HOPPER ENGINEERING ASSOCIATES

4'x8' STEELDECK® PLATFORM EVALUATION 1

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PRO 196

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CALCULATION SHEET

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Revision 0:

Prepared by: _______________ 1/25/07

Scott Bauer — Date

Reviewed by: _______________ Jan. 30, 2007

Wes Brown — Date

Revision 1:

Prepared by: ______________________ 4/5/07

Scott Bauer — Date

Reviewed by: ______________________ April 5, 2007

Wes Brown — Date

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1.1 Problem Statement

The purpose of this calculation package is to evaluate a 4'x8' Steeldeck® platform for a UBC [1] stage live load of 125 psf and to determine the maximum allowable leg length assuming a lateral seismic load of 0.1g. Wind uplift will also be evaluated. 1

1.2 Analysis Approach

The plywood decking is analyzed using hand calculations only. The steel truss girders are analyzed using the structural analysis program STAAD [4], with much of the input determined by hand calculations. The pipe legs are analyzed using hand calculations based on loads provided by the STAAD output.

1.3 Results Summary

The plywood deck, steel truss girders, and pipe legs are capable of supporting a live load of 125 psf. Under both compressive and lateral seismic loads, the maximum allowed unbraced length of pipe leg is 6'-0". In order to prevent uplift, the maximum allowed wind speed is 40 mph. 1

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Refer to Figures 2.1 – 2.2 for drawings of the 4'x8' Platform. These details are from the full size drawings [6]. All platform details are provided by Steeldeck, Inc. [5]. 1
Multiple 4'x8' panels can be joined together to form large platforms.

Each platform panel is composed of a layer of ¾" MDO plywood fixed to a welded steel frame below by a total of fifteen #14 screws spaced at 24" on center. There are two 8' long and two 4' long trusses forming the perimeter of the frame. An 8' long truss spans across the center of the platform in the long direction, and two separate 2' long trusses span across the center of the platform in the short direction. These 1
central trusses are welded to each other at their point of intersection. The total 4'x8' platform depth is 7" and weighs approximately 160 pounds.

The top chord of the trusses is composed of a ¾" x 1-1/2" x 16 ga. ASTM A-513 steel tube. The bottom chord of the trusses is composed of a ¾" x ¾" x 14 ga. ASTM A-513 steel tube. The truss web is composed of a 5/16" diameter steel rod (A36 steel) bent in a serpentine shape. The clear distance between the top and bottom chords is 3-1/2". The total depth of the steel truss is 5-3/4", and the depth of the frame and plywood is 6-1/2".

The four corners of the platform consist of 7" long 2.36" x 2.36" x 0.158" steel tube corner post pockets that can receive the pipe legs. The pipe legs are 1-1/2" diameter Schedule 40 (A53 Gr. B) steel pipes that vary in length from 8" to 12'-0". In a multi-panel platform, there is only one pipe leg at the corner intersections, meaning that a single pipe leg will be supporting the corners of four platforms. The other three platforms are attached to the single pipe leg by means of a clamped coupler. Coupler 1 is outside the scope of this analysis. Bracing for the pipe legs is typically provided when the height of the platform is 4' or higher.

1

In a multi-panel configuration, the 4'x8' panels are attached to each other with two 10 mm cap screws on each of the four sides. The perimeter of the finished deck can be fitted with a 44" high steel guardrail attached to the perimeter truss by a bolt-on clamp plate.

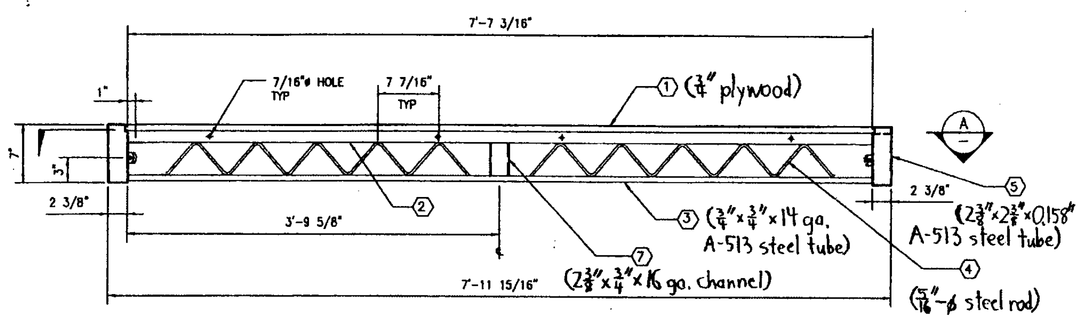


FIGURE 2.1 – ELEVATION

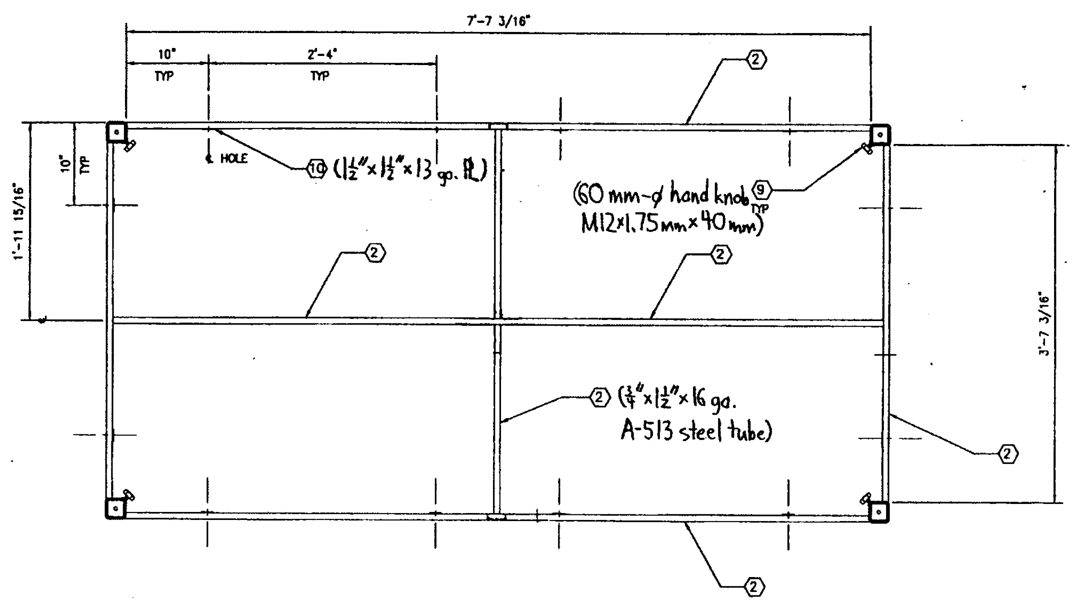


FIGURE 2.2– SECTION A

The plywood is analyzed by hand as a rectangular plate using the plate stress formulas [3].

