PROJECT: I-75 Costing Plans

AASHTO STANDARD Load: Self

Coord. system: Global Units: kips kips-ft

| Memb | Node | Fx | Fy | Fz | Mx | My | Mz |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -0 | 228.7 | 0 | -0 | 0 | 0 |
| 1 | 2 | -0 | -156.9 | 0 | -0 | 0 | 0 |
| 2 | 3 | -0 | 228.9 | 0 | -0 | 0 | 0 |
| 2 | 4 | -0 | -157.1 | 0 | -0 | 0 | 0 |
| 3 | 5 | 0 | 0.5455 | 0 | 0 | 0 | -1.093 |
| 3 | 2 | 0 | 16.93 | 0 | 0 | 0 | -4.369 |
| 4 | 2 | 0 | 139.7 | 0 | 0 | 0 | 0 |
| 4 | 6 | 0 | -122.2 | 0 | 0 | 0 | 392.8 |
| 5 | 6 | 0 | 122.3 | 0 | 0 | 0 | -392.1 |
| 5 | 7 | 0 | -116.5 | 0 | 0 | 0 | 511.5 |
| 6 | 7 | 0 | 116.5 | 0 | 0 | 0 | -511.5 |
| 6 | 8 | 0 | -69.9 | 0 | 0 | 0 | 1257 |
| 7 | 8 | 0 | 69.9 | 0 | 0 | 0 | -1257 |
| 7 | 9 | 0 | -23.3 | 0 | 0 | 0 | 1630 |
| 8 | 9 | 0 | 23.3 | 0 | 0 | 0 | -1630 |
| 8 | 10 | 0 | 23.3 | 0 | 0 | 0 | 1630 |
| 9 | 10 | 0 | -23.3 | 0 | 0 | 0 | -1630 |
| 9 | 11 | 0 | 69.9 | 0 | 0 | 0 | 1257 |
| 10 | 11 | 0 | -69.9 | 0 | 0 | 0 | -1257 |
| 10 | 12 | 0 | 122.3 | 0 | 0 | 0 | 391.9 |
| 11 | 12 | 0 | -122.5 | 0 | 0 | 0 | -393.7 |
| 11 | 4 | 0 | 140 | 0 | 0 | 0 | 0 |
| 12 | 4 | 0 | 18.98 | 0 | 0 | 0 | 4.369 |
| 12 | 13 | 0 | -1.503 | 0 | 0 | 0 | 6.554 |

PROJECT: I-75 Costing Plans

AASHTO STANDARD Load: D1-Include Add'l Cap DL

Coord. system: Global Units: kips kips-ft

| Memb | Node | Fx | Fy | Fz | Mx | My | Mz |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -0 | 825.5 | 0 | 0.04014 | 0.0005007 | 0 |
| 1 | 2 | -0 | -825.5 | 0 | -0.04052 | -0.0005007 | 0 |
| 2 | 3 | -0 | 575.7 | 0 | 0.03535 | 0.0005007 | 0 |
| 2 | 4 | -0 | -575.7 | 0 | -0.03497 | -0.0005007 | 0 |
| 3 | 5 | 0 | 0.372 | 0 | 0 | 0 | 1.381 |
| 3 | 2 | 0 | -5.772 | 0 | 0 | 0 | -0.01533 |
| 4 | 2 | 0 | 830.8 | 0 | 0.04052 | 0.0005007 | 0 |
| 4 | 6 | 0 | -836.2 | 0 | -0.04052 | -0.0004381 | 2501 |
| 5 | 6 | 0 | 836.2 | 0 | 0.04052 | 0.0004381 | -2501 |
| 5 | 7 | 0 | -838 | 0 | -0.04052 | -0.0004172 | 3338 |
| 6 | 7 | 0 | 544.4 | 0 | 0.02572 | 0.0004172 | -3338 |
| 6 | 8 | 0 | -558.8 | 0 | -0.02572 | -0.0002503 | 7751 |
| 7 | 8 | 0 | 255.1 | 0 | 0.01042 | 0.0002503 | -7751 |
| 7 | 9 | 0 | -269.5 | 0 | -0.01042 | 0 | 9849 |
| 8 | 9 | 0 | -34.18 | 0 | -0.004879 | 0 | -9849 |
| 8 | 10 | 0 | 19.78 | 0 | 0.004879 | 0 | 9633 |
| 9 | 10 | 0 | -323.5 | 0 | -0.02018 | 0 | -9633 |
| 9 | 11 | 0 | 309.1 | 0 | 0.02018 | 0.0002503 | 7103 |
| 10 | 11 | 0 | -602.7 | 0 | -0.03497 | -0.0002503 | -7103 |
| 10 | 12 | 0 | 586.5 | 0 | 0.03497 | 0.0004381 | 1752 |
| 11 | 12 | 0 | -586.3 | 0 | -0.03497 | -0.0004381 | -1751 |
| 11 | 4 | 0 | 580.9 | 0 | 0.03497 | 0.0005007 | 0 |
| 12 | 4 | 0 | -5.772 | 0 | 0 | 0 | -1.35 |
| 12 | 13 | 0 | 0.372 | 0 | 0 | 0 | -1.381 |

PROJECT: I-75 Costing Plans

AASHTO STANDARD Load: (L+In)3- Impact factor not included.
Coord. system: Global Units: kips kips-ft

| Memb | Node | Fx | Fy | Fz | Mx | My | Mz |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -0 | 125.2 | -0.007187 | 26.83 | 0.1725 | 0 |
| 1 | 2 | -0 | -125.2 | 0.007187 | -26.96 | -0.1725 | 0 |
| 2 | 3 | -0 | 106 | 0.007187 | 25.18 | 0.1725 | 0 |
| 2 | 4 | -0 | -106 | -0.007187 | -25.05 | -0.1725 | 0 |
| 3 | 5 | 0 | 0 | 0 | 0 | 0 | 0 |
| 3 | 2 | 0 | 0 | 0 | 0 | 0 | 0 |
| 4 | 2 | 0 | 125.2 | -0.007187 | 26.96 | 0.1725 | 0 |
| 4 | 6 | 0 | -125.2 | 0.007187 | -26.96 | -0.1509 | 375.7 |
| 5 | 6 | 0 | 125.2 | -0.007187 | 26.96 | 0.1509 | -375.7 |
| 5 | 7 | 0 | -125.2 | 0.007187 | -26.96 | -0.1437 | 500.9 |
| 6 | 7 | 0 | 125.2 | -0.007187 | 26.96 | 0.1437 | -500.9 |
| 6 | 8 | 0 | -125.2 | 0.007187 | -26.96 | -0.08624 | 1503 |
| 7 | 8 | 0 | 78.99 | -0.007187 | 16.56 | 0.08624 | -1503 |
| 7 | 9 | 0 | -78.99 | 0.007187 | -16.56 | -0.02875 | 2135 |
| 8 | 9 | 0 | -13.49 | -0.007187 | -4.246 | 0.02875 | -2135 |
| 8 | 10 | 0 | 13.49 | 0.007187 | 4.246 | 0.02875 | 2027 |
| 9 | 10 | 0 | -94.41 | -0.007187 | -22.45 | -0.02875 | -2027 |
| 9 | 11 | 0 | 94.41 | 0.007187 | 22.45 | 0.08624 | 1272 |
| 10 | 11 | 0 | -106 | -0.007187 | -25.05 | -0.08624 | -1272 |
| 10 | 12 | 0 | 106 | 0.007187 | 25.05 | 0.1509 | 317.9 |
| 11 | 12 | 0 | -106 | -0.007187 | -25.05 | -0.1509 | -317.9 |
| 11 | 4 | 0 | 106 | 0.007187 | 25.05 | 0.1725 | 0 |
| 12 | 4 | 0 | 0 | 0 | 0 | 0 | 0 |
| 12 | 13 | 0 | 0 | 0 | 0 | 0 | 0 |

$$I = \frac{50}{48+125} = 0.289$$

$$M_{LL+I} = 1.289\,(2135^{k'}) = 2752^{k'}$$

PROJECT: I-75 Costing Plans

AASHTO STANDARD Comb: 3 (SER GP I)

Coord. system: Global Units: kips kips-ft

| Memb | Node | Fx | Fy | Fz | Mx | My | Mz |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -0 | 1203 | -0.008563 | 31.93 | 0.2055 | 0 |
| 1 | 2 | -0 | -1131 | 0.008563 | -32.09 | -0.2055 | 0 |
| 2 | 3 | -0 | 930.6 | 0.008563 | 29.97 | 0.2055 | 0 |
| 2 | 4 | -0 | -858.8 | -0.008563 | -29.81 | -0.2055 | 0 |
| 3 | 5 | 0 | 0.9175 | 0 | 0 | 0 | 0.2881 |
| 3 | 2 | 0 | 11.16 | 0 | 0 | 0 | -4.384 |
| 4 | 2 | 0 | 1119 | -0.008563 | 32.09 | 0.2055 | 0 |
| 4 | 6 | 0 | -1107 | 0.008563 | -32.09 | -0.1798 | 3340 |
| 5 | 6 | 0 | 1107 | -0.008563 | 32.09 | 0.1798 | -3340 |
| 5 | 7 | 0 | -1103 | 0.008563 | -32.09 | -0.1713 | 4445 |
| 6 | 7 | 0 | 809.7 | -0.008563 | 32.07 | 0.1713 | -4445 |
| 6 | 8 | 0 | -777.5 | 0.008563 | -32.07 | -0.1028 | 1.079e+004 |
| 7 | 8 | 0 | 418.9 | -0.008563 | 19.69 | 0.1028 | -1.079e+004 |
| 7 | 9 | 0 | -386.7 | 0.008563 | -19.69 | -0.03425 | 1.402e+004 |
| 8 | 9 | 0 | -26.92 | -0.008563 | -5.052 | 0.03425 | -1.402e+004 |
| 8 | 10 | 0 | 59.12 | 0.008563 | 5.052 | 0.03425 | 1.367e+004 |
| 9 | 10 | 0 | -459 | -0.008563 | -26.71 | -0.03425 | -1.367e+004 |
| 9 | 11 | 0 | 491.2 | 0.008563 | 26.71 | 0.1028 | 9871 |
| 10 | 11 | 0 | -798.5 | -0.008563 | -29.81 | -0.1028 | -9871 |
| 10 | 12 | 0 | 834.8 | 0.008563 | 29.81 | 0.1798 | 2522 |
| 11 | 12 | 0 | -834.8 | -0.008563 | -29.81 | -0.1798 | -2523 |
| 11 | 4 | 0 | 846.9 | 0.008563 | 29.81 | 0.2055 | 0 |
| 12 | 4 | 0 | 13.21 | 0 | 0 | 0 | 3.019 |
| 12 | 13 | 0 | -1.131 | 0 | 0 | 0 | 5.173 |

PROJECT: I-75 Costing Plans

AASHTO STANDARD Envelope Strength

Coord. system: Global Units: kips kips-ft

| Memb | Node | Fx | Fy | Fz | Mx | My | Mz (Max/Min) |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -49.94 | 1406 | -2.27e-005 | 0.05528 | 0.0005448 | 858.5 |
| | | 49.94 | 1334 | -3.154e-005 | 0.04908 | 0.000757 | -858.5 |
| 1 | 2 | -45.79 | -1785 | 65.32 | -329.1 | -313.6 | 0 |
| | | -45.79 | -1785 | 65.32 | -329.1 | -313.6 | 0 |
| 2 | 3 | -49.57 | 1153 | 2.27e-005 | 0.05007 | 0.0005448 | 851.8 |
| | | 49.57 | 939.1 | 3.154e-005 | 0.04184 | 0.000757 | -851.8 |
| 2 | 4 | -45.42 | -1493 | -49.82 | -377.9 | -319.8 | 0 |
| | | -45.42 | -1493 | -49.82 | -377.9 | -319.8 | 0 |
| 3 | 5 | 0 | 1.193 | 0 | 0 | 0 | 0.3745 |
| | | 0 | 1.147 | 0 | 0 | 0 | 0.3601 |
| 3 | 2 | 0 | 13.95 | 1.141 | 0 | 1.711 | -5.48 |
| | | 0 | 14.5 | 1.186 | 0 | 1.779 | -5.699 |
| 4 | 2 | 45.79 | 1769 | -64.18 | 329.1 | 315.3 | 0 |
| | | 45.79 | 1769 | -64.18 | 329.1 | 315.3 | 0 |
| 4 | 6 | 0 | -1754 | 0.08554 | -99.82 | -1.796 | 5284 |
| | | -44.03 | -1163 | 36.09 | -134.6 | -67.49 | 3513 |
| 5 | 6 | 44.03 | 1164 | -36.09 | 134.6 | 67.49 | -3512 |
| | | 0 | 1754 | -0.08554 | 99.82 | 1.796 | -5284 |
| 5 | 7 | 0 | -1748 | 0.08554 | -99.82 | -1.711 | 7035 |
| | | -44.03 | -1158 | 36.09 | -134.6 | -31.4 | 4673 |
| 6 | 7 | 26.88 | 763.7 | -23.69 | 74.36 | 31.4 | -4590 |
| | | 0 | 1305 | -0.08554 | 85.99 | 1.711 | -7035 |
| 6 | 8 | 0 | -1263 | 0.08554 | -85.99 | -1.026 | 1.731e+004 |
| | | -26.88 | -723.4 | 23.69 | -74.36 | -158.1 | 1.054e+004 |
| 7 | 8 | 9.726 | 371.3 | -11.28 | 14.14 | 158.1 | -1.046e+004 |
| | | 0 | 576.4 | -0.08554 | 20.22 | 1.026 | -1.731e+004 |
| 7 | 9 | 0 | -649.2 | 0.02446 | -56.31 | -0.09783 | 2.218e+004 |
| | | -9.726 | -331.1 | 11.28 | -14.14 | -248.4 | 1.326e+004 |
| 8 | 9 | -7.428 | -21.07 | -1.119 | -46.09 | 248.4 | -1.318e+004 |
| | | 0 | -60 | -0.02446 | -14.44 | 0.09783 | -2.218e+004 |
| 8 | 10 | 0 | 10.16 | -0.0244 | -14.43 | -0.09762 | 2.19e+004 |
| | | 9.369 | 61.32 | 1.119 | 46.09 | 239.4 | 1.285e+004 |
| 9 | 10 | -26.52 | -413.4 | -13.52 | -106.3 | -239.4 | -1.277e+004 |
| | | 0 | -719.3 | 0.0244 | -56.32 | 0.09762 | -2.19e+004 |
| 9 | 11 | 0 | 480.4 | 0.1099 | 21.33 | 1.319 | 1.735e+004 |
| | | 26.52 | 453.7 | 13.52 | 106.3 | 131.2 | 9300 |
| 10 | 11 | -43.68 | -738 | -25.93 | -166.5 | -131.2 | -9217 |
| | | 0 | -1415 | -0.1099 | -103.1 | -1.319 | -1.735e+004 |
| 10 | 12 | 0 | 1462 | 0.1099 | 103.1 | 2.308 | 4408 |
| | | 43.68 | 783.3 | 25.93 | 166.5 | 102.1 | 2371 |
| 11 | 12 | -43.68 | -783.3 | -25.93 | -166.5 | -102.1 | -2373 |
| | | 0 | -1462 | -0.1099 | -103.1 | -2.308 | -4409 |
| 11 | 4 | 45.42 | 1478 | 48.68 | 377.9 | 321.5 | 0 |
| | | 45.42 | 1478 | 48.68 | 377.9 | 321.5 | 0 |
| 12 | 4 | 0 | 17.17 | 1.186 | 0 | -1.779 | 3.924 |
| | | 0 | 16.51 | 1.141 | 0 | -1.711 | 3.773 |
| 12 | 13 | 0 | -1.47 | 0 | 0 | 0 | 6.725 |
| | | 0 | -1.413 | 0 | 0 | 0 | 6.467 |

