFD 5.8.2.5, or having an overall depth of less than 16 in, the following values may be used.

$\beta := 2.0$

$\theta := 45$

Range definition $ns1 := 0..2$

Area of double no. 5 bars (in^2)= $Av_0 := 1.24$

Area of double no. 6 bars (in^2) = $Av_1 := 1.76$

Area of double no. 7 bars (in^2) = $Av_2 := 2.4$

Nominal resistance of concrete (k) = $Vc := 0.0316 \cdot \beta \cdot \sqrt{fc} \cdot bv \cdot dv$ $Vc = 299.15$

Calculate required nominal steel strength (k) =

$$Vs := \left| \begin{array}{l} j \leftarrow \dfrac{Vu}{\phi} - Vc \\ 0 \ \text{ if } \ Vu < 0.5\cdot\phi\cdot(Vc) \\ \text{otherwise} \\ \quad \left| \begin{array}{l} 0 \ \text{ if } \ j < 0 \\ j \ \text{ otherwise} \end{array} \right. \end{array} \right.$$

$Vs = 208.916$

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

$$S1_{ns1} := \left| \begin{array}{l} Smax \ \text{ if } \ Vs = 0 \\ j \leftarrow \dfrac{Av_{ns1}\cdot fy\cdot dv\cdot \cot\left(\theta\cdot\dfrac{\pi}{180}\right)}{Vs} \ \text{ otherwise} \end{array} \right.$$

actual required spacing

$$S1 = \begin{pmatrix} 23.172 \\ 32.889 \\ 44.849 \end{pmatrix}$$

maximum allowable spacing

$Smax = 24$

$$Av = \begin{pmatrix} 1.24 \\ 1.76 \\ 2.4 \end{pmatrix}$$

Spacing of stirrups considering the max allowable

Actual spacing to use (in) =

$$S_{ns1} := \left| \begin{array}{l} Smax \ \text{ if } \ Vs = 0 \\ \text{otherwise} \\ \quad \left| \begin{array}{l} j \leftarrow \dfrac{Av_{ns1}\cdot fy\cdot dv\cdot \cot\left(\theta\cdot\dfrac{\pi}{180}\right)}{Vs} \\ Smax \ \text{ if } \ j > Smax \\ j \ \text{ otherwise} \end{array} \right. \end{array} \right.$$

$$S = \begin{pmatrix} 23.172 \\ 24 \\ 24 \end{pmatrix}$$