The steel trusses are analyzed as a composite section comprising the steel truss frame and the effective width of the plywood deck. All of the relevant section properties are calculated by hand and inputted into the structural analysis program STAAD [4]. The live load placed on each member is proportional to the tributary width of the member, which varies along the length of the member. The overall uniform live load is 125 psf [1].

The pipe legs are analyzed by hand using loads provided by the STAAD output. The longest and therefore most critical pipe leg is 12'-0", so the analysis will begin at this length. The pipe leg will be analyzed assuming that there is no lateral bracing in order to determine the maximum allowable unbraced length of pipe leg. Two load cases will be used: first with both compression due to vertical loads and bending due to lateral seismic loads, and then with compression only. Since the platforms are temporary structures with limited exposure, the seismic loading used is 0.1 x dead load. The structure is also checked for uplift caused by wind pressures.

Analysis of the base plates and soil bearing pressures is beyond the scope of this calculation package.

Steel design is per the AISC Steel Manual [2]. ASTM A-513 "as welded" steel grade 1010 has a yield stress of 32 ksi. A36 steel has a yield stress of 36 ksi. A53 Gr. B steel has a yield stress of 35 ksi.

4.1 Plywood Analysis

The deck consists of one layer of $\frac{3}{4}''$ plywood. This essentially forms a plate of dimensions 2'×4' with fixed edges.

$\sigma_{max} = \frac{-\beta_1 q b^2}{t^2}$ [3, page 508, case 8A]

$\beta_1 = 0.4974$ for $a/b = 2$

$b = 24''$

$t = \frac{3}{4}''$

$q = 125 \text{ psf} + 35 \frac{\text{lb}}{\text{ft}^3} \times \frac{0.75''}{12} = 127.19 \text{ psf} = 0.883 \text{ psi}$

$\sigma_{max} - \frac{(-0.4974)(0.883 \text{ psi})(24'')^2}{(\frac{3}{4}'')^2} = 449.9 \text{ psi}$

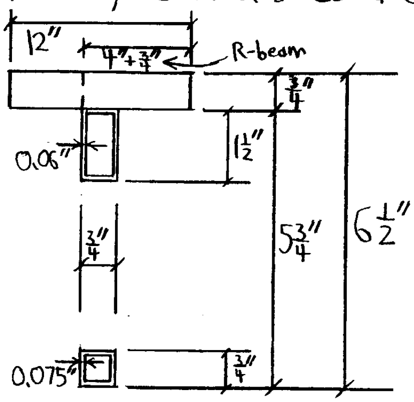
$F_b' = 1000 \text{ psi} \times 1.0 = 1000 \text{ psi} > 449.9 \text{ psi}$ ok

Douglas Fir Construction Grade [7]

psf max. allow $= \frac{1000}{449.9} \times 125 \text{ psf} = \underline{278 \text{ psf}}$

4.2 Truss Analysis

Analyze truss as a composite section with the plywood deck.



$b_{flange} \le \frac{1}{4} \times L = \frac{1}{4} \times 4' = 1' = 12''$
$b_{ohang} \le 8 \times t = 8 \times \frac{3}{4}'' = 6''$
$\le \frac{1}{2} \times \text{spacing} \approx \frac{1}{2} \times 2' = 1' = 12''$
} for T-beam

$b_{ohang} \le \frac{1}{12} \times L = \frac{1}{12} \times 4' = \frac{1}{3}' = 4''$
$\le 6 \times t = 6 \times \frac{3}{4}'' = 4\frac{1}{2}''$
$\le \frac{1}{2} \times \text{spacing} \approx \frac{1}{2} \times 2' = 1' = 12''$
} for R-beam

T-beam:

$b_{flange} = 12'' \times \frac{1500 \text{ ksi}}{29000 \text{ ksi}} = 0.62''$ (as steel)

$A_x = 0.62'' \times 0.75'' + (1.5'' \times 0.75'' - 1.38'' \times 0.63'') + (0.75'' \times 0.75'' - 0.6'' \times 0.6'')$
$= 0.465 + 0.2556 + 0.2025 \text{ in}^2 = 0.9231 \text{ in}^2$

$A_y = 0.62'' \times 0.75'' + 2 \times 1.5'' \times 0.06'' + 2 \times 0.75'' \times 0.075'' = 0.7575 \text{ in}^2$

$A_z = 2 \times 0.75'' \times 0.06'' + 2 \times 0.75'' \times 0.075'' = 0.2025 \text{ in}^2$

$Y_p = 6.5''$

$Z_p = 0.75''$

$\bar{y} = \frac{\Sigma A y}{\Sigma A} = \frac{0.465 \text{ in}^2 \times 6.125'' + 0.2556 \text{ in}^2 \times 5'' + 0.2025 \text{ in}^2 \times 0.375''}{0.9231 \text{ in}^2} = \frac{4.202 \text{ in}^3}{0.9231 \text{ in}^2} = 4.552''$

$I_z = \frac{(0.62'')(0.75'')^3}{12} + (0.465 \text{ in}^2)(6.125'' - 4.552'')^2 + \frac{(0.75'')(1.5'')^3}{12} - \frac{(0.63'')(1.38'')^3}{12}$
$+ (0.2556 \text{ in}^2)(5'' - 4.552'')^2 + \frac{(0.75'')^4}{12} - \frac{(0.6'')^4}{12} + (0.2025 \text{ in}^2)(4.552'' - 0.375'')^2$
$= 0.021797 + 1.150563 + 0.210938 - 0.137974 + 0.051300$
$+ 0.026367 - 0.010800 + 3.533084 \text{ in}^4 = 4.845 \text{ in}^4$

$$I_y = \frac{(0.75'')(0.62'')^3}{12} + \frac{(1.5'')(0.75'')^3}{12} - \frac{(1.38'')(0.63'')^3}{12} + \frac{(0.75'')^4}{12} - \frac{(0.6'')^4}{12}$$
$$= 0.014896 + 0.052734 - 0.028755 + 0.026367 - 0.010800 \text{ in}^4$$
$$= 0.054 \text{ in}^4$$
$$I_x \approx 1.0 \text{ in}^4$$