COLUMN BOTTOM LOADS WITHOUT IMPACT

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -49.94 | 1406 | -2.27e-005 | 0.05528 | 0.0005448 | 858.5 |
| | | 49.94 | 1334 | -3.154e-005 | 0.04908 | 0.000757 | -858.5 |
| 2 | 3 | -49.57 | 1153 | 2.27e-005 | 0.05007 | 0.0005448 | 851.8 |
| | | 49.57 | 939.1 | 3.154e-005 | 0.04184 | 0.000757 | -851.8 |

PROJECT: I-75 Costing Plans

AASHTO STANDARD Envelope Service

Coord. system: Global Units: kips kips-ft

| Memb | Node | Fx | Fy(Max/Min) | Fz | Mx | My | Mz |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -0 | 1524 | -0.1452 | 78.53 | 3.486 | 0 |
| | | -38.42 | 953.7 | -28.87 | 631.6 | 140.6 | 660.4 |
| 1 | 2 | 35.23 | -881.9 | 28.87 | -107.7 | -140.6 | 0 |
| | | -0 | -1452 | 0.1452 | -81.16 | -3.486 | 0 |
| 2 | 3 | -0 | 1068 | 0.008563 | 60.88 | 0.2055 | 0 |
| | | 36.39 | 792.3 | -25.91 | -616 | -171.8 | -625 |
| 2 | 4 | -33.31 | -720.5 | 25.09 | 155.3 | 171.8 | 0 |
| | | -0 | -996.2 | -0.008563 | -60.73 | -0.2055 | 0 |
| 3 | 5 | 0 | 0.9175 | 0 | 0 | 0 | 0.2881 |
| | | 0 | 0.9175 | 0 | 0 | 0 | 0.2881 |
| 3 | 2 | 0 | 11.16 | -0.9126 | 0 | -1.369 | -4.384 |
| | | 0 | 11.16 | -0.9126 | 0 | -1.369 | -4.384 |
| 4 | 2 | 0 | 1440 | -0.1452 | 81.16 | 3.486 | 0 |
| | | -35.23 | 870 | -28.87 | 107.7 | 140.6 | 0 |
| 4 | 6 | 35.23 | -857.9 | 28.87 | -107.7 | -53.99 | 2592 |
| | | 0 | -1428 | 0.1452 | -81.16 | -3.05 | 4302 |
| 5 | 6 | 0 | 1428 | -0.1452 | 81.16 | 3.05 | -4301 |
| | | -35.23 | 858 | -28.87 | 107.7 | 53.99 | -2591 |
| 5 | 7 | 35.23 | -854 | 28.87 | -107.7 | -25.12 | 3447 |
| | | 0 | -1424 | 0.1452 | -81.16 | -2.905 | 5727 |
| 6 | 7 | 0 | 977.5 | -0.04273 | 64.05 | 0.8547 | -5178 |
| | | 0 | 580.7 | 0.0939 | 33.89 | -1.878 | -4903 |
| 6 | 8 | 0 | -548.5 | -0.0939 | -33.89 | 1.127 | 9420 |
| | | 0 | -945.3 | 0.04273 | -64.05 | -0.5128 | 1.287e+004 |
| 7 | 8 | 0 | 527.1 | -0.008563 | 44.04 | 0.1028 | -1.244e+004 |
| | | 0 | 287.8 | -0.1452 | 32.87 | 1.743 | -1.155e+004 |
| 7 | 9 | 0 | -255.6 | 0.1452 | -32.87 | -0.581 | 1.372e+004 |
| | | 0 | -494.9 | 0.008563 | -44.04 | -0.03425 | 1.653e+004 |
| 8 | 9 | 0 | 60.68 | -0.0171 | -18.98 | 0.06839 | -1.377e+004 |
| | | 0 | -144.3 | 0.0939 | 45.84 | -0.3756 | -1.552e+004 |
| 8 | 10 | 0 | 176.5 | -0.0939 | -45.84 | -0.3756 | 1.424e+004 |
| | | 0 | -28.48 | 0.0171 | 18.98 | 0.06839 | 1.413e+004 |
| 9 | 10 | 21.22 | -315.2 | -10.82 | -85.05 | -191.5 | -1.083e+004 |
| | | 0 | -571.2 | 0.01706 | 53.33 | 0.06823 | -1.608e+004 |
| 9 | 11 | 0 | 603.4 | -0.01706 | -53.33 | -0.2047 | 1.138e+004 |
| | | -21.22 | 347.4 | 10.82 | 85.05 | 105 | 8181 |
| 10 | 11 | -33.31 | -660.3 | 24.58 | 155.3 | -125.5 | -8213 |
| | | 0 | -936 | -0.008563 | -60.73 | -0.1028 | -1.152e+004 |
| 10 | 12 | 0 | 972.2 | 0.008563 | 60.73 | 0.1798 | 2934 |
| | | 33.31 | 696.5 | -24.89 | -155.3 | -97.11 | 2107 |
| 11 | 12 | -33.31 | -696.6 | 24.89 | 155.3 | 97.11 | -2108 |
| | | 0 | -972.2 | -0.008563 | -60.73 | -0.1798 | -2935 |
| 11 | 4 | 0 | 984.3 | 0.008563 | 60.73 | 0.2055 | 0 |
| | | 33.31 | 708.6 | -24.99 | -155.3 | -171.9 | 0 |
| 12 | 4 | 0 | 13.21 | -0.9126 | 0 | 1.369 | 3.019 |
| | | 0 | 13.21 | -0.9126 | 0 | 1.369 | 3.019 |
| 12 | 13 | 0 | -1.131 | 0 | 0 | 0 | 5.173 |
| | | 0 | -1.131 | 0 | 0 | 0 | 5.173 |

COLUMN BOTTOM LOADS WITHOUT IMPACT

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| 1 | 1 | -0 | 1449 | -0.1222 | 66.07 | 2.933 | 0 |
| | | -38.42 | 953.7 | -28.87 | 631.6 | 140.6 | 660.4 |
| 2 | 3 | -0 | 1026 | 0.007207 | 51.22 | 0.173 | 0 |
| | | 36.39 | 792.3 | -25.91 | -616 | -171.8 | -625 |

## CAP DESIGN

**CAP DESIGN SHOWN FOR SHEAR REINFORCEMENT ONLY**

### CAP DESIGN

Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design
Units: US
Pier View: Upstation.

### DESIGN PARAMETERS

f'c = 6000.0 psi
Fy flex = 60000.0 psi
Fy shear = 60000.0 psi
phi flex = 0.90
phi shear = 0.85
Ec = 4696.0 ksi
Es = 29000.0 ksi
crack control factor z = 170.00 kips / in
Concrete Type : Normal Weight.
Design of cap at face of column.

### CAP GEOMETRY

Straight Cap : Length(X) = 54.00 ft Depth(Z) = 48.00 in

### MAIN REINFORCEMENT

| | Bar size | Quantity | Bar dist. in | As total in^2 | From ft | To ft | Hook |
|---|---|---|---|---|---|---|---|
| TOP | | | | | | | |
| | # 11 | 4 | 3.45 | 6.240 | 0.00 | 54.00 | Both |
| BOTTOM | | | | | | | |
| | # 11 | 11 | 3.45 | 17.160 | 0.00 | 54.00 | Both |
| | # 11 | 11 | 6.98 | 17.160 | 0.00 | 54.00 | Both |
| | # 11 | 11 | 10.51 | 17.160 | 3.00 | 51.00 | Both |

### STIRRUPS

| From ft | To ft | Stirrup Size | n legs | Spacing in | Aprv/s in^2 / ft |
|---|---|---|---|---|---|
| 0.00 | 4.00 | # 6 | 4 | 24.00 | 0.88 |
| 4.00 | 7.00 | # 6 | 4 | 6.00 | 3.52 |
| 7.00 | 15.00 | # 6 | 4 | 12.00 | 1.76 |
| 15.00 | 39.00 | # 6 | 4 | 24.00 | 0.88 |
| 39.00 | 50.00 | # 6 | 4 | 12.00 | 1.76 |
| 50.00 | 54.00 | # 6 | 4 | 24.00 | 0.88 |

Clear Cover on Sides = 2.00 in

## FLEXURE DESIGN

### Span 1: From 0.00 ft To 3.00 ft

| Loc ft | AbsLoc ft | H in | Mmax Mmin kips-ft | pMn kips-ft | Comb | Asb-req in^2 | Asb-prv in^2 | Asb-eff in^2 | Ast-req in^2 | Ast-prv in^2 | Ast-eff in^2 |
|---|---|---|---|---|---|---|---|---|---|---|---|

### Span 2: From 3.00 ft To 51.00 ft

| Loc ft | AbsLoc ft | H in | Mmax Mmin kips-ft | pMn kips-ft | Comb | Asb-req in^2 | Asb-prv in^2 | Asb-eff in^2 | Ast-req in^2 | Ast-prv in^2 | Ast-eff in^2 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 3.0 | 6.0 | 117 | 5284.2 | 24127.3 | 1205 | 14.50 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| 4.0 | 7.0 | 117 | 7034.8 | 24127.3 | 1205 | 19.41 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| 12.0 | 15.0 | 117 | 17310.1 | 24127.3 | 1205 | 36.62 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| 20.0 | 23.0 | 117 | 22179.7 | 24127.3 | 1203 | 47.53 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| 28.0 | 31.0 | 117 | 21899.1 | 24127.3 | 1214 | 46.90 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| 36.0 | 39.0 | 117 | 17352.4 | 24127.3 | 1201 | 36.72 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| 45.0 | 48.0 | 117 | 4409.1 | 24127.3 | 1201 | 12.06 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |
| | | | 0.0 | -3377.7 | 0 | 0.00 | 51.48 | 51.48 | 0.00 | 6.24 | 6.24 |

### Span 3: From 51.00 ft To 54.00 ft

| Loc ft | AbsLoc ft | H in | Mmax Mmin kips-ft | pMn kips-ft | Comb | Asb-req in^2 | Asb-prv in^2 | Asb-eff in^2 | Ast-req in^2 | Ast-prv in^2 | Ast-eff in^2 |
|---|---|---|---|---|---|---|---|---|---|---|---|

### Flexure Design : Notes

* The provided reinforcement is not adequate, either less than required or larger than maximum allowed.

## SHEAR AND TORSION DESIGN

**Span 1: From 0.00 ft To 3.00 ft**

| Loc ft | AbsLoc ft | Pos | Vu kips | Comb | Tu kips-ft | Comb | phi*Vc kips | T-lim kips-ft | Avs/s in^2/ft | 2Ats/s in^2/ft | Av/s in^2/ft | Aprv/s in^2/ft | Alt in^2 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|

**Span 2: From 3.00 ft To 51.00 ft**

| Loc ft | AbsLoc ft | Pos | Vu kips | Comb | Tu kips-ft | Comb | phi*Vc kips | T-lim kips-ft | Avs/s in^2/ft | 2Ats/s in^2/ft | Av/s in^2/ft | Aprv/s in^2/ft | Alt in^2 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3.00 | 6.00 | R | 1753.7 | 1205 | 329.1 | 2081 | 692.2 | 521.5 | 2.28 | 0.00 | 2.28 | 3.52 | 0.00 |
| 4.00 | 7.00 | L | 1748.5 | 1205 | 329.1 | 2081 | 692.2 | 521.5 | 2.27 | 0.00 | 2.27 | 3.52 | 0.00 |
| | | R | 1311.1 | 1216 | 236.8 | 2093 | 692.2 | 521.5 | 1.33 | 0.00 | 1.33 | 1.76 | 0.00 |
| 12.00 | 15.00 | L | 1269.2 | 1216 | 236.8 | 2093 | 692.2 | 521.5 | 1.24 | 0.00 | 1.24 | 1.76 | 0.00 |
| | | R | 744.3 | 1204 | 141.9 | 2083 | 692.2 | 521.5 | 0.48 | 0.00 | 0.48 | 0.88 | 0.00 |
| 20.00 | 23.00 | L | 702.4 | 1204 | 141.9 | 2083 | 692.2 | 521.5 | 0.48 | 0.00 | 0.48 | 0.88 | 0.00 |
| | | R | 172.2 | 1205 | 160.6 | 1718 | 692.2 | 521.5 | 0.00 | 0.00 | 0.00 | 0.88 | 0.00 |
| 28.00 | 31.00 | L | 214.0 | 1205 | 180.6 | 2078 | 692.2 | 521.5 | 0.00 | 0.00 | 0.00 | 0.88 | 0.00 |
| | | R | 772.6 | 1217 | 278.5 | 2085 | 692.2 | 521.5 | 0.48 | 0.00 | 0.48 | 0.88 | 0.00 |
| 36.00 | 39.00 | L | 814.4 | 1217 | 278.5 | 2085 | 692.2 | 521.5 | 0.48 | 0.00 | 0.48 | 0.88 | 0.00 |
| | | R | 1414.7 | 1201 | 377.9 | 2077 | 692.2 | 521.5 | 1.55 | 0.00 | 1.55 | 1.76 | 0.00 |
| 45.00 | 48.00 | L | 1461.8 | 1201 | 377.9 | 2077 | 692.2 | 521.5 | 1.65 | 0.00 | 1.65 | 1.76 | 0.00 |

**Span 3: From 51.00 ft To 54.00 ft**

| Loc ft | AbsLoc ft | Pos | Vu kips | Comb | Tu kips-ft | Comb | phi*Vc kips | T-lim kips-ft | Avs/s in^2/ft | 2Ats/s in^2/ft | Av/s in^2/ft | Aprv/s in^2/ft | Alt in^2 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|

**Shear and Torsion Design : Notes**

- Pos is the design position. L suggests the calculation is done at immediate left of "Loc" and R suggests at immediate right of it.
- T-lim is the limiting value of torsion for the concrete section. If actual torsion is higher than this value, torsional steel has to be provided.
- Avs/s is the required area of steel per unit length for shear force.
- 2Ats/s is the required area of steel per unit length for two legs of torsional reinforcement.
- Av/s is the total required area of steel per unit length due to shear plus torsion.
- Aprvs/s is the total provided area of steel per unit length due to shear (stirrups).
- Alt is the total longitudinal steel required due to torsion in addition to the REQUIRED flexural steel.

**CRACKING/FATIGUE CHECK**

**Span 1: From 0.00 ft To 3.00 ft**

| Loc ft | AbsLoc ft | H in | Cracking fs-t fs-b ksi | Cracking ratio fs-t ratio fs-b | Cracking Comb | Fatigue fs-t fs-b ksi | Fatigue ratio fs-t ratio fs-b |
|---|---|---|---|---|---|---|---|

**Span 2: From 3.00 ft To 51.00 ft**

| Loc ft | AbsLoc ft | H in | Cracking fs-t fs-b ksi | Cracking ratio fs-t ratio fs-b | Cracking Comb | Fatigue fs-t fs-b ksi | Fatigue ratio fs-t ratio fs-b |
|---|---|---|---|---|---|---|---|
| 3.00 | 6.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 9.3 | 0.26 | 23 | 1.2 | 0.06 |
| 4.00 | 7.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 12.3 | 0.34 | 23 | 1.7 | 0.08 |
| 12.00 | 15.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 30.2 | 0.84 | 23 | 4.6 | 0.27 |
| 20.00 | 23.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 38.7 | **1.07 *** | 21 | 5.9 | 0.39 |
| 28.00 | 31.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 38.2 | **1.06 *** | 32 | 5.9 | 0.39 |
| 36.00 | 39.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 30.1 | 0.84 | 19 | 5.3 | 0.30 |
| 45.00 | 48.0 | 116.5 | 0.0 | 0.00 | 0 | 0.0 | 0.00 |
| | | | 7.7 | 0.21 | 19 | 1.3 | 0.06 |

**Span 3: From 51.00 ft To 54.00 ft**

| Loc ft | AbsLoc ft | H in | Cracking fs-t fs-b ksi | Cracking ratio fs-t ratio fs-b | Cracking Comb | Fatigue fs-t fs-b ksi | Fatigue ratio fs-t ratio fs-b |
|---|---|---|---|---|---|---|---|

**Cracking and fatigue Check : Notes**

* Cracking / fatigue checking failed.

## COLUMN DESIGN

| COLUMN DESIGN - Column: 1 |
|---|
| Code: AASHTO STANDARD (17th Edition 2002) - Factored Load Design |
| Units: US |
| Pier View: Upstation. |
| Design/Analysis Method: Moment Magnification - Unbraced Column. |

**Column Type: Rectangular 72.00 x 72.00 in**

**Column Section Properties**

| Sec. | Area ft^2 | Ixx in^4 | Izz in^4 |
|---|---|---|---|
| 1 | 36.00 | 2239488.00 | 2239488.00 |

**DESIGN PARAMETERS**

| | |
|---|---|
| f'c = 3500.0 psi | fy = 60000.0 psi |
| phi flex = 0.90 | phi axial = 0.70 |
| Ec = 3586.6 ksi | Es = 29000 ksi |
| Concrete Type : Normal Weight. | |

**Reinforcement**

| |
|---|
| Rebar Pattern: Rectangular |
| Rebar Orientation: Face Parallel |

**Reinforcement Schedule**

| Layer | Dir | Size | No. bars | Bar Dist. in |
|---|---|---|---|---|
| 1 | X | 11 | 10 | 3.20 |
| 1 | Z | 11 | 9 | 3.20 |

**Reinforcement summary**

| |
|---|
| Main bars summary: |
| 34 # 11 bars |

Total number of bars in the column: 34

Ties size: # 3

**Moment Magnification calculation - Mx (global)**

| Loc ft | Comb | K | Cm | Beta | Delta B | Delta S | pPc kips |
|---|---|---|---|---|---|---|---|
| 75.70 | 45C | 3.4219 | 1.0000 | 0.7322 | --- | 1.0890 | 20097.60 |
| 95.15 | 45C | 3.4219 | 1.0000 | 0.7169 | --- | 1.0820 | 20276.64 |
| 75.70 | 894C | 3.4219 | 1.0000 | 0.1539 | --- | 1.0299 | 30170.10 |
| 95.15 | 894C | 3.4219 | 1.0000 | 1.0000 | --- | 1.0432 | 17406.75 |

**Moment Magnification calculation - Mz (global)**

| Loc ft | Comb | K | Cm | Beta | Delta B | Delta S | pPc kips |
|---|---|---|---|---|---|---|---|
| 75.70 | 45C | 1.6758 | 1.0000 | 0.7322 | --- | 1.0200 | 83798.78 |
| 95.15 | 45C | 1.6758 | 1.0000 | 0.7169 | --- | 1.0185 | 84545.34 |
| 75.70 | 894C | 1.6758 | 1.0000 | 0.5606 | --- | 1.0091 | 93015.59 |
| 95.15 | 894C | 1.6758 | 1.0000 | 1.0000 | --- | 1.0112 | 72579.07 |

**Design values used after Moment Magnification ( e-min effect included ).**

| Loc ft | Comb | Fx kips | Fy kips | Fz kips | Mx kips-ft | My kips-ft | Mz kips-ft |
|---|---|---|---|---|---|---|---|
| 75.70 | 45C | -0.0 | 1895.7 | -0.1 | 474.8 | 2.1 | 444.7 |
| 95.15 | 45C | -0.0 | 1793.2 | 0.1 | -446.3 | 2.1 | 420.1 |
| 75.70 | 894C | -22.5 | 995.1 | 68.9 | 1540.6 | -303.4 | 414.5 |
| 95.15 | 894C | 20.3 | 921.2 | -65.2 | -221.0 | -303.4 | 214.3 |

**COLUMN DESIGN**

| Bot/Top Elev. ft | Comb | Pu kips | Mux kips-ft | Muz kips-ft | pMn kips-ft | Incl deg | pPn/Pu | pMn/Mu |
|---|---|---|---|---|---|---|---|---|
| 75.70 | 894C | 995.1 | 1540.6 | 414.5 | 8713.9 | 15.06 | 1.00 | 5.46190 |
| 95.15 | 45C | 1793.2 | 446.3 | 420.1 | 9071.6 | 43.27 | 1.00 | 14.80138 |

**COLUMN DESIGN**

| Bot/Top Elev. ft | Comb | As_min in^2 | As_max in^2 | As_req in^2 |
|---|---|---|---|---|
| 75.70 | 894C | 51.84 | 414.72 | 53.04 |
| 95.15 | 45C | 51.84 | 414.72 | 53.04 |

**K values for all columns used in unbraced moment magnification**

| Column | Kx | Kz |
|---|---|---|
| 1 | 3.42 | 1.68 |

| Column | Kx | Kz |
|---|---|---|
| 2 | 3.42 | 1.68 |

**Column Design : Notes**

Min reinforcement = 1.0000 % of Ag = 51.84 in^2.

For layer 1, direction X: Minimum bar distance is: 1.41 in. Effective bar distance is: 5.88 in.

For layer 1, direction Z: Minimum bar distance is: 1.41 in. Effective bar distance is: 6.79 in.

--- Values do not exist at that location as computation is done at the top and bottom of clear length of column.

**COLUMN DESIGN - Column: 2**

Code: AASHTO STANDARD (17th Edition 2002) - Factored Load Design

Units: US

Pier View: Upstation.

Design/Analysis Method: Moment Magnification - Unbraced Column.

**Column Type: Rectangular 72.00 x 72.00 in**

**Column Section Properties**

| Sec. | Area ft^2 | Ixx in^4 | Izz in^4 |
|---|---|---|---|
| 1 | 36.00 | 2239488.00 | 2239488.00 |

**DESIGN PARAMETERS**

| | |
|---|---|
| f'c = 3500.0 psi | fy = 60000.0 psi |
| phi flex = 0.90 | phi axial = 0.70 |
| Ec = 3586.6 ksi | Es = 29000 ksi |
| Concrete Type : Normal Weight. | |

## Reinforcement

Rebar Pattern: Rectangular

Rebar Orientation: Face Parallel

## Reinforcement Schedule

| Layer | Dir | Size | No. bars | Bar Dist. in |
|---|---|---|---|---|
| 1 | X | 11 | 10 | 3.20 |
| 1 | Z | 11 | 9 | 3.20 |

## Reinforcement summary

Main bars summary:

34 # 11 bars

Total number of bars in the column: 34

Ties size: # 3

## Moment Magnification calculation - Mx (global)

| Loc ft | Comb | K | Cm | Beta | Delta B | Delta S | pPc kips |
|---|---|---|---|---|---|---|---|
| 75.70 | 37C | 3.4219 | 1.0000 | 0.6614 | --- | 1.0884 | 20954.00 |
| 95.15 | 37C | 3.4219 | 1.0000 | 0.6382 | --- | 1.0814 | 21251.01 |
| 75.70 | 922C | 3.4219 | 1.0000 | 0.1547 | --- | 1.0303 | 30149.40 |
| 95.15 | 922C | 3.4219 | 1.0000 | 0.6474 | --- | 1.0436 | 21132.70 |

## Moment Magnification calculation - Mz (global)

| Loc ft | Comb | K | Cm | Beta | Delta B | Delta S | pPc kips |
|---|---|---|---|---|---|---|---|
| 75.70 | 37C | 1.6758 | 1.0000 | 0.6614 | --- | 1.0199 | 87369.65 |
| 95.15 | 37C | 1.6758 | 1.0000 | 0.6382 | --- | 1.0184 | 88608.04 |
| 75.70 | 922C | 1.6758 | 1.0000 | 0.2494 | --- | 1.0079 | 116186.35 |
| 95.15 | 922C | 1.6758 | 1.0000 | 1.0000 | --- | 1.0112 | 72579.07 |

## Design values used after Moment Magnification ( e-min effect included ).

| Loc ft | Comb | Fx kips | Fy kips | Fz kips | Mx kips-ft | My kips-ft | Mz kips-ft |
|---|---|---|---|---|---|---|---|
| 75.70 | 37C | -0.0 | 1596.0 | -0.1 | 399.5 | -2.8 | 374.4 |
| 95.15 | 37C | -0.0 | 1493.6 | 0.1 | -371.5 | -2.8 | 349.8 |
| 75.70 | 922C | 38.0 | 749.8 | -47.6 | -1166.1 | -285.4 | -707.7 |
| 95.15 | 922C | -35.0 | 675.9 | 44.5 | 254.9 | -285.4 | 157.2 |

## COLUMN DESIGN

| Bot/Top Elev. ft | Comb | Pu kips | Mux kips-ft | Muz kips-ft | pMn kips-ft | Incl deg | pPn/Pu | pMn/Mu |
|---|---|---|---|---|---|---|---|---|
| 75.70 | 922C | 749.8 | 1166.1 | 707.7 | 8554.7 | 31.25 | 1.00 | 6.27139 |
| 95.15 | 37C | 1493.6 | 371.5 | 349.8 | 8769.7 | 43.28 | 1.00 | 17.18579 |

## COLUMN DESIGN

| Bot/Top Elev. ft | Comb | As_min in^2 | As_max in^2 | As_req in^2 |
|---|---|---|---|---|
| 75.70 | 922C | 51.84 | 414.72 | 53.04 |
| 95.15 | 37C | 51.84 | 414.72 | 53.04 |

## K values for all columns used in unbraced moment magnification

| Column | Kx | Kz |
|---|---|---|
| 1 | 3.42 | 1.68 |
| 2 | 3.42 | 1.68 |

## Column Design : Notes

Min reinforcement = 1.0000 % of Ag = 51.84 in^2.

For layer 1, direction X: Minimum bar distance is: 1.41 in. Effective bar distance is: 5.88 in.

For layer 1, direction Z: Minimum bar distance is: 1.41 in. Effective bar distance is: 6.79 in.

--- Values do not exist at that location as computation is done at the top and bottom of clear length of column.

# ISOLATED FOOTING DESIGN

| ISOLATED FOOTING DESIGN |
|---|
| Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design |
| Units: US |
| Pier View: Upstation. |

| GEOMETRY |
|---|
| Name : FTG01 |
| Shape : Rectangular, Type : Pile/Shaft Cap |
| Bf(X) = 12.00 ft, Hf(Z) = 12.00 ft, Thickness(Y) = 42.00 in |
| Ag = 144.00 ft^2, Ix = 105.00 ft^2, Iz = 92.60 ft^2 |
| Footing eccentric: Start at X = -6.00 ft from centerline of column. |
| Footing eccentric: Start at Z = -6.00 ft from centerline of column. |
| Columns located on the footing: |
| Column No. 1 at x = 0.00 ft, Rectangular 72.00 in x 72.00 in |
| Surcharge Variation on Z direction |
| Surcharge P1 = 0.00 ksf Surcharge P2 = 0.00 ksf |
| Piles: Chamfer Size: W = 13.83 in, D = 14.69 in, T1 = 0.62 in, T2 = 0.00 in |
| Service Capacity: 234.00 kips Factored Capacity: 328.00 kips |
| Piles Section Properties: Area = 0.18 ft^2 Ix = 325.51 in^4 Iz = 892.22 in^4 |

| DESIGN PARAMETERS | |
|---|---|
| f'c = 3500.00 psi | fy = 60000.00 psi |
| phi flex = 0.90 | phi shear = 0.85 |
| Ec = 3586.6 ksi | Es = 29000.0 ksi |
| Crack control factor z = 130.00 kips/in | |
| Concrete Type : Normal Weight. | |

**Pile Reactions, Service (Without the reduction of overstress allowance)**

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -4.50 | -2.00 | 18.00 | -48.0 | 1025 | 1.400 | -1225.27 | -845.13 | 322.92 | 165.20 |
| | | | | | 458 | 1.400 | -1095.43 | 665.88 | -708.85 | 47.94 |
| 2 | 0.00 | -2.00 | 72.00 | -48.0 | 899 | 1.400 | -1213.88 | -1138.63 | -145.01 | 173.19 |
| | | | | | 462 | 1.400 | -1072.48 | 1196.65 | 328.56 | 69.01 |
| 3 | 4.00 | -2.00 | 120.00 | -48.0 | 863 | 1.400 | -1209.40 | -1040.39 | -311.90 | 193.94 |
| | | | | | 461 | 1.400 | -1076.94 | 1087.60 | 564.74 | 58.66 |
| 4 | -2.50 | -4.50 | 42.00 | -18.0 | 1025 | 1.400 | -1225.27 | -845.13 | 322.92 | 143.72 |
| | | | | | 449 | 1.400 | -1055.47 | 531.49 | -675.76 | 83.11 |
| 5 | 2.00 | -4.50 | 96.00 | -18.0 | 863 | 1.400 | -1209.40 | -1040.39 | -311.90 | 156.88 |