R-beam:

$$b_{flange} = 4.75'' \times \frac{1500 \text{ ksi}}{29000 \text{ ksi}} = 0.25'' \text{ (as steel)}$$
$$A_x = \overbrace{0.25'' \times 0.75''}^{.1875} + 0.2556 + 0.2025 \text{ in}^2 = 0.6456 \text{ in}^2$$
$$A_y = 0.25'' \times 0.75'' + 2 \times 1.5'' \times 0.06'' + 2 \times 0.75'' \times 0.075'' = 0.48 \text{ in}^2$$
$$A_z = 0.2025 \text{ in}^2$$
$$Y_D = 6.5''$$
$$Z_D = 0.75''$$
$$\bar{y} = \frac{\Sigma Ay}{\Sigma A} = \frac{0.1875 \text{ in}^2 \times 6.125'' + 0.2556 \text{ in}^2 \times 5'' + 0.2025 \text{ in}^2 \times 0.375''}{0.6456 \text{ in}^2} = \frac{2.502 \text{ in}^3}{0.6456 \text{ in}^2} = 3.876''$$
$$I_z = \frac{(0.25'')(0.75'')^3}{12} + (0.1875 \text{ in}^2)(6.125'' - 3.876'')^2 + \frac{(0.75'')(1.5'')^3}{12} - \frac{(0.63'')(1.38'')^3}{12}$$
$$+ (0.2556 \text{ in}^2)(5'' - 3.876'')^2 + \frac{(0.75'')^4}{12} - \frac{(0.6'')^4}{12} + (0.2025 \text{ in}^2)(3.876'' - 0.375'')^2$$
$$= 0.008789 + 0.948375 + 0.210938 - 0.137974 + 0.322919$$
$$+ 0.026367 - 0.010800 + 2.482043 \text{ in}^4 = 3.851 \text{ in}^4$$
$$I_y = \frac{(0.75'')(0.25'')^3}{12} + 0.052734 - 0.028755 + 0.026367 - 0.010800 \text{ in}^4$$
$$= 0.000977 + 0.052734 - 0.028755 + 0.026367 - 0.010800 \text{ in}^4$$
$$= 0.041 \text{ in}^4$$
$$I_x \approx 1.0 \text{ in}^4$$

These sections will be inputted into STAAD as User-Defined General Sections. This type of section requires additional cross-sectional properties.

T-beam:

$D = Y_D = 6.5''$

$TD = \frac{2 \times 0.06'' + 2 \times 0.075''}{2} = 0.135''$

$B = 0.75''$

$TB = 0.135''$

$S_z = \frac{I_z}{y} = \frac{4.845\ in^4}{4.552''} = 1.064\ in^3$ (minimum)

$S_y = \frac{I_y}{z} = \frac{0.054\ in^4}{0.75''/2} = 0.144\ in^3$

$P_z = P_y = HSS = DEE = 0$

R-beam:

$D = Y_D = 6.5''$

$TD = 0.135''$

$B = 0.75''$

$TB = 0.135''$

$S_z = \frac{I_z}{y} = \frac{3.851\ in^4}{3.876''} = 0.994\ in^3$ (minimum)

$S_y = \frac{I_y}{z} = \frac{0.041\ in^4}{0.75''/2} = 0.109\ in^3$

$P_z = P_y = HSS = DEE = 0$

Selfweight:

Inner beams (T-beams):

Plywood: $35\ \frac{lb}{ft^3} \times \frac{0.75''}{12} \times 2' = 4.375$ plf

Upper chord: $490\ \frac{lb}{ft^3} \times \frac{0.2556\ in^2}{144} = 0.870$ plf

Lower chord: $490\ \frac{lb}{ft^3} \times \frac{0.2025\ in^2}{144} = 0.689$ plf

Truss web: $\frac{5}{16}''$-ϕ steel rod

$$A = \pi(\tfrac{5}{16}'')^2/4 = 0.0767\ in^2$$

$$490\ \frac{lb}{ft^3} \times \frac{0.0767\ in^2}{144} \times 1.414 = 0.369\ plf$$

$\Longrightarrow$ Total weight = 6.303 plf = 0.00052525 kip/in

Outer beams (R-beams):

Plywood: $35\ \frac{lb}{ft^3} \times \frac{0.75''}{12} \times 1' = 2.1875$ plf

Upper chord: 0.870 plf

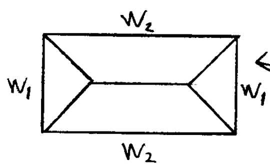
Lower chord: 0.689 plf

Truss web: 0.369 plf

$\Longrightarrow$ Total weight = 4.116 plf = 0.00034296 kip/in

Live load:

2'x4' section:



← Tributary areas

Live load = 125 psf

$W_{1max} = 125 \text{ psf} \times 1' = 125 \text{ plf} = 0.010417 \text{ kip/in}$

$W_{1min} = 0 \text{ plf}$

$W_{2max} = 125 \text{ psf} \times 1' = 125 \text{ plf} = 0.010417 \text{ kip/in}$

$W_{2min} = 0 \text{ plf}$

See full live load distribution on page A4

See Appendix A for STAAD analysis of Steel Deck beams.