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 448 | 1.400 | -1040.04 | 665.89 | 708.85 | 90.51 |
| 6 | -2.50 | -7.50 | 42.00 | 18.0 | 774 | 1.400 | -1224.71 | 1040.29 | 311.90 | 146.18 |
| | | | | | 445 | 1.400 | -1056.30 | -956.94 | -666.34 | 77.35 |
| 7 | 2.00 | -7.50 | 96.00 | 18.0 | 576 | 1.400 | -1208.84 | 845.03 | -322.92 | 154.29 |
| | | | | | 443 | 1.400 | -1040.04 | -665.96 | 708.85 | 90.51 |
| 8 | -4.50 | -10.00 | 18.00 | 48.0 | 461 | 1.400 | -1076.94 | 1087.60 | 564.74 | 172.21 |
| | | | | | 446 | 1.400 | -1058.53 | -1087.68 | -564.74 | 35.22 |
| 9 | 0.00 | -10.00 | 72.00 | 48.0 | 810 | 1.400 | -1220.23 | 1138.54 | 145.01 | 173.20 |
| | | | | | 447 | 1.400 | -1063.00 | -1196.73 | -328.56 | 69.47 |
| 10 | 4.00 | -10.00 | 120.00 | 48.0 | 576 | 1.400 | -1208.84 | 845.03 | -322.92 | 186.93 |
| | | | | | 443 | 1.400 | -1040.04 | -665.96 | 708.85 | 64.16 |

## Pile Reactions, Factored

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -4.50 | -2.00 | 18.0 | -48.0 | 2207 | — | -1531.58 | -1056.41 | 403.65 | 206.49 |
| | | | | | 1640 | — | -1369.29 | 832.35 | -886.06 | 59.93 |
| 2 | 0.00 | -2.00 | 72.0 | -48.0 | 2081 | — | -1517.35 | -1423.29 | -181.26 | 216.49 |
| | | | | | 1644 | — | -1340.60 | 1495.81 | 410.70 | 86.26 |
| 3 | 4.00 | -2.00 | 120.0 | -48.0 | 2045 | — | -1511.75 | -1300.48 | -389.87 | 242.42 |
| | | | | | 1643 | — | -1346.18 | 1359.50 | 705.92 | 73.33 |
| 4 | -2.50 | -4.50 | 42.0 | -18.0 | 1205 | — | -1815.10 | -82.22 | -0.00 | 183.01 |
| | | | | | 1631 | — | -1319.33 | 664.36 | -844.71 | 103.89 |
| 5 | 2.00 | -4.50 | 96.0 | -18.0 | 1205 | — | -1815.10 | -82.22 | -0.00 | 201.60 |
| | | | | | 1630 | — | -1300.05 | 832.36 | 886.06 | 113.14 |
| 6 | -2.50 | -7.50 | 42.0 | 18.0 | 1206 | — | -1815.10 | 82.08 | -0.00 | 183.01 |
| | | | | | 1627 | — | -1320.38 | -1196.17 | -832.92 | 96.68 |
| 7 | 2.00 | -7.50 | 96.0 | 18.0 | 1206 | — | -1815.10 | 82.08 | -0.00 | 201.60 |
| | | | | | 1625 | — | -1300.05 | -832.45 | 886.06 | 113.14 |
| 8 | -4.50 | -10.00 | 18.0 | 48.0 | 1643 | — | -1346.18 | 1359.50 | 705.92 | 215.26 |
| | | | | | 1628 | — | -1323.17 | -1359.60 | -705.92 | 44.03 |
| 9 | 0.00 | -10.00 | 72.0 | 48.0 | 1992 | — | -1525.28 | 1423.18 | 181.26 | 216.50 |
| | | | | | 1629 | — | -1328.75 | -1495.91 | -410.70 | 86.84 |
| 10 | 4.00 | -10.00 | 120.0 | 48.0 | 1758 | — | -1511.05 | 1056.29 | -403.65 | 233.67 |
| | | | | | 1625 | — | -1300.05 | -832.45 | 886.06 | 80.20 |

## Pile Reactions, Service (After the reduction of overstress allowance)

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -4.50 | -2.00 | 18.00 | -48.0 | 5 | 1.000 | -1217.05 | -28.77 | -0.00 | 118.36 |
| | | | | | 458 | 1.400 | -1095.43 | 665.88 | -708.85 | 34.25 |
| 2 | 0.00 | -2.00 | 72.00 | -48.0 | 5 | 1.000 | -1217.05 | -28.77 | -0.00 | 130.92 |
| | | | | | 462 | 1.400 | -1072.48 | 1196.65 | 328.56 | 49.29 |
| 3 | 4.00 | -2.00 | 120.00 | -48.0 | 5 | 1.000 | -1217.05 | -28.77 | -0.00 | 142.09 |
| | | | | | 461 | 1.400 | -1076.94 | 1087.60 | 564.74 | 41.90 |
| 4 | -2.50 | -4.50 | 42.00 | -18.0 | 5 | 1.000 | -1217.05 | -28.77 | -0.00 | 123.26 |
| | | | | | 449 | 1.400 | -1055.47 | 531.49 | -675.76 | 59.36 |
| 5 | 2.00 | -4.50 | 96.00 | -18.0 | 5 | 1.000 | -1217.05 | -28.77 | -0.00 | 135.82 |

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 448 | 1.400 | -1040.04 | 665.89 | 708.85 | 64.65 |
| 6 | -2.50 | -7.50 | 42.00 | 18.0 | 6 | 1.000 | -1217.05 | 28.68 | -0.00 | 123.25 |
| | | | | | 445 | 1.400 | -1056.30 | -956.94 | -666.34 | 55.25 |
| 7 | 2.00 | -7.50 | 96.00 | 18.0 | 6 | 1.000 | -1217.05 | 28.68 | -0.00 | 135.82 |
| | | | | | 443 | 1.400 | -1040.04 | -665.96 | 708.85 | 64.65 |
| 8 | -4.50 | -10.00 | 18.00 | 48.0 | 461 | 1.400 | -1076.94 | 1087.60 | 564.74 | 123.01 |
| | | | | | 446 | 1.400 | -1058.53 | -1087.68 | -564.74 | 25.16 |
| 9 | 0.00 | -10.00 | 72.00 | 48.0 | 6 | 1.000 | -1217.05 | 28.68 | -0.00 | 130.92 |
| | | | | | 447 | 1.400 | -1063.00 | -1196.73 | -328.56 | 49.62 |
| 10 | 4.00 | -10.00 | 120.00 | 48.0 | 6 | 1.000 | -1217.05 | 28.68 | -0.00 | 142.08 |
| | | | | | 443 | 1.400 | -1040.04 | -665.96 | 708.85 | 45.83 |

## Footing Design : Notes

Only max. force in piles is considered for design.
Pile coordinates X and Z are from the most left edge of the footing.
Plong= Lateral load in longitudinal direction at the top of pile, Kips.
Php= Available resisting horizontal component due to batter= batter * Vertical pile reaction, Kips.
Plong-Php= Remaining lateral force required to resist by pile.

## Max. Pile Reaction Used in Design: (without selfweight and surcharge)

| | |
|---|---|
| Factored pile reaction | 232.11 kips |
| Service pile reaction | 185.69 kips |
| Fatigue pile reaction | 17.38 kips |

## Reinforcement Schedule

| Dir | Quantity | Size | Bar dist. in | As total in^2 | Spacing in | Hook |
|---|---|---|---|---|---|---|
| X | 12 | #8 | 3.50 | 9.48 | 12.45 | None |
| Z | 12 | #8 | 4.50 | 9.48 | 12.45 | None |

## Flexure

| Dir ft | Loc | d in | Mmax kft | Comb | Asb_req in^2 | Asb_prv in^2 | Asb_eff in^2 | Ast_req in^2 | Ast_prv in^2 | Ast_eff in^2 |
|---|---|---|---|---|---|---|---|---|---|---|
| X | -3.00 | 38.50 | 696.3 | 2045 | 5.40 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |
| X | 3.00 | 38.50 | 464.2 | 2045 | 3.59 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |
| Z | -3.00 | 37.50 | 696.3 | 2045 | 5.55 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |
| Z | 3.00 | 37.50 | 696.3 | 2045 | 5.55 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |

## Cracking/Fatigue

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| Dir | Loc ft | d in | Cracking Mmax kft | Cracking Comb | Cracking fs ksi | Cracking ratio fs | Fatigue Mmax kft | Fatigue Comb | Fatigue fs ksi | Fatigue ratio fs |
|---|---|---|---|---|---|---|---|---|---|---|
| X | -3.00 | 38.50 | 557.1 | 863 | 19.30 | 0.79 | 52.1 | 5 | 1.81 | 0.09 |
| X | 3.00 | 38.50 | 371.4 | 863 | 12.87 | 0.53 | 34.8 | 5 | 1.20 | 0.06 |
| Z | -3.00 | 37.50 | 557.1 | 863 | 19.83 | 0.81 | 52.1 | 5 | 1.86 | 0.09 |
| Z | 3.00 | 37.50 | 557.1 | 863 | 19.83 | 0.81 | 52.1 | 5 | 1.86 | 0.09 |

## One Way Shear

| Col | Dir | Dist ft | Comb | d in | Vu kips | phi*Vc kips |
|---|---|---|---|---|---|---|
| 1 | X | -6.21 | Outside of Footing | --- | --- | --- |
| | X | 6.21 | Outside of Footing | --- | --- | --- |
| | Z | -6.13 | Outside of Footing | --- | --- | --- |
| | Z | 6.13 | Outside of Footing | --- | --- | --- |

## Two Way Shear

| # | Bo ft | Ao ft^2 | Comb | Avg. d in | Vu kips | phi*Vc kips |
|---|---|---|---|---|---|---|
| Columns | | | | | | |
| 1 | No Two Way Shear | | | | | |
| Piles - max | | | | | | |
| 4 | 17.28 | 18.66 | 2045 | 38.00 | 232.1 | 1584.7 |
| Piles - min | | | | | | |
| 1 | 7.82 | 15.22 | 2045 | 38.00 | 232.1 | 717.2 |

## Two Way Shear Note

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.

# ISOLATED FOOTING DESIGN

## ISOLATED FOOTING DESIGN

Code: AASHTO STANDARD (17th Edition 2002) - Ultimate Strength Design
Units: US
Pier View: Upstation.

## GEOMETRY

Name : FTG02
Shape : Rectangular, Type : Pile/Shaft Cap
Bf(X) = 12.00 ft, Hf(Z) = 12.00 ft, Thickness(Y) = 42.00 in
Ag = 144.00 ft^2, Ix = 105.00 ft^2, Iz = 92.60 ft^2
Footing eccentric: Start at X = -6.00 ft from centerline of column.
Footing eccentric: Start at Z = -6.00 ft from centerline of column.
Columns located on the footing:
Column No. 2 at x = 0.00 ft, Rectangular 72.00 in x 72.00 in
Surcharge Variation on Z direction
Surcharge P1 = 0.00 ksf Surcharge P2 = 0.00 ksf
Piles: Chamfer Size: W = 13.83 in, D = 14.69 in, T1 = 0.62 in, T2 = 0.00 in
Service Capacity: 234.00 kips Factored Capacity: 328.00 kips
Piles Section Properties: Area = 0.18 ft^2 Ix = 892.22 in^4 Iz = 325.51 in^4

## DESIGN PARAMETERS

| | |
|---|---|
| f'c = 3500.00 psi | fy = 60000.00 psi |
| phi flex = 0.90 | phi shear = 0.85 |
| Ec = 3586.6 ksi | Es = 29000.0 ksi |
| Crack control factor z = 130.00 kips/in | |
| Concrete Type : Normal Weight. | |

## Pile Reactions, Service (Without the reduction of overstress allowance)

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -4.50 | -2.00 | 18.00 | -48.0 | 1021 | 1.400 | -962.77 | -702.81 | 320.91 | 135.87 |
| | | | | | 449 | 1.400 | -824.30 | 426.18 | -671.78 | 34.20 |
| 2 | 0.00 | -2.00 | 72.00 | -48.0 | 895 | 1.400 | -974.16 | -952.31 | -144.23 | 142.02 |
| | | | | | 462 | 1.400 | -807.29 | 1004.03 | 327.02 | 49.72 |
| 3 | 4.00 | -2.00 | 120.00 | -48.0 | 859 | 1.400 | -978.64 | -866.39 | -310.03 | 162.05 |
| | | | | | 448 | 1.400 | -729.83 | 539.63 | 704.32 | 35.35 |
| 4 | -2.50 | -4.50 | 42.00 | -18.0 | 1021 | 1.400 | -962.77 | -702.81 | 320.91 | 116.69 |
| | | | | | 449 | 1.400 | -824.30 | 426.18 | -671.78 | 62.75 |
| 5 | 2.00 | -4.50 | 96.00 | -18.0 | 859 | 1.400 | -978.64 | -866.39 | -310.03 | 130.18 |

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 448 | 1.400 | -729.83 | 539.63 | 704.32 | 59.93 |
| 6 | -2.50 | -7.50 | 42.00 | 18.0 | 770 | 1.400 | -963.33 | 866.30 | 310.03 | 118.81 |
| | | | | | 445 | 1.400 | -823.47 | -788.76 | -662.62 | 57.71 |
| 7 | 2.00 | -7.50 | 96.00 | 18.0 | 572 | 1.400 | -979.20 | 702.73 | -320.91 | 128.15 |
| | | | | | 443 | 1.400 | -729.83 | -539.69 | 704.32 | 59.93 |
| 8 | -4.50 | -10.00 | 18.00 | 48.0 | 770 | 1.400 | -963.33 | 866.30 | 310.03 | 141.64 |
| | | | | | 445 | 1.400 | -823.47 | -788.76 | -662.62 | 20.74 |
| 9 | 0.00 | -10.00 | 72.00 | 48.0 | 806 | 1.400 | -967.81 | 952.22 | 144.23 | 140.76 |
| | | | | | 447 | 1.400 | -816.77 | -1004.10 | -327.02 | 52.08 |
| 10 | 4.00 | -10.00 | 120.00 | 48.0 | 458 | 1.400 | -894.24 | 539.62 | -704.32 | 158.28 |
| | | | | | 443 | 1.400 | -729.83 | -539.69 | 704.32 | 35.34 |