From STAAD, the maximum vertical deflection under a live load of 125 psf is 0.144" (center node)

$$\text{Allowable deflection} = \frac{L}{360} = \frac{\sqrt{(96")^2 + (48")^2}}{360} = \frac{107.3"}{360}$$

$$= 0.298" > 0.144" \quad \underline{\text{ok}}$$

From STAAD, $M_{max} = 16.75$ kip-in, $f_{bmax} = 16.7$ ksi

$F_b = 0.66 \times 32 \text{ ksi} = 21.12 \text{ ksi} > 16.7 \text{ ksi}$ ok

$T = C = M/d = 16.75 \text{ kip-in}/(5.75'' - \frac{1.5''}{2} - \frac{0.75''}{2}) = 3.62 \text{ kips}$

$\Rightarrow$ Compressive stress in top chord:

$3.62 \text{ kips}/(1.50'' \times 0.75'' - 1.38'' \times 0.63'') = 14.16 \text{ ksi}$

$r_{min} = \sqrt{\frac{I}{A}}$

$I = \frac{(1.5'')(0.75'')^3}{12} - \frac{(1.38'')(0.63'')^3}{12} = 0.023979 \text{ in}^4$

$A = (1.5'')(0.75'') - (1.38'')(0.63'') = 0.2556 \text{ in}^2$

$r = \sqrt{\frac{0.023979 \text{ in}^4}{0.2556 \text{ in}^2}} = 0.306''$

$\frac{KL}{r} = \frac{0.65 \times 24''}{0.306''} = 50.98$

$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2(29000 \text{ ksi})}{32 \text{ ksi}}} = 133.75 > 50.98$

$\Rightarrow F_a = \frac{\left[1 - \frac{(KL/r)^2}{2C_c^2}\right] F_y}{\frac{5}{3} + \frac{3(KL/r)}{8C_c} - \frac{(KL/r)^3}{8C_c^3}} = \frac{\left[1 - \frac{50.98^2}{2(133.75)^2}\right] 32 \text{ ksi}}{\frac{5}{3} + \frac{3(50.98)}{8(133.75)} - \frac{(50.98)^3}{8(133.75)^3}}$

$= \frac{0.927(32 \text{ ksi})}{\frac{5}{3} + 0.14293 - 0.00692}$

$= 16.46 \text{ ksi} > 14.16 \text{ ksi}$ ok

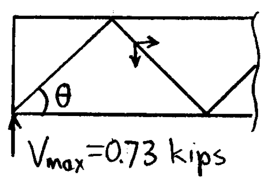
Check local buckling:

$\frac{b}{t} = \frac{1.5''}{0.06''} = 25$

$\frac{190}{\sqrt{F_y}} = \frac{190}{\sqrt{32}} = 33.6 > 25 \Rightarrow$ section is compact ok

1

Truss members in web:



$\theta \approx 45°$

Web truss members are $\frac{5''}{16}$-ø Steel rods

$C = 0.73 \text{ kips}/\sin 45° = 1.03 \text{ kips}$

$r \approx \frac{d}{4} = \frac{5/16''}{4} = 0.0781''$

$L \approx (3.5^2 + 3.5^2)^{\frac{1}{2}} = 4.95''$

$K = 0.65$ (fixed-fixed)

$\frac{KL}{r} = \frac{0.65(4.95'')}{0.0781''} = 41.2$

$\Longrightarrow F_a = 19.09 \text{ ksi}$

$f_a = \frac{P}{A} = \frac{1.03 \text{ kips}}{\pi(5/32'')^2} = 13.43 \text{ ksi}$

$\frac{f_a}{F_a} = \frac{13.43}{19.09} = 0.70 < 1.0$ ok

Minimum weld required:

$\frac{(0.73^k)(0.707)\times 2}{(0.707)(t)\times\pi(\frac{5}{16}'')} = 21 \text{ ksi}$

$\Longrightarrow t > 0.071'' < \frac{1}{8}''$ min. weld

Determine psf max. allowable due to compressive stress in top chord:

$\left.\begin{array}{l} \frac{21.12 \text{ ksi}}{16.7 \text{ ksi}} \times 125 \text{ psf} = 158 \text{ psf} \\ \frac{16.46 \text{ ksi}}{14.16 \text{ ksi}} \times 125 \text{ psf} = 145.3 \text{ psf} \end{array}\right\}$ say 145 psf

1

4.3 Pipe Legs Analysis

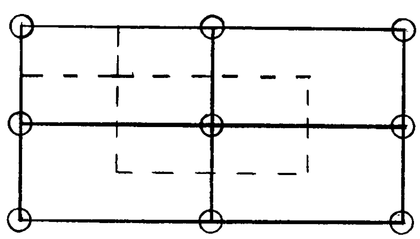
From STAAD, the maximum support reaction is:

$R_y = 1.04$ kips

Legs are $1\frac{1}{2}''$-ø Sch. 40 steel pipe, and their length can vary from 8" to 12'-0".

1

There is only one leg at a corner where deck frames meet. A corner leg will support the STAAD support reactions. An interior leg, however, will support the vertical support reactions of four deck frames — interior leg controls.



Compression only:

$P_{max} = 1.04^k \times 4 = 4.16$ kips (from STAAD)

$f_a = \frac{P_{max}}{A} = \frac{4.16^k}{0.799 in^2} = 5.21$ ksi

For $L = 12'$-$0''$, $\frac{KL}{r} = \frac{(1.0)(12' \times 12)}{0.623''} = 231.1 > 200$

$C_c = \sqrt{\frac{2\pi^2 E}{F_y}} = \sqrt{\frac{2\pi^2(29000 \text{ ksi})}{35 \text{ ksi}}} = 127.9 < \frac{KL}{r}$

1

$$\Longrightarrow F_a = \frac{12\pi^2 E}{23(KL/r)^2}$$

$$= \frac{12\pi^2(29000\text{ ksi})}{23(231.1)^2}$$

$$= 2.80\text{ ksi} < 5.21\text{ ksi} \qquad \text{N.G.}$$

Determine maximum allowable unbraced length

$$5.21\text{ ksi} = \frac{12\pi^2(29000\text{ ksi})}{23(KL/r)^2}$$

$$\frac{KL}{r} \leq \sqrt{\frac{12\pi^2(29000\text{ ksi})}{23\times 5.21\text{ ksi}}} = 169.3$$

$$L_{max} = \frac{169.3\times 0.623''}{1.0} = 105.5'' = 8'\text{-}9\tfrac{1}{2}''$$

$\Longrightarrow$ Maximum unbraced (both directions) length of pipe leg in pure compression is $\underline{\underline{8'\text{-}0''}}$.