## Pile Reactions, Factored

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -4.50 | -2.00 | 18.0 | -48.0 | 2203 | — | -1203.46 | -878.51 | 401.14 | 169.84 |
| | | | | | 1631 | — | -1030.38 | 532.73 | -839.72 | 42.75 |
| 2 | 0.00 | -2.00 | 72.0 | -48.0 | 2077 | — | -1217.70 | -1190.39 | -180.29 | 177.52 |
| | | | | | 1644 | — | -1009.12 | 1255.04 | 408.78 | 62.14 |
| 3 | 4.00 | -2.00 | 120.0 | -48.0 | 2041 | — | -1223.30 | -1082.98 | -387.53 | 202.57 |
| | | | | | 1630 | — | -912.29 | 674.53 | 880.40 | 44.18 |
| 4 | -2.50 | -4.50 | 42.0 | -18.0 | 1201 | — | -1510.24 | -84.44 | -0.00 | 154.07 |
| | | | | | 1631 | — | -1030.38 | 532.73 | -839.72 | 78.43 |
| 5 | 2.00 | -4.50 | 96.0 | -18.0 | 1201 | — | -1510.24 | -84.44 | -0.00 | 169.70 |
| | | | | | 1630 | — | -912.29 | 674.53 | 880.40 | 74.91 |
| 6 | -2.50 | -7.50 | 42.0 | 18.0 | 1202 | — | -1510.24 | 84.31 | -0.00 | 154.07 |
| | | | | | 1627 | — | -1029.34 | -985.95 | -828.28 | 72.14 |
| 7 | 2.00 | -7.50 | 96.0 | 18.0 | 1202 | — | -1510.24 | 84.31 | -0.00 | 169.70 |
| | | | | | 1625 | — | -912.29 | -674.61 | 880.40 | 74.91 |
| 8 | -4.50 | -10.00 | 18.0 | 48.0 | 1952 | — | -1204.16 | 1082.88 | 387.53 | 177.05 |
| | | | | | 1627 | — | -1029.34 | -985.95 | -828.28 | 25.92 |
| 9 | 0.00 | -10.00 | 72.0 | 48.0 | 1988 | — | -1209.76 | 1190.28 | 180.29 | 175.94 |
| | | | | | 1629 | — | -1020.97 | -1255.13 | -408.78 | 65.10 |
| 10 | 4.00 | -10.00 | 120.0 | 48.0 | 1640 | — | -1117.80 | 674.53 | -880.40 | 197.85 |
| | | | | | 1625 | — | -912.29 | -674.61 | 880.40 | 44.18 |

## Pile Reactions, Service (After the reduction of overstress allowance)

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | -4.50 | -2.00 | 18.00 | -48.0 | 1021 | 1.400 | -962.77 | -702.81 | 320.91 | 97.05 |
| | | | | | 449 | 1.400 | -824.30 | 426.18 | -671.78 | 24.43 |
| 2 | 0.00 | -2.00 | 72.00 | -48.0 | 1 | 1.000 | -970.98 | -29.54 | -0.00 | 106.24 |
| | | | | | 462 | 1.400 | -807.29 | 1004.03 | 327.02 | 35.51 |
| 3 | 4.00 | -2.00 | 120.00 | -48.0 | 859 | 1.400 | -978.64 | -866.39 | -310.03 | 115.75 |
| | | | | | 448 | 1.400 | -729.83 | 539.63 | 704.32 | 25.25 |
| 4 | -2.50 | -4.50 | 42.00 | -18.0 | 1 | 1.000 | -970.98 | -29.54 | -0.00 | 99.88 |
| | | | | | 449 | 1.400 | -824.30 | 426.18 | -671.78 | 44.82 |
| 5 | 2.00 | -4.50 | 96.00 | -18.0 | 1 | 1.000 | -970.98 | -29.54 | -0.00 | 110.05 |

| Pile | Loc(X) ft | Loc(Z) ft | X in | Z in | comb | Ovs | P kips | Mxx kft | Mzz kft | Pile Reac. kips |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | 448 | 1.400 | -729.83 | 539.63 | 704.32 | 42.81 |
| 6 | -2.50 | -7.50 | 42.00 | 18.0 | 2 | 1.000 | -970.98 | 29.46 | -0.00 | 99.88 |
| | | | | | 445 | 1.400 | -823.47 | -788.76 | -662.62 | 41.22 |
| 7 | 2.00 | -7.50 | 96.00 | 18.0 | 2 | 1.000 | -970.98 | 29.46 | -0.00 | 110.05 |
| | | | | | 443 | 1.400 | -729.83 | -539.69 | 704.32 | 42.80 |
| 8 | -4.50 | -10.00 | 18.00 | 48.0 | 770 | 1.400 | -963.33 | 866.30 | 310.03 | 101.17 |
| | | | | | 445 | 1.400 | -823.47 | -788.76 | -662.62 | 14.81 |
| 9 | 0.00 | -10.00 | 72.00 | 48.0 | 2 | 1.000 | -970.98 | 29.46 | -0.00 | 106.23 |
| | | | | | 447 | 1.400 | -816.77 | -1004.10 | -327.02 | 37.20 |
| 10 | 4.00 | -10.00 | 120.00 | 48.0 | 2 | 1.000 | -970.98 | 29.46 | -0.00 | 115.27 |
| | | | | | 443 | 1.400 | -729.83 | -539.69 | 704.32 | 25.25 |

## Footing Design : Notes

Only max. force in piles is considered for design.
Pile coordinates X and Z are from the most left edge of the footing.
Plong= Lateral load in longitudinal direction at the top of pile, Kips.
Php= Available resisting horizontal component due to batter= batter * Vertical pile reaction, Kips.
Plong-Php= Remaining lateral force required to resist by pile.

## Max. Pile Reaction Used in Design: (without selfweight and surcharge)

| | |
|---|---|
| Factored pile reaction | 192.26 kips |
| Service pile reaction | 153.81 kips |
| Fatigue pile reaction | 18.46 kips |

## Reinforcement Schedule

| Dir | Quantity | Size | Bar dist. in | As total in^2 | Spacing in | Hook |
|---|---|---|---|---|---|---|
| X | 12 | # 8 | 3.50 | 9.48 | 12.45 | None |
| Z | 12 | # 8 | 4.50 | 9.48 | 12.45 | None |

## Flexure

| Dir ft | Loc | d in | Mmax kft | Comb | Asb_req in^2 | Asb_prv in^2 | Asb_eff in^2 | Ast_req in^2 | Ast_prv in^2 | Ast_eff in^2 |
|---|---|---|---|---|---|---|---|---|---|---|
| X | -3.00 | 38.50 | 576.8 | 2041 | 4.46 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |
| X | 3.00 | 38.50 | 384.5 | 2041 | 2.97 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |
| Z | -3.00 | 37.50 | 576.8 | 2041 | 4.59 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |
| Z | 3.00 | 37.50 | 576.8 | 2041 | 4.59 | 9.48 | 9.48 | 0.00 | 0.00 | 0.00 |

## Cracking/Fatigue

| Dir | Loc ft | d in | Cracking Mmax kft | Cracking Comb | Cracking fs ksi | Cracking ratio fs | Fatigue Mmax kft | Fatigue Comb | Fatigue fs ksi | Fatigue ratio fs |
|---|---|---|---|---|---|---|---|---|---|---|
| X | -3.00 | 38.50 | 461.4 | 859 | 15.99 | 0.65 | 55.4 | 1 | 1.92 | 0.09 |
| X | 3.00 | 38.50 | 307.6 | 859 | 10.66 | 0.44 | 36.9 | 1 | 1.28 | 0.06 |
| Z | -3.00 | 37.50 | 461.4 | 859 | 16.42 | 0.67 | 55.4 | 1 | 1.97 | 0.10 |
| Z | 3.00 | 37.50 | 461.4 | 859 | 16.42 | 0.67 | 55.4 | 1 | 1.97 | 0.10 |

## One Way Shear

| Col | Dir | Dist ft | Comb | d in | Vu kips | phi*Vc kips |
|---|---|---|---|---|---|---|
| 1 | X | -6.21 | Outside of Footing | — | — | — |
| | X | 6.21 | Outside of Footing | — | — | — |
| | Z | -6.13 | Outside of Footing | — | — | — |
| | Z | 6.13 | Outside of Footing | — | — | — |

## Two Way Shear

| # | Bo ft | Ao ft^2 | Comb | Avg. d in | Vu kips | phi*Vc kips |
|---|---|---|---|---|---|---|
| Columns | | | | | | |
| 1 | No Two Way Shear | | | | | |
| Piles - max | | | | | | |
| 4 | 17.28 | 18.66 | 2041 | 38.00 | 192.3 | 1584.7 |
| Piles - min | | | | | | |
| 1 | 7.82 | 15.22 | 2041 | 38.00 | 192.3 | 717.2 |

## Two Way Shear Note

TWO WAY SHEAR IN FOOTING IS NOT DESIGNED AND STIRRUPS ARE NOT CONSIDERED.

| | | | STRAND LAYOUT #1 | | | STRAND LAYOUT #2 | | | STRAND LAYOUT #3 | | | STRAND LAYOUT #4 | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Row | Spa | Y | Strands | n x d | Total Strands | Strands | n x d | Total Strands | Strands | n x d | Total Strands | Strands | n x d | Total Strands |
| 1 | 3.00 | 3.00 | 16 | 48 | **56** | 16 | 48 | **56** | | 0 | **0** | | 0 | **0** |
| 2 | 2.00 | 5.00 | 16 | 80 | | 16 | 80 | | | 0 | | | 0 | |
| 3 | 2.00 | 7.00 | 6 | 42 | **Sum(n x d)** | 6 | 42 | **Sum(n x d)** | | 0 | **Sum(n x d)** | | 0 | **Sum(n x d)** |
| 4 | 2.00 | 9.00 | 6 | 54 | **1126** | 6 | 54 | **1126** | | 0 | **0** | | 0 | **0** |
| 5 | 2.00 | 11.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 6 | 2.00 | 13.00 | | 0 | **Y strands** | | 0 | **Y strands** | | 0 | **Y strands** | | 0 | **Y strands** |
| 7 | 2.00 | 15.00 | | 0 | **20.107** | | 0 | **20.107** | | 0 | **0.000** | | 0 | **0.000** |
| 8 | 2.00 | 17.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 9 | 2.00 | 19.00 | | 0 | **Y beam** | | 0 | **Y beam** | | 0 | **Y beam** | | 0 | **Y beam** |
| 10 | 2.00 | 21.00 | | 0 | **48.750** | | 0 | **48.750** | | 0 | **0.000** | | 0 | **0.000** |
| 11 | 2.00 | 23.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 12 | 2.00 | 25.00 | | 0 | **e** | | 0 | **e** | | 0 | **e** | | 0 | **e** |
| 13 | 2.00 | 27.00 | | 0 | **28.643** | | 0 | **28.643** | | 0 | **0.000** | | 0 | **0.000** |
| 14 | 2.00 | 29.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 15 | 2.00 | 31.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 16 | 2.00 | 33.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 17 | 2.00 | 35.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 18 | 2.00 | 37.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 19 | 2.00 | 39.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 20 | 2.00 | 41.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 21 | 2.00 | 43.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 22 | 2.00 | 45.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 23 | 2.00 | 47.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 24 | 2.00 | 49.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 25 | 2.00 | 51.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 26 | 2.00 | 53.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 27 | 2.00 | 55.00 | 4 | 220 | | 4 | 220 | | | 0 | | | 0 | |
| 28 | 2.00 | 57.00 | 4 | 228 | | 4 | 228 | | | 0 | | | 0 | |
| 29 | 2.00 | 59.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 30 | 2.00 | 61.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 31 | 2.00 | 63.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 32 | 2.00 | 65.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 33 | 2.00 | 67.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 34 | 2.00 | 69.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 35 | 2.00 | 71.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 36 | 2.00 | 73.00 | | 0 | | | 0 | | | 0 | | | 0 | |
| 37 | 38.50 | 111.50 | | 0 | | | 0 | | | 0 | | | 0 | |
| 38 | 2.00 | 113.50 | 4 | 454 | | 4 | 454 | | | 0 | | | 0 | |

**Project:** I-75 HOV Lanes

Bridge 19C Straddle Bent Cap

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:** _____

# AASHTO 9.16 LOSS OF PRESTRESS

## AASHTO 9.16.2 Prestress Loss

$\Delta f_s = SH + ES + CR_c + CR_s$

Low-Relaxation — Type of Prestressing Strand

$f'_s$ = 270.00 Prestressing Steel Ultimate Strength (ksi)

### 9.16.2.1.1: Shrinkage

RH = 70.00 Mean Annual Ambient Relative Humidity (%)

**SH = 17,000 - 150 RH = 6,500 psi**

### 9.16.2.1.2: Elastic Shortening

| | | |
|---|---|---|
| $E_s$ = | 28,000 | Modulus of Elasticity of Prestress Steel (ksi) |
| $E_{ci}$ = | 3,990 | Modulus of Elasticity of Concrete at Release (ksi) |
| $A^*_s$ = | 12.152 | Total Area of Prestressing Steel ($in^2$) |
| $A_b$ = | 7320.0 | Area of Non-Composite Beam ($in^2$) |
| $I_b$ = | 8,649,900 | Moment of Inertia of Non-Composite Beam ($in^4$) |
| $M_{b\ D/L}$ = | 2780.0 | Dead Load Moment of Non-Composite Beam (kip-ft) |
| e = | 28.643 | Non-Composite Eccentricity of C.G. of P/S Steel @ Max. Moment (in) |

$f_{si}$ = 194.7 ksi (Assumed)* $f_{si}$ = 194.7 ksi (Calculated) **OK**

* per AASHTO, you can start with an assumption of 0.69 f's = 186.3 ksi

$P_i = f_{si}$ (Assumed) x $A^*_s$ = 2,366.0 kips

Concrete Stress @ C.G. Strands = $f_{cir}$

$$f_{cir} = \left( \frac{P_i}{A_b} + \frac{P_i e^2}{I_b} \right) - \frac{M_{b\ D/L} \times e}{I_b} = 437 \text{ psi}$$

$$ES = \frac{E_s}{E_{ci}} \times f_{cir} = 3{,}068 \text{ psi}$$

**Note:** _____

**Project:** I-75 HOV Lanes

Bridge 19C Straddle Bent Cap

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:** _____

# AASHTO 9.16 LOSS OF PRESTRESS

### 9.16.2.1.3: Creep of Concrete

| | | |
|---|---|---|
| $f_{cir}$ = | 437 | Concrete Stress @ C.G. of P/S Steel at Release (psi) |
| $M_{D/L}$ = | 11,479.0 | Total Non-Composite Dead Load Moment (kip-ft) |
| $M_{SD/L}$ = | 0.0 | Total Composite Dead Load Moment (kip-ft) |
| $I_c$ = | 8,649,900 | Moment of Inertia of Composite Beam ($in^4$) |
| $e_c$ = | 28.643 | Composite Eccentricity of C.G. of P/S Steel @ Max. Moment (in) |