Compression and Bending due to lateral seismic loads:

$P_{max} = 4.16$ kips

Seismic force $= 0.1\times DL$

$DL = [6.303\text{ plf}\times(8'+4') + 4.116\text{ plf}\times(8'\times 2 + 4'\times 2)/4\text{ sup.}]\times 4\text{ frames}$

$= 174.42\text{ lb.} = 0.1744\text{ kips}$

Seismic force $= 0.1\times 0.1744\text{ kips} = 0.0174\text{ kips}$

For $L = 12'\text{-}0''$, $M_{max} = 0.0174\text{ kips}\times 144'' = 2.51\text{ kip-in}$

$$f_b = \frac{M_{max}}{S} = \frac{2.51\text{ kip-in}}{0.326\text{ in}^3} = 7.70\text{ ksi}$$

$$F_b = 0.60\times F_y = 0.60\times 35\text{ ksi} = 21.0\text{ ksi}$$

1

Determine maximum allowable unbraced length under both compression and bending:

$$\frac{f_a}{F_a} + \frac{C_m f_b}{(1 - f_a/F_e')F_b} \le 1.0 \times 1.33$$

seismic factor

$$\frac{f_a}{0.60F_y} + \frac{f_b}{F_b} \le 1.0 \times 1.33$$

$C_m = 0.85$ (sidesway possible)

$$F_e' = \frac{12\pi^2 E}{23(KL/r)^2}$$

From before, maximum unbraced length of pipe leg in pure compression is 8'-0".

Try a 12'-long column with an unbraced length of 6'-0":

$$\frac{KL}{r} = \frac{6' \times 12}{0.623''} = 115.6 < 200$$

$\Longrightarrow F_a \approx 10.89$ ksi (Tables C-36, C-50)

$$F_e' = \frac{12\pi^2(29000 \text{ ksi})}{23(115.6)^2} = 11.17 \text{ ksi}$$

$$\frac{5.21 \text{ ksi}}{10.89 \text{ ksi}} + \frac{0.85(7.70 \text{ ksi})}{(1 - 5.21/11.17)(21.0 \text{ ksi})} = 0.478 + 0.584 = 1.06 < 1.33 \quad \underline{\text{ok}}$$

$$\frac{5.21 \text{ ksi}}{0.60 \times 35 \text{ ksi}} + \frac{7.70 \text{ ksi}}{21.0 \text{ ksi}} = 0.248 + 0.367 = 0.615 < 1.33 \quad \underline{\text{ok}}$$

Try a 12′-long column with an unbraced length of 7′-0″:

$\frac{KL}{r} = \frac{7' \times 12}{0.623''} = 134.8 < 200$

$\Longrightarrow F_a \approx 8.22$ ksi (Tables C-36, C-50)

$F_e' = \frac{12\pi^2(29000 \text{ ksi})}{23(134.8)^2} = 8.22$ ksi

$\frac{5.21 \text{ ksi}}{8.22 \text{ ksi}} + \frac{0.85(7.70 \text{ ksi})}{(1-5.21/8.22)(21.0 \text{ ksi})} = 0.634 + 0.851 = 1.49 > 1.33$ N.G.

$\Longrightarrow$ Maximum unbraced length of pipe leg under both compression and lateral seismic loading is 6′-0″.

Check wind load:

There is very little side area for the wind to blow against, so by inspection, seismic forces will govern lateral loads. However, wind-induced uplift is a concern.

$P = C_e C_q q_s I_w$

$C_e = 1.06$ (Exposure C, <15′)

$C_q = 0.7$ (upward)

$I_w = 1.0$

$q_s = 0.00256V^2$

$P = 1.06 \times 0.7 \times 0.00256V^2 \times 1.0 = 0.0019V^2$ psf

$DL \approx 160$ lb./(8′x4′) = 5 psf

$\Longrightarrow 5 \text{ psf} \geq 1.5 \times 0.0019V^2$

$\Longrightarrow V \leq 41.9$ mph $\Longrightarrow$ say 40 mph

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CALCULATION SHEET

TITLE:	4'X8' STEELDECK® PLATFORM EVALUATION	**DATE:**	1/22/07			**PAGE:**	17		1
SUBJECT:	5.0 CONCLUSIONS	**BY:**	SPB	**CK:**	WDB	**SHT:**	1	**OF**	1

The plywood deck, steel truss frame, and pipe legs are capable of supporting a live load of 125 psf. The steel truss frames are analyzed as a composite section consisting of the plywood deck and the steel truss chords. According to the STAAD output, the maximum moment in any of the steel trusses is 16.75 kip-in, which occurs in the central longitudinal truss at its junction with the central transverse trusses. 1
Compression in the top chord and in the web members was checked and found to be acceptable under the specified live load. The maximum deflection under the 125 psf live load is 0.144", which occurs at the intersection of the center spans. This deflection is approximately half of the UBC recommended maximum deflection (L/745 = ½ x (UBC L/360)).

The 12' long pipe legs are capable of supporting the full live load, but only if lateral bracing is provided. Under pure compression, the maximum allowable unbraced length of pipe leg is 8'-0". Under both compressive and lateral seismic loads, the maximum allowable unbraced length of pipe leg is 6'-0". Lateral seismic loads are 0.1xDL since the structure is temporary. By inspection, lateral seismic loads exceed lateral wind loads. However, uplift due to wind is possible if the wind speed exceeds 40 mph.