Concrete Stress @ C.G. of Strands due to Other* Dead Loads = $f_{cds}$ = 346 psi

* All Dead Loads except for Beam Dead Load

**$CR_c$ = 12 $f_{cir}$ - 7 $f_{cds}$ = 2,826 psi**

### 9.16.2.1.4: Relaxation of Prestressing Steel

At Release: Assume losses due to Elastic Shortening only

**$CR_{si}$ = 5,000 - 0.10 x ES = 4,693 psi**

Final: As Defined by AASHTO E'qn 9-10A

**$CR_s$ = 5,000 - 0.10 x ES - 0.05 x (SH + $CR_c$) = 4,227 psi**

**INITIAL LOSS AT RELEASE**

**$\Delta f_{si}$ = ES + $CR_{si}$ = 7,761 psi = 3.83% of Initial**

**TOTAL LOSS**

**$\Delta f_s$ = SH + ES + $CR_c$ + $CR_s$ = 16,621 psi = 8.21% of Initial**

**Note:** _____

**Project:** I-75 HOV Lanes

Bridge 19C Straddle Bent Cap

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

# LFD FLEXURAL STRENGTH OF PRESTRESSED RECTANGULAR SECTIONS

**AASHTO 9.17**

## STRENGTH CAPACITY REDUCTION FACTORS

| | | | |
|---|---|---|---|
| $\phi_{ps}$ | 1.00 | **Factory Produced Prestressed Concrete Members** | **AASHTO 9.14** |

## BEAM DATA

| | | |
|---|---|---|
| **Span (ft)** | 48.00 | Span Length |
| **b (in)** | 48.00 | Width of Top of Prestressed Member |
| **h (in)** | 116.50 | Height of Prestressed Member |
| **$A_b$ (in$^2$)** | 7,320 | Area of Beam |
| **$I_b$ (in$^2$)** | 8,649,900 | Moment of Inertia of Beam |
| **$y_b$ (in)** | 48.75 | Neutral Axis of Prestressed Member |
| **$e_m$ (in)** | 28.64 | Eccentricity of Prestress Steel at Midspan |
| **$e_e$ (in)** | 28.64 | Eccentricity of Prestress Steel at End of Beam |
| **$t_{cf}$ (in)** | 116.50 | Compression Flange Thickness (Enter the Total Depth for a Rectangular Section) |

## CONCRETE DATA

| | | | |
|---|---|---|---|
| **$\gamma_c$ (kcf)** | 0.150 | Unit Weight of Concrete | |
| **$f'_{ci}$ (ksi)** | 4.800 | Compressive Strength of Concrete at Release | |
| **$f'_c$ (ksi)** | 6.000 | Compressive Strength of Concrete at 28 Days | |
| **$n_{ci}$** | 0.60 | Factor: Allowable Compressive Stress at Release | AASHTO 9.15.2.1 |
| **$n_{ti}$** | 3 | Factor: Allowable Tensile Stress at Release | AASHTO 9.15.2.1 |
| **$n_{cfa}$** | 0.60 | Factor: Allowable Final Compressive Stress, Case a | AASHTO 9.15.2.2 |
| **$n_{cfb}$** | 0.40 | Factor: Allowable Final Compressive Stress, Case b | AASHTO 9.15.2.2 |
| **$n_{cfc}$** | 0.40 | Factor: Allowable Final Compressive Stress, Case b | AASHTO 9.15.2.2 |
| **$n_{tf}$** | 6 | Factor: Allowable Final Tensile Stress | AASHTO 9.15.2.2 |

## PRESTRESS STRAND DATA

| | | |
|---|---|---|
| Low-Relaxation | | Type of Prestressing Strand |
| **$A_{str}$ (in$^2$)** | 0.217 | Area of an Individual Prestress Strand |
| **$n_s$** | 56 | Total Number of Strands |
| **$f'_s$ (ksi)** | 270 | Ultimate Stress of Prestress Steel |

## SERVICE LOAD MOMENTS

| | | |
|---|---|---|
| **$M_{Beam}$ (kip-ft)** | 2,780 | Dead Load Moment of Beam Only (at Release) |
| **$M_{DL}$ (kip-ft)** | 11,479 | Total Dead Load Moment (at Final Loading) |
| **$M_{LL+I}$ (kip-ft)** | 2,752 | Total Live Load + Impact Moment |
| **$M_S$ (kip-ft)** | 14,231 | Maximum Service Load Moment |

## FACTORED MOMENTS

| | | |
|---|---|---|
| **$M_U$ (kip-ft)** | 22,180 | Maximum Load Factored Moment |

**Project:** I-75 HOV Lanes

Bridge 19C Straddle Bent Cap

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

# LFD FLEXURAL STRENGTH OF PRESTRESSED RECTANGULAR SECTIONS

**AASHTO 9.17**

## BEAM PROPERTIES

| | | |
|---|---|---|
| $S_b$ (in$^3$) | 177,434 | Section Modulus to Bottom Flange = $I_b / y_b$ |
| $S_t$ (in$^3$) | 127,674 | Section Modulus to Top Flange = $I_b / (h - y_b)$ |
| d (in) | 96.39 | Depth to Centroid of Prestressing Steel = $h - y_b + e_m$ |

## PRESTRESS PROPERTIES

| | | |
|---|---|---|
| $f^*_y$ (ksi) | 243 | Ultimate Stress of Prestress Steel |
| $A^*_s$ (in$^2$) | 12.152 | Area of Prestressing Steel |
| 0.75 f's (ksi) | 202.5 | Prestress Prior to Transfer |

Initial Prestress Force

Calculated by "Prestress Loss GDOT" — Method to Calculate Initial Losses After Release

| | | |
|---|---|---|
| $f_{si}$ (ksi) | 194.7 | Prestress Immediately After Transfer |
| $P_i$ (kips) | 2,366 | Initial Prestress Force = $f_{si} \times A^*_s$ |

Final Prestress Force

Calculated by "Prestress Loss GDOT" — Method to Calculate Final Losses

| | | |
|---|---|---|
| $f_{loss}$ (ksi) | 16.6 | Total Loss of Prestress |
| $P_f$ (kips) | 2,259 | Final Prestress Force = $(0.75 \times f'_s - f_{loss}) A^*_s$ |

## FLEXURAL STRENGTH OF PRESTRESSED MEMBER

| | | | |
|---|---|---|---|
| $p^*$ | 0.0026 | Ratio of Prestress Steel = $A^*_s / bd$ | |
| $\gamma^*$ | 0.28 | Factor for Type of Prestressing Steel | AASHTO 9.1.2 |
| $\beta_1$ | 0.75 | Compression Zone Depth Ratio | AASHTO 8.16.2.7 |

Average Stress in Prestressing Steel at Ultimate Load

$$f^*_{su} = \text{MIN}\{ f'_s[1 - (\gamma^* / \beta_1)(p^* f'_s / f'_c)], f^*_y \} = 243.0 \text{ ksi}$$

$$\phi_{ps}M_n = \phi\left\{ A^*_s f^*_{su} d \left( 1 - 0.6 \frac{p^* f^*_{su}}{f'_c} \right) \right\} = 22{,}206 \text{ kip-ft} \quad 100\% \quad \textbf{OK}$$

**Project:** I-75 HOV Lanes

Bridge 19C Straddle Bent Cap

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

# LFD FLEXURAL STRENGTH OF PRESTRESSED RECTANGULAR SECTIONS

**AASHTO 9.17**

## SERVICE LOAD STRESS at RELEASE

Bottom Flange Compression at the End of the Beam — Allowable Stress $F_{bi}$ = 2.880 ksi

$$f_{bi} = \frac{-P_i}{A_b} + \frac{-P_i\, e_e}{S_b} = \mathbf{-0.705}\ \mathbf{ksi} \quad 24\% \quad \mathbf{OK}$$

Top Flange Tension at the End of the Beam — Allowable Stress $F_{bi}$ = 0.208 ksi

$$f_{ti} = \frac{-P_i}{A_b} + \frac{P_i\, e_e}{S_t} = \mathbf{0.208}\ \mathbf{ksi} \quad 100\% \quad \mathbf{OK}$$

## SERVICE LOAD STRESS at FINAL

Top Flange Compression at Midspan Case a: Maximum Service Load Combination

Allowable Stress $F_{tf_a}$ = 3.600 ksi

$$f_{tf_a} = \frac{-P_f}{A_b} + \frac{P_f\, e_m - M_S}{S_t} = \mathbf{-1.139}\ \mathbf{ksi} \quad 32\% \quad \mathbf{OK}$$

Top Flange Compression at Midspan Case b: Prestress + Dead Load

Allowable Stress $F_{tf_b}$ = 2.400 ksi

$$f_{tf_b} = \frac{-P_f}{A_b} + \frac{P_f\, e_m - M_{DL}}{S_t} = \mathbf{-0.881}\ \mathbf{ksi} \quad 37\% \quad \mathbf{OK}$$

Top Flange Compression at Midspan Case c: Live Load + ½ (Prestress + Dead Load)

Allowable Stress $F_{tf_c}$ = 2.400 ksi

$$f_{tf_c} = \frac{-\tfrac{1}{2} P_f}{A_b} + \frac{\tfrac{1}{2}(P_f\, e_m - M_{DL}) - M_{LL+I}}{S_t} = \mathbf{-0.699}\ \mathbf{ksi} \quad 29\% \quad \mathbf{OK}$$

Bottom Flange Tension at Midspan — Allowable Stress $F_{bf}$ = 0.465 ksi

$$f_{tf_a} = \frac{-P_f}{A_b} + \frac{-P_f\, e_m + M_S}{S_b} = \mathbf{0.289}\ \mathbf{ksi} \quad 62\% \quad \mathbf{OK}$$

**Note:**

**Project:** I-75 HOV Lanes

**Bridge 19C Straddle Bent Cap - Check Losses by AASHTO**

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

# LFD FLEXURAL STRENGTH OF PRESTRESSED RECTANGULAR SECTIONS

**AASHTO 9.17**

## STRENGTH CAPACITY REDUCTION FACTORS

| | | | |
|---|---|---|---|
| $\phi_{ps}$ | 1.00 | **Factory Produced Prestressed Concrete Members** | **AASHTO 9.14** |

## BEAM DATA

| | | |
|---|---|---|
| Span (ft) | 48.00 | Span Length |
| b (in) | 48.00 | Width of Top of Prestressed Member |
| h (in) | 116.50 | Height of Prestressed Member |
| $A_b$ (in$^2$) | 7,320 | Area of Beam |
| $I_b$ (in$^2$) | 8,649,900 | Moment of Inertia of Beam |
| $y_b$ (in) | 48.75 | Neutral Axis of Prestressed Member |
| $e_m$ (in) | 28.64 | Eccentricity of Prestress Steel at Midspan |
| $e_e$ (in) | 28.64 | Eccentricity of Prestress Steel at End of Beam |
| $t_{cf}$ (in) | 116.50 | Compression Flange Thickness (Enter the Total Depth for a Rectangular Section) |

## CONCRETE DATA

| | | | |
|---|---|---|---|
| $\gamma_c$ (kcf) | 0.150 | Unit Weight of Concrete | |
| $f'_{ci}$ (ksi) | 4.800 | Compressive Strength of Concrete at Release | |
| $f'_c$ (ksi) | 6.000 | Compressive Strength of Concrete at 28 Days | |
| $n_{ci}$ | 0.60 | Factor: Allowable Compressive Stress at Release | AASHTO 9.15.2.1 |
| $n_{ti}$ | 3 | Factor: Allowable Tensile Stress at Release | AASHTO 9.15.2.1 |
| $n_{cfa}$ | 0.60 | Factor: Allowable Final Compressive Stress, Case a | AASHTO 9.15.2.2 |
| $n_{cfb}$ | 0.40 | Factor: Allowable Final Compressive Stress, Case b | AASHTO 9.15.2.2 |
| $n_{cfc}$ | 0.40 | Factor: Allowable Final Compressive Stress, Case b | AASHTO 9.15.2.2 |
| $n_{tf}$ | 6 | Factor: Allowable Final Tensile Stress | AASHTO 9.15.2.2 |

## PRESTRESS STRAND DATA

| | | |
|---|---|---|
| Low-Relaxation | | Type of Prestressing Strand |
| $A_{str}$ (in$^2$) | 0.217 | Area of an Individual Prestress Strand |
| $n_s$ | 56 | Total Number of Strands |
| $f'_s$ (ksi) | 270 | Ultimate Stress of Prestress Steel |

## SERVICE LOAD MOMENTS

| | | |
|---|---|---|
| $M_{Beam}$ (kip-ft) | 2,780 | Dead Load Moment of Beam Only (at Release) |
| $M_{DL}$ (kip-ft) | 11,479 | Total Dead Load Moment (at Final Loading) |
| $M_{LL+I}$ (kip-ft) | 2,752 | Total Live Load + Impact Moment |
| $M_S$ (kip-ft) | 14,231 | Maximum Service Load Moment |

## FACTORED MOMENTS

| | | |
|---|---|---|
| $M_U$ (kip-ft) | 22,180 | Maximum Load Factored Moment |

**Project:** I-75 HOV Lanes

**Bridge 19C Straddle Bent Cap - Check Losses by AASHTO**

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

# LFD FLEXURAL STRENGTH OF PRESTRESSED RECTANGULAR SECTIONS
## AASHTO 9.17

### BEAM PROPERTIES

| | | |
|---|---|---|
| $S_b$ (in$^3$) | 177,434 | Section Modulus to Bottom Flange = $I_b / y_b$ |
| $S_t$ (in$^3$) | 127,674 | Section Modulus to Top Flange = $I_b / (h - y_b)$ |
| d (in) | 96.39 | Depth to Centroid of Prestressing Steel = $h - y_b + e_m$ |

### PRESTRESS PROPERTIES

| | | |
|---|---|---|
| $f^*_y$ (ksi) | 243 | Ultimate Stress of Prestress Steel |
| $A^*_s$ (in$^2$) | 12.152 | Area of Prestressing Steel |
| 0.75 f's (ksi) | 202.5 | Prestress Prior to Transfer |

Initial Prestress Force

AASHTO 9.16.2.1.2 (0.69 f's) — Method to Calculate Initial Losses After Release

| | | |
|---|---|---|
| $f_{si}$ (ksi) | 186.3 | Prestress Immediately After Transfer |
| $P_i$ (kips) | 2,264 | Initial Prestress Force = $f_{si} \times A^*_s$ |

Final Prestress Force

AASHTO Table 9.16.2.2 (45 ksi) — Method to Calculate Final Losses

| | | |
|---|---|---|
| $f_{loss}$ (ksi) | 45.0 | Total Loss of Prestress |
| $P_f$ (kips) | 1,914 | Final Prestress Force = $(0.75 \times f'_s - f_{loss}) A^*_s$ |

### FLEXURAL STRENGTH OF PRESTRESSED MEMBER

| | | | |
|---|---|---|---|
| $p^*$ | 0.0026 | Ratio of Prestress Steel = $A^*_s / bd$ | |
| $\gamma^*$ | 0.28 | Factor for Type of Prestressing Steel | AASHTO 9.1.2 |
| $\beta_1$ | 0.75 | Compression Zone Depth Ratio | AASHTO 8.16.2.7 |

Average Stress in Prestressing Steel at Ultimate Load

$$f^*_{su} = \text{MIN}\{ f'_s[1 - (\gamma^* / \beta_1)(p^* f'_s / f'_c)], f^*_y \} = 243.0 \text{ ksi}$$

$$\phi_{ps}M_n = \phi\left\{ A^*_s f^*_{su} d \left( 1 - 0.6 \frac{p^* f^*_{su}}{f'_c} \right) \right\} = 22{,}206 \text{ kip-ft} \quad 100\% \quad \text{OK}$$

**Project:** I-75 HOV Lanes

**Bridge 19C Straddle Bent Cap - Check Losses by AASHTO**

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

# LFD FLEXURAL STRENGTH OF PRESTRESSED RECTANGULAR SECTIONS

**AASHTO 9.17**

## SERVICE LOAD STRESS at RELEASE

Bottom Flange Compression at the End of the Beam — Allowable Stress $F_{bi}$ = 2.880 ksi

$$f_{bi} = \frac{-P_i}{A_b} + \frac{-P_i\, e_e}{S_b} = \mathbf{-0.675}\ \mathbf{ksi} \quad 23\% \quad \mathbf{OK}$$

Top Flange Tension at the End of the Beam — Allowable Stress $F_{bi}$ = 0.208 ksi

$$f_{ti} = \frac{-P_i}{A_b} + \frac{P_i\, e_e}{S_t} = \mathbf{0.199}\ \mathbf{ksi} \quad 96\% \quad \mathbf{OK}$$

## SERVICE LOAD STRESS at FINAL

Top Flange Compression at Midspan Case a: Maximum Service Load Combination

Allowable Stress $F_{tf_a}$ = 3.600 ksi

$$f_{tf_a} = \frac{-P_f}{A_b} + \frac{P_f\, e_m - M_S}{S_t} = \mathbf{-1.170}\ \mathbf{ksi} \quad 32\% \quad \mathbf{OK}$$

Top Flange Compression at Midspan Case b: Prestress + Dead Load

Allowable Stress $F_{tf_b}$ = 2.400 ksi

$$f_{tf_b} = \frac{-P_f}{A_b} + \frac{P_f\, e_m - M_{DL}}{S_t} = \mathbf{-0.911}\ \mathbf{ksi} \quad 38\% \quad \mathbf{OK}$$

Top Flange Compression at Midspan Case c: Live Load + ½ (Prestress + Dead Load)

Allowable Stress $F_{tf_c}$ = 2.400 ksi

$$f_{tf_c} = \frac{-\tfrac{1}{2} P_f}{A_b} + \frac{\tfrac{1}{2}(P_f\, e_m - M_{DL}) - M_{LL+I}}{S_t} = \mathbf{-0.714}\ \mathbf{ksi} \quad 30\% \quad \mathbf{OK}$$

Bottom Flange Tension at Midspan — Allowable Stress $F_{bf}$ = 0.465 ksi

$$f_{tf_a} = \frac{-P_f}{A_b} + \frac{-P_f\, e_m + M_S}{S_b} = \mathbf{0.392}\ \mathbf{ksi} \quad 84\% \quad \mathbf{OK}$$

**Note:**

## LOADS

### Pier Info:

Pier View: Upstation.

### Load Cases: 49

### Loadcase ID: D1 Name: Include Add'l Cap DL

Multiplier = 1.000

### Cap loads

| Type | Dir | Arm ft | Mag1 | x1/L | Mag2 | x2/L |
|---|---|---|---|---|---|---|
| UDL | Y | | 1.80 klf | 0.00 | | 1.00 |

### Bearing loads

| Line # | Bearing # | Dir | Load kips |
|---|---|---|---|
| 1 | 1 | Y | -146.81 |
| 1 | 2 | Y | -151.84 |
| 1 | 3 | Y | -151.84 |
| 1 | 4 | Y | -151.84 |
| 1 | 5 | Y | -146.81 |
| 2 | 1 | Y | -146.81 |
| 2 | 2 | Y | -151.84 |
| 2 | 3 | Y | -151.84 |
| 2 | 4 | Y | -151.84 |
| 2 | 5 | Y | -146.81 |

← DL + FWS

FWS = 0.030 ksf × 36' × 138'/(2×5) = 14.9 k/BRG

DL = 151.84 − 14.9 = 124.6 k

### Auto generation details:

### Generated Dead Load

Imported dead load reaction from Conspan for Pier: 1

Multiplier = 1.000

**Bearing loads**

| Line # | Bearing # | Dir | Load kips |
|---|---|---|---|
| 1 | 1 | Y | 0.00 |
| 1 | 2 | Y | 0.00 |
| 1 | 3 | Y | -7.00 |
| 1 | 4 | Y | -19.60 |
| 1 | 5 | Y | -63.00 |
| 2 | 1 | Y | 0.00 |
| 2 | 2 | Y | 0.00 |
| 2 | 3 | Y | -11.06 |
| 2 | 4 | Y | -30.97 |
| 2 | 5 | Y | -99.56 |

← MAX LL+I

**Auto generation details:**

**Generated Live Load**

**Longitudinal Reaction: Simple Span Distribution**

Selected Vehicles:

HS20 truck
H20/HS20 Lane Load
Military

**Transverse Positioning**

Number of loaded lanes = all combinations
Live Load Positions = Variable Spacing
Minimum Spacing Between Positions = 1.00 ft

Generate Braking/Longitudinal Force = Selected
Generate Centrifugal Force = Not Selected

Total number of Considered Truck Positions = 22
Total number of Possible Combination = 254

**Project:** I-75 HOV Lanes

**Bridge 19; 140'-0" Span; 74" Bulb-T; 8'-0" Spacing**

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

| | | | |
|---|---|---|---|
| $\eta_D$ | 1.00 | Ductility Load Modifier | A1.3.3 |
| $\eta_R$ | 1.00 | Redundancy Load Modifier | A1.3.4 |
| $\eta_I$ | 1.00 | Importance Load Modifier | A1.3.5 |

| | | |
|---|---|---|
| $V_{DC}$ | 124.6 | Unfactored Dead Load (kips) |
| $V_{DW}$ | 14.9 | Unfactored Future Wearing Surface (kips) |
| $V_{LL+I}$ | 99.6 | Unfactored Live Load + Impact (kips) |

| | | |
|---|---|---|
| fy | 60 | Reinforcement Yield Strength (ksi) |
| f'c | 6.000 | 28-Day Strength of Concrete (ksi) |
| $\gamma_c$ | 0.150 | Unit Weight of Concrete (kip/ft$^3$) |

| | | |
|---|---|---|
| Span | 140.000 | Span Length C-C Brg (ft) |
| $W_{c-c}$ | 36.000 | Bridge Width Curb-Curb (ft) |
| # Beams | 5 | |

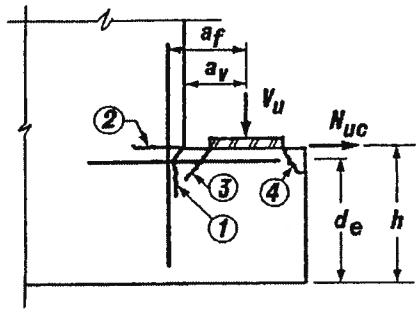


**Figure 5.13.2.5.1-1 Notation and Potential Crack Locations for Ledge Beams.**

| | | | |
|---|---|---|---|
| h | 36.000 | Depth of Beam Ledge (in) | Fig A5.13.2.5.1-1 |
| $d_e$ | 33.500 | Depth of Horizontal Steel (in) | Fig A5.13.2.5.1-1 |
| S | 96.000 | CL-CL Girder Spacing (in) | Fig A5.13.2.5.2-1 |
| c | 60.000 | CL Exterior Girder to End of Ledge Beam (in) | Fig A5.13.2.5.2-1 |
| W | 20.000 | Width of Bearing Area (in) | Fig A5.13.2.5.2-1 |
| $a_v$ | 12.000 | CL Brg to Face of Beam Web (in) | Fig A5.13.2.5.2-1 |
| $a_f$ | 14.500 | CL Brg to Main Hanger Reinforcement (in) | Fig A5.13.2.5.3-1 |
| L | 10.000 | Length of Bearing Area (in) | Fig A5.13.2.5.4-1 |
| $b_f$ | 96.000 | Width of Bottom of Inverted T (in) | Fig A5.13.2.5.5-2 |
| $d_f$ | 33.000 | Top Ledge to Compression Reinforcement (in) | Fig A5.13.2.5.5-2 |
| $b_t$ | 48.000 | Width of Top of Inverted T (in) | |

| | | |
|---|---|---|
| $Bar_{crack1}$ | #6 | Main Horizontal Reinforcement Across Crack 1 |
| $spa_{crack1}$ | 12.000 | Spacing of Main Horizontal Reinforcement Across Crack 1 (in) |
| $Bar_{crack2}$ | #6 | Main Hanger Reinforcement Across Crack 2 |
| $spa_{crack2}$ | 12.000 | Spacing of Main Hanger Reinforcement Across Crack 2 (in) |
| # Legs | 4 | Number of Legs in the Main Hanger Reinforcement Across Crack 2 |

| | | | |
|---|---|---|---|
| $\eta_i$ | 1.00 | Load Modifier | A1.3.2.1 |
| $\gamma_{p-DC}$ | 1.25 | Strength I Load Factor for Dead Load | Table A3.4.1-2 |
| $\gamma_{p-DW}$ | 1.50 | Strength I Load Factor for Future Wearing Surface | Table A3.4.1-2 |
| $\gamma_{-LL+I}$ | 1.75 | Strength I Load Factor for Live Load + Impact | Table A3.4.1-2 |

**Project:** I-75 HOV Lanes

**Bridge 19; 140'-0" Span; 74" Bulb-T; 8'-0" Spacing**

**Job No:** 100010194 **By:** DRB 10/20/2009 **Chk:**

| | | | |
|---|---|---|---|
| **# Lanes** | **3** | | |
| $N_{BR}$ | **4.1** | Unfactored Longitudinal Braking Force (kips) | |
| $\phi_v$ | **0.90** | Resistance Factor for Shear in Normal Weight Concrete | A5.5.4.2.1 |
| $\beta_1$ | **0.75** | Compresion Zone Depth Ratio | A5.7.2.2 |
| $b_l$ | **24.000** | Width of Ledge (in) | |

**CRACK 1: Design for Shear** **A5.13.2.5.2**

| | | | |
|---|---|---|---|
| $b_{w\text{-}v1}$ | **68.00** | Width of the Concrete Face for Shear Along Crack 1 (in) | A5.13.2.5.2 |
| $V_{n1}$ | **1,822.4** | Nominal Interface Shear Resistance of Concrete Along Cr 1 (kips) | A5.8.4.1-3 |
| $V_{ledge\text{-}DL1}$ | **5.1** | UnFactored Shear at Crack 1 due to Ledge Dead Load (kips) | |
| $V_{u1}$ | **358.7** | Strength I Factored Shear at Crack 1 (kips) | |

$\phi_v V_{n1}$ = **1,640.2** **kip** >= $\eta_i V_{u1}$ = **358.7** **kip** **OK 457%** A5.8.4.1-1

**CRACK 1: Design for Flexure and Horizontal Force** **A5.13.2.5.3**

| | | | |
|---|---|---|---|
| $b_{w\text{-}f1}$ | **92.50** | Width of the Concrete Face for Flexure Case 1 (in) | A5.13.2.5.3 |
| $A_s$ | **3.39** | Area of Tension Reinforcement in the Overhang ($in^2$) | |
| $c_{no}$ | **0.58** | Location of Neutral Axis (in) | A5.7.3.1.2-4 |
| $a_{no}$ | **0.43** | Depth of equivalent stress block (in) | A5.7.3.2.2 |
| $\phi_s$ | **0.90** | Resistance Factor for Strength Load | Figure AC5.5.4.2.1-1 |
| $N_{u1}$ | **7.2** | Strength I Factored Lateral Force at Crack 1 (kips) | |
| $\phi_s M_{n1}$ | **490.0** | Factored Flexural resistance of the Beam Overhang (kip-ft) | A5.7.3.2.1 |
| $M_{u1}$ | **358.7** | Strength I Factored Moment at Crack 1 (kip-ft) | |

$\phi_s M_{n1}$ = **490.0** **kip-ft** >= $\eta_i M_{u1}$ = **358.7** **kip-ft/ft** **OK 137%**

**CRACK 3: Design for Punching Shear** **A5.13.2.5.4**

Interior Pads, or Exterior Pads where c > S/2

| | | | |
|---|---|---|---|
| $V_{n3\text{-}1}$ | **1,097.5** | Nominal Punching Shear Resistance of Concrete (kips) | A5.13.2.5.4-1 |

Exterior Pads where (c < S/2) and (c - 0.5W < $d_e$)

**Project:** **I-75 HOV Lanes**

**Bridge 19; 140'-0" Span; 74" Bulb-T; 8'-0" Spacing**

**Job No:** **100010194** **By:** **DRB** **10/20/2009** **Chk:**

| | | | |
|---|---|---|---|
| $V_{n3-2}$ | **0.0** | Nominal Punching Shear Resistance of Concrete (kips) | A5.13.2.5.4-2 |

Exterior Pads where (c < S/2) and (c - 0.5W > $d_e$)

| | | | |
|---|---|---|---|
| $V_{n3-3}$ | **0.0** | Nominal Punching Shear Resistance of Concrete (kips) | A5.13.2.5.4-2 |
| $V_{n3}$ | **1097.5** | Controlling Nominal Punching Shear Resistance of Concrete (kips) | |
| $V_{ledge-DL3}$ | **5.6** | UnFactored Shear at Crack 3 due to Ledge Dead Load (kips) | |
| $V_{u3}$ | **359.4** | Strength I Factored Punching Shear at Crack 3 (kips) | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $\phi_v V_{n3}$ = | **987.8** | **kip** | >= | $\eta_i V_{u3}$ = | **358.7** | **kip** | **OK 275%** | A5.8.4.1-1 |

**CRACK 2: Design of Hanger Reinforcement - Inverted T** **A5.13.2.5.5**

| | | | |
|---|---|---|---|
| $A_{hr}$ | **0.44** | Area of One Leg of Hanger Reinforcement ($in^2$) | |