1

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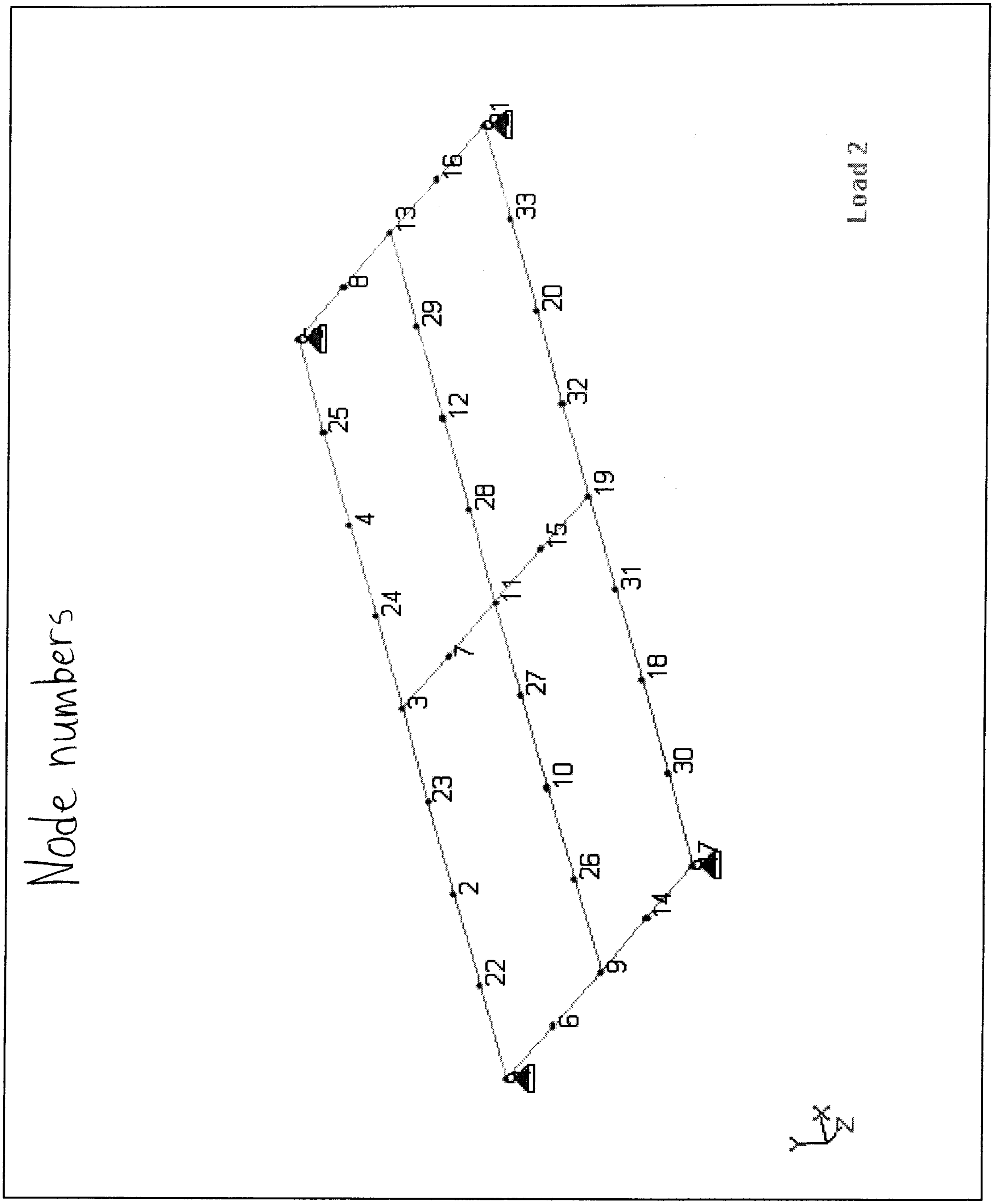
TITLE:	4'x8' STEELDECK® PLATFORM EVALUATION	**DATE:**	1/22/07		**PAGE:**	18		
SUBJECT:	6.0 REFERENCES	**BY:** SPB	**CK:** HDB		**SHT:**	1	**OF**	1