Service Limit State Single-Beam Ledges x 2

| | | | |
|---|---|---|---|
| $b_{w-ser}$ | **56.00** | Effective Length of Ledge = MIN(S, W + $3a_v$) (in) | |
| $V_{n2-ser}$ | **246.4** | Nominal Shear Resistance for Service Limit State (kips) | A5.13.2.5.5-1 |

Strength Limit State Single-Beam Ledges x 2

| | | | |
|---|---|---|---|
| $b_{w-str1}$ | **96.00** | Effective Length of Ledge = S (in) | |
| $V_{n2-str1}$ | **844.8** | Nominal Shear Resistance for Strength Limit State (kips) | A5.13.2.5.5-2 |

Inverted T-Beams

Check Minimum Interior & Exterior Beam Spacing

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **S =** | **96.0** | **in** | >= | **W/2 + $d_f$ =** | **43.0** | **in** | **OK 223%** |
| **c - W/2 =** | **50.0** | **in** | >= | **$d_f$ =** | **33.0** | **in** | **OK 152%** |

| | | | |
|---|---|---|---|
| $b_{w-str2}$ | **86.00** | Effective Length of Ledge = W + $2d_f$ (in) | |
| $V_{n2-str2}$ | **867.3** | Nominal Shear Resistance for Strength Limit State (kips) | A5.13.2.5.5-3 |
| $V_{n2-str}$ | **844.8** | Nominal Interface Shear Resistance for Strength Limit State (kips) | |
| $V_{ledge-DL2}$ | **28.8** | UnFactored Shear at Crack 2 due to Dead Load of the Ledge Only (kips) | |
| $V_{u2}$ | **740.7** | Strength I Factored Hanger Load at Crack 2 (kips) | |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $\phi_v V_{n2-str}$ = | **760.3** | **kip** | >= | $\eta_i V_u$ = | **740.7** | **kip** | **OK 103%** | A5.8.4.1-1 |

**Note:** **Approximate using LRFD Methodology with HS20 Load**

**Flexural Strength of Rectangular Sections** — **AASHTO 9.17.2**

| | | |
|---|---|---|
| $\phi_{pt} = 0.95$ | Post-Tensioned Cast-in-Place Concrete Members | AASHTO 9.14 |
| $\phi = 0.90$ | Plain Reinforced Concrete Members in Flexure | AASHTO 8.16.1.2.2 |

Beam Data

| | |
|---|---|
| $L_s = 54.00$ ft | Span Length |
| $b = 48.00$in | Width of Post-Tensioned Member |
| $d = 116.50$in | Depth of Post-Tensioned Member |
| $A_b = 7320$ in$^2$ | Area of Beam |
| $I_b = 8649900$ in$^4$ | Moment of Inertia of Beam |
| $y_b = 48.75$in | Neutral Axis of Post-Tensioned Member |
| $d_{ptm} = 110.50$in | Depth to Post-Tensioned Steel at Midspan |
| $d_{pte} = 96.00$in | Depth to Post-Tensioned Steel at End |
| $d_t = 112.50$in | Depth to Plain Reinforcing Steel |
| $t = 116.50$in | Compression Flange Thickness (Enter the Total Depth of a Rectangular Section) |

Concrete Data

| | | |
|---|---|---|
| $\gamma_c = 0.150$kcf | Unit Weight of Concrete | |
| $f_{ci} = 6.0$ksi | Compressive Strength of Concrete at Jacking | |
| $f_c = 6.0$ksi | Compressive Strength of Concrete at 28 days | |
| $f_{loss} = 34$ksi | Estimated Loss of Prestress (Excluding Friction) | AASHTO 9.16.2.2 |
| $n_{ci} = 0.55$ | Factor: Allowable Compressive Stress at Release | AASHTO 9.15.2.1 |
| $n_{ti} = 3$ | Factor: Allowable Tensile Stress at Release | AASHTO 9.15.2.1 |
| $n_{cfa} = 0.60$ | Factor: Allowable Final Compressive Stress, Case a | AASHTO 9.15.2.2 |
| $n_{cfb} = 0.40$ | Factor: Allowable Final Compressive Stress, Case b | AASHTO 9.15.2.2 |
| $n_{cfc} = 0.40$ | Factor: Allowable Final Compressive Stress, Case c | AASHTO 9.15.2.2 |
| $n_{tf} = 6$ | Factor: Allowable Final Tensile Stress | AASHTO 9.15.2.2 |

Reinforcing Steel Data

| | | | |
|---|---|---|---|
| None | Rebar Size | $D_{bar} = 0.000$ in | $A_{bar} = 0.00$ in$^2$ |
| $n_b = 8$ | Total Number of Plain Reinforcing Bars | | |
| $f_{sy} = 60$ksi | Yield Stress of Plain Reinforcing Steel | | |

## Post-Tension Strand Data

| | | |
|---|---|---|
| $A_{str} = 0.217\ in^2$ | Area of an Individual Post-Tension Strand | |
| $n_s = 54$ | Total Number of Strands | |
| $f_s = 270ksi$ | Ultimate Stress of Post-Tension Steel | |
| $f_{psy} = 243ksi$ | Yield of Post-Tension Steel | AASHTO 9.1.2 |
| $\gamma_{pt} = 0.28$ | Factor for Type of Post-Tension Steel | AASHTO 9.1.2 |
| $K = \frac{0.0002}{ft}$ | Friction Wobble Coefficient | AASHTO 9.16.1 |
| $\mu = 0.23$ | Friction Curvature Coefficient | AASHTO 9.16.1 |
| $\alpha = 0.200rad$ | Angular Change of Post-Tension Steel | AASHTO 9.16.1 |

## Service Load Moments

| | |
|---|---|
| $M_{DL} = 11479kip\ ft$ | Total Dead Load Moment |
| $M_{LLI} = 2752kip\ ft$ | Total Live Load + Impact Moment |
| $M_S = 14020kip\ ft$ | Total Maximum Service Load Moment |

## Load Factor Moments

| | |
|---|---|
| $M_U = 22180kip\ ft$ | Total Maximum Load Factor Moment |

Section Modulus $S_b = \frac{I_b}{y_b}$ $S_b = 177433.846\ in^3$ $S_t = \frac{I_b}{d - y_b}$ $S_t = 127673.801\ in^3$

Area of Post-Tensioned Steel $A_{pt} = n_s\ A_{str}$ $A_{pt} = 11.718\ in^2$

Eccentricity @ Midspan $e_{ptm} = y_b - d + d_{ptm}$ $e_{ptm} = 42.75\ in$

Eccentricity @ End of Beam $e_{pte} = y_b - d + d_{pte}$ $e_{pte} = 28.25\ in$

Total Initial Post-Tension Force $P_{pti} = 0.70\ f_s\ A_{pt}$ $P_{pti} = 2215\ kip$

Ratio of Post-Tensioned Steel $p_{pt} = \frac{A_{pt}}{b\ d_{ptm}}$ $p_{pt} = 0.002$

Area of Plain Reinforcing Steel $A_s = n_b\ A_{bar}$ $A_s = 0.000\ in^2$

Ratio of Plain Reinforcing Steel $p = \frac{A_s}{b\ \max(d_t, 0.01in)}$ $p = 0.000$

Compression zone depth ratio AASHTO 8.16.2.7

$$\beta_1 = \begin{cases} 0.85 & \text{if } f_c \le 4\ ksi \\ \max\left[0.65, 0.85 - \frac{(f_c - 4\ ksi)\ 0.05}{1\ ksi}\right] & \text{otherwise} \end{cases}$$

$\beta_1 = 0.75$

Average Stress of Post-Tensioned Steel at Ultimate Load (Assume Bonded Member) AASHTO 9.17.4

$$f_{su} = \min\left[f_s\left[1 - \left(\frac{\gamma_{pt}}{\beta_1}\right)\left(\frac{p_{pt}\ f_s}{f_c} + \frac{d_{ptm}}{\max(d_t, 0.01in)}\ \frac{p\ f_s}{f_c}\right)\right], f_{psy}\right] \quad f_{su} = 243.0\ ksi$$

Friction Loss $T_x = 0.70\ f_s$ $T_x = 189$ ksi

AASHTO 9.16.1

Assumptions:
Compute Loss at Midspan
Assume $T_x = 0.70 f_s$

$$T_o = \begin{cases} T_x\, e^{\left(K\frac{L_s}{2}+\mu\,\alpha\right)} & \text{if } K\frac{L_s}{2}+\mu\,\alpha > 0.3 \\ \left[T_x\left(1+K\frac{L_s}{2}+\mu\,\alpha\right)\right] & \text{otherwise} \end{cases}$$

$T_o = 198.7$ ksi

$FR = T_o - T_x$ $FR = 9.7$ ksi

Final Total Post-Tension Force $P_{ptf} = P_{pti} - (f_{loss} + FR)\ A_{pt}$ $P_{ptf} = 1702$ kip

Type of Memeber [Rectangular / Flanged]

$$Section_{type} = \begin{cases} \text{"Rectangular"} & \text{if } \frac{A_{pt}\, f_{su} + A_s\, f_{sy}}{0.85\, f_c\, b} \le t \\ \text{"Flanged"} & \text{otherwise} \end{cases}$$

$Section_{type}$ = "Rectangular"

**NOTE Formulas are Incomplete for Flanged Sections**

Maximum Post-Tensioning Steel

AASHTO 9.18

$$Max_{pt} = \begin{cases} \text{"OK"} & \text{if } Section_{type} = \text{"Rectangular"} \wedge \frac{p_{pt}\, f_{su}}{f_c} \le 0.36\,\beta_1 \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Max_{pt}$ = "OK"

**Flexural Strength of Post-Tensioned Member**

AASHTO Eq'n 9-13a

$$\phi M_{npt} = \phi_{pt}\left[A_{pt}\, f_{su}\, d_{ptm}\left[1-0.6\left(\frac{p_{pt}\, f_{su}}{f_c}+\frac{d_t}{d_{ptm}}\,\frac{p\, f_{sy}}{f_c}\right)\right] + A_s\, f_{sy}\, d_t\left[1-0.6\left(\frac{d_{ptm}}{\max(d_t, 0.01\text{in})}\,\frac{p_{pt}\, f_{su}}{f_c}+\frac{p\, f_{sy}}{f_c}\right)\right]\right]$$

$\phi M_{npt} = 23572$ ft kip

$$Result_{factor} = \begin{cases} \text{"OK"} & \text{if } \phi M_{npt} \ge M_U \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Result_{factor}$ = "OK"

**Flexural Strength of Plain Reinforced Member Only**

AASHTO Eq'n 8-15

$$\phi M_n = \phi\left[A_s\, f_{sy}\, d_t\left[1-0.6\left(\frac{p\, f_{sy}}{f_c}\right)\right]\right]$$

$\phi M_n = 0$ ft kip

Factored Beam Dead Load $M_{U_DL} = 1.3\, A_b\, \gamma_c\, \frac{L_s^2}{8}$ $M_{U_DL} = 3613$ ft kip

$$Result_{plain} = \begin{cases} \text{"OK"} & \text{if } \phi M_n \ge M_{U_DL} \\ \text{"OK"} & \text{if } bar_{size} = 1 \\ \text{"No Good"} & \text{otherwise} \end{cases}$$

$Result_{plain}$ = "OK"

**Service Load Stress at Release**

Bottom Flange Compression $f_{bi} = \frac{-P_{pti}}{A_b} + \frac{-P_{pti}\ e_{pte}}{S_b}$ $f_{bi} = -0.655$ ksi

$F_{bi} = -n_{ci}\ f_{ci}$ $F_{bi} = -3.300$ ksi $Result_{bi} = \begin{cases} \text{"OK"} & \text{if } f_{bi} \ge F_{bi} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

$Result_{bi}$ = "OK"

Top Flange Tension $f_{ti} = \frac{-P_{pti}}{A_b} + \frac{P_{pti}\ e_{pte}}{S_t}$ $f_{ti} = 0.187$ ksi

$F_{ti} = n_{ti}\sqrt{\frac{f_{ci}}{0.001\ \text{ksi}}}$ psi $F_{ti} = 0.232$ ksi $Result_{ti} = \begin{cases} \text{"OK"} & \text{if } f_{ti} \le F_{ti} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

$Result_{ti}$ = "OK"

**Service Load Stress at Final** AASHTO 9.15.2.2

Top Flange Compression Case a: Maximum Service Load Combination

$f_{tf_a} = \frac{-M_S}{S_t} + \frac{-P_{ptf}}{A_b} + \frac{P_{ptf}\ e_{ptm}}{S_t}$ $f_{tf_a} = -0.980$ ksi

$F_{tf_a} = -n_{cfa}\ f_c$ $F_{tf_a} = -3.600$ ksi $Result_{tf_a} = \begin{cases} \text{"OK"} & \text{if } f_{tf_a} \ge F_{tf_a} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

$Result_{tf_a}$ = "OK"

Top Flange Compression Case b: Prestress + Dead Load

$f_{tf_b} = \frac{-M_{DL}}{S_t} + \frac{-P_{ptf}}{A_b} + \frac{P_{ptf}\ e_{ptm}}{S_t}$ $f_{tf_b} = -0.741$ ksi

$F_{tf_b} = -n_{cfb}\ f_c$ $F_{tf_b} = -2.400$ ksi $Result_{tf_b} = \begin{cases} \text{"OK"} & \text{if } f_{tf_b} \ge F_{tf_b} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

$Result_{tf_b}$ = "OK"

Top Flange Compression Case c: Live Load + $^1/_2$ (Prestress + Dead Load)

$f_{tf_c} = \frac{-M_{LLI}}{S_t} + \frac{1}{2}\left(\frac{-M_{DL}}{S_t} + \frac{-P_{ptf}}{A_b} + \frac{P_{ptf}\ e_{ptm}}{S_t}\right)$ $f_{tf_c} = -0.629$ ksi

$F_{tf_c} = -n_{cfc}\ f_c$ $F_{tf_c} = -2.400$ ksi $Result_{tf_c} = \begin{cases} \text{"OK"} & \text{if } f_{tf_c} \ge F_{tf_c} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

$Result_{tf_c}$ = "OK"

Bottom Flange Tension

$f_{bf} = \frac{M_S}{S_b} + \frac{-P_{ptf}}{A_b} + \frac{-P_{ptf}\ e_{ptm}}{S_b}$ $f_{bf} = 0.305$ ksi

$F_{bf} = n_{tf}\sqrt{\frac{f_c}{0.001\ \text{ksi}}}$ ps $F_{bf} = 0.465$ ksi $Result_{bf} = \begin{cases} \text{"OK"} & \text{if } f_{bf} \le F_{bf} \\ \text{"No Good"} & \text{otherwise} \end{cases}$

$Result_{bf}$ = "OK"