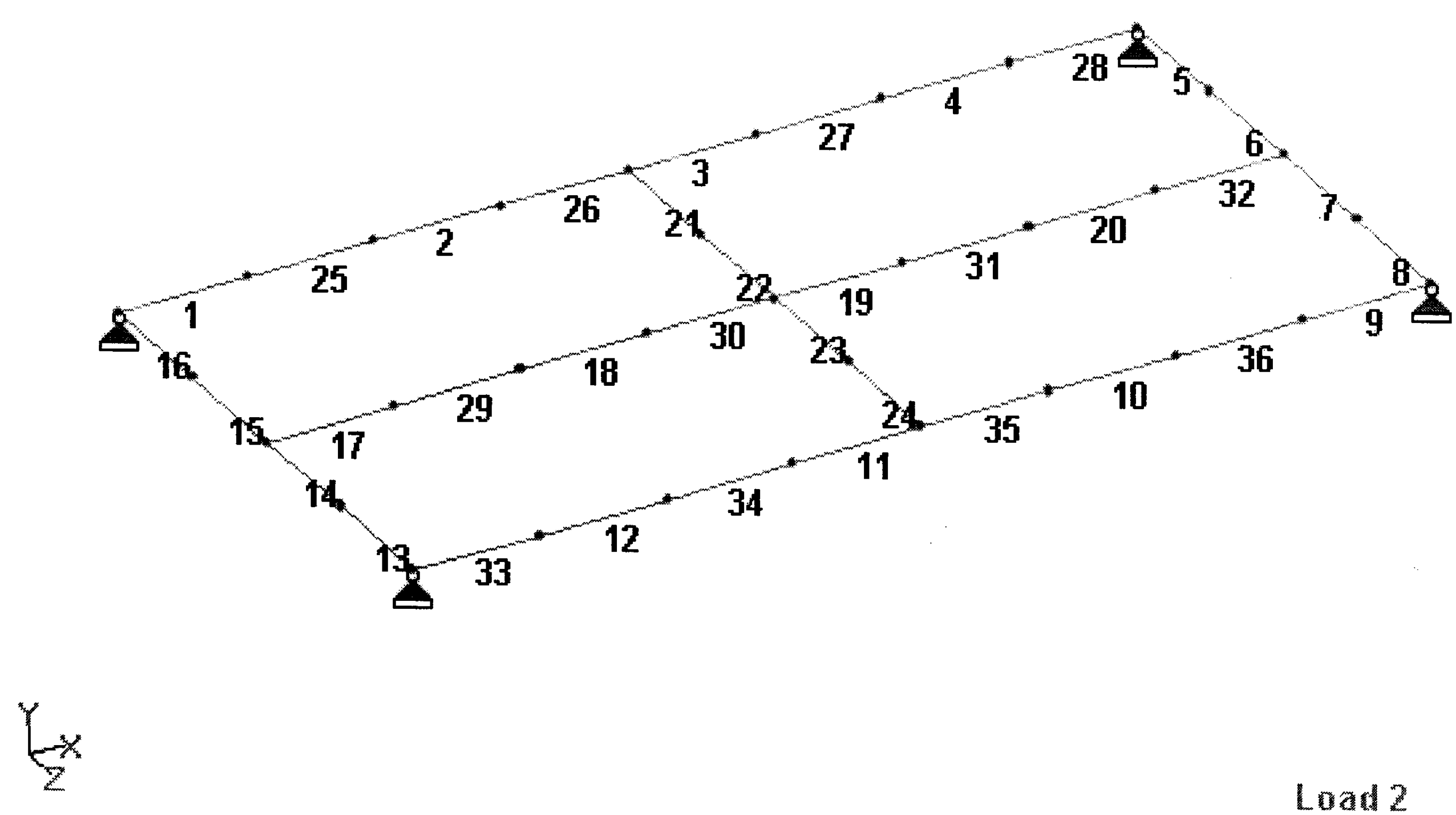
1

1. International Conference of Building Officials. Uniform Building Code, Volume 2 – Structural Engineering Design Provisions. 1997.
2. American Institute of Steel Construction. Manual of Steel Construction – Allowable Stress Design. 9th Edition. 1989.
3. Warren C. Young; Richard G. Budynas. Roark's Formulas for Stress and Strain. Seventh Edition. 2002.
4. Research Engineers, International. STAAD.Pro 2004.
5. Meeting between HEA and Steel Deck on 12/7/06 with sample 4'x8' steel deck platform.
6. Hopper Engineering Associates drawings.
7. Approved American National Standard (ANSI) and American Forest & Paper Association (AF&PA). National Design Specification (NDS) for Wood Construction – Supplement. 2001 Edition.

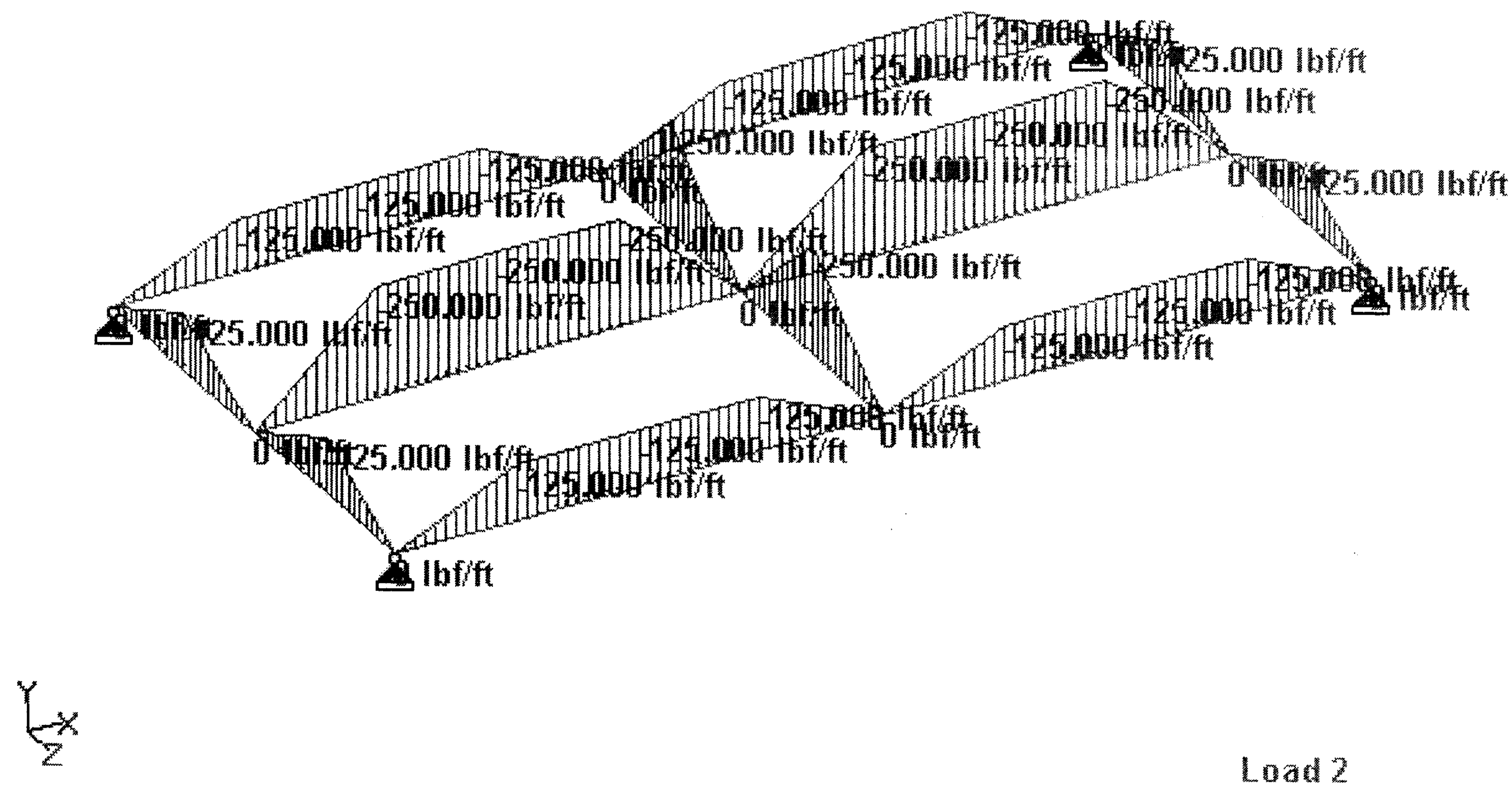
Appendix A – STAAD Output


Node numbers
Load 2
1
16
13
8
33
20
29
32
12
25
19
28
4
15
11
31
24
7
18
27
3
10
30
23
26
2
17
14
22
9
6
X
Y
Z

Member numbers



Live Load Distribution



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****************************************************
*                                                  *
*           STAAD.Pro                              *
*           Version  2006    Bld 1002.US           *
*           Proprietary Program of                 *
*           Research Engineers,  Intl.             *
*           Date=    JAN 22, 2007                  *
*           Time=    16:52:22                      *
*                                                  *
*      USER ID: Hopper Engineering Associates      *
****************************************************
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```
     1. STAAD SPACE
INPUT FILE: SteelDeck.STD
     2. START JOB INFORMATION
     3. ENGINEER DATE 28-DEC-06
     4. END JOB INFORMATION
     5. INPUT WIDTH 79
     6. UNIT INCHES KIP
     7. JOINT COORDINATES
     8. 1 0 0 0; 2 24 0 0; 3 48 0 0; 4 72 0 0; 5 96 0 0; 6 0 0 12; 7 48 0 12
     9. 8 96 0 12; 9 0 0 24; 10 24 0 24; 11 48 0 24; 12 72 0 24; 13 96 0 24; 14 0 0 36
    10. 15 48 0 36; 16 96 0 36; 17 0 0 48; 18 24 0 48; 19 48 0 48; 20 72 0 48
    11. 21 96 0 48; 22 12 0 0; 23 36 0 0; 24 60 0 0; 25 84 0 0; 26 12 0 24; 27 36 0 24
    12. 28 60 0 24; 29 84 0 24; 30 12 0 48; 31 36 0 48; 32 60 0 48; 33 84 0 48
    13. MEMBER INCIDENCES
    14. 1 1 22; 2 2 23; 3 3 24; 4 4 25; 5 5 8; 6 8 13; 7 13 16; 8 16 21; 9 21 33
    15. 10 20 32; 11 19 31; 12 18 30; 13 17 14; 14 14 9; 15 9 6; 16 6 1; 17 9 26
    16. 18 10 27; 19 11 28; 20 12 29; 21 3 7; 22 7 11; 23 11 15; 24 15 19; 25 22 2
    17. 26 23 3; 27 24 4; 28 25 5; 29 26 10; 30 27 11; 31 28 12; 32 29 13; 33 30 17
    18. 34 31 18; 35 32 19; 36 33 20
    19. START USER TABLE
    20. TABLE 1
    21. UNIT INCHES KIP
    22. GENERAL
    23. T-BEAM
    24. 0.9231 6.5 0.135 0.75 0.135 4.845 0.054 1 1.064 0.144 0.7575 0.2025 0 0 0 0
    25. R-BEAM
    26. 0.6456 6.5 0.135 0.75 0.135 3.851 0.041 1 0.994 0.109 0.48 0.2025 0 0 0 0
    27. END
    28. DEFINE MATERIAL START
    29. ISOTROPIC STEEL
    30. E 29000
    31. POISSON 0.3
    32. DENSITY 0.000283
    33. ALPHA 6.5E-006
    34. DAMP 0.03
    35. END DEFINE MATERIAL
    36. MEMBER PROPERTY AMERICAN
    37. 17 TO 24 29 TO 32 UPTABLE 1 T-BEAM
    38. 1 TO 16 25 TO 28 33 TO 36 UPTABLE 1 R-BEAM
    39. CONSTANTS
    40. MATERIAL STEEL ALL
```

```
41. SUPPORTS
42. 1 5 17 21 PINNED
43. UNIT FEET POUND
44. LOAD 1 LOADTYPE DEAD  TITLE SELF WEIGHT
45. MEMBER LOAD
46. 17 TO 24 29 TO 32 UNI GY -6.303
47. 1 TO 16 25 TO 28 33 TO 36 UNI GY -4.116
48. LOAD 2 LOADTYPE LIVE  TITLE LIVE LOAD
49. MEMBER LOAD
50. 2 4 10 12 25 27 34 36 UNI GY -125
51. 1 3 5 7 9 11 13 15 LIN Y 0 -125
52. 6 8 14 16 26 28 33 35 LIN Y -125 0
53. 18 20 29 31 UNI GY -250
54. 17 19 21 23 LIN Y 0 -250
55. 22 24 30 32 LIN Y -250 0
56. LOAD COMBINATION 3
57. 1 1.0 2 1.0
58. UNIT INCHES KIP
59. PERFORM ANALYSIS
```

P R O B L E M   S T A T I S T I C S

```
NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS =     33/     36/      4
ORIGINAL/FINAL BAND-WIDTH=     21/      4/      30 DOF
TOTAL PRIMARY LOAD CASES =     2, TOTAL DEGREES OF FREEDOM =     186
SIZE OF STIFFNESS MATRIX =         6 DOUBLE  KILO-WORDS
REQRD/AVAIL. DISK SPACE  =      12.1/    4930.7 MB
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60. PRINT SUPPORT REACTIONS
```

SUPPORT REACTIONS -UNIT KIP INCH STRUCTURE TYPE = SPACE

JOINT	LOAD	FORCE-X	FORCE-Y	FORCE-Z	MOM-X	MOM-Y	MOM Z
1	1	0.00	0.04	0.00	0.00	0.00	0.00
	2	0.00	1.00	0.00	0.00	0.00	0.00
	3	0.00	1.04	0.00	0.00	0.00	0.00
5	1	0.00	0.04	0.00	0.00	0.00	0.00
	2	0.00	1.00	0.00	0.00	0.00	0.00
	3	0.00	1.04	0.00	0.00	0.00	0.00
17	1	0.00	0.04	0.00	0.00	0.00	0.00
	2	0.00	1.00	0.00	0.00	0.00	0.00
	3	0.00	1.04	0.00	0.00	0.00	0.00
21	1	0.00	0.04	0.00	0.00	0.00	0.00
	2	0.00	1.00	0.00	0.00	0.00	0.00
	3	0.00	1.04	0.00	0.00	0.00	0.00

************** END OF LATEST ANALYSIS RESULT **************

61. LOAD LIST 3
62. PRINT JOINT DISCPLACEMENTS

JOINT DISPLACEMENT (INCH RADIANS)   STRUCTURE TYPE = SPACE

JOINT	LOAD	X-TRANS	Y-TRANS	Z-TRANS	X-ROTAN	Y-ROTAN	Z-ROTAN
1	3	0.00000	0.00000	0.00000	0.00116	0.00000	-0.00435
2	3	0.00000	-0.09598	0.00000	0.00084	0.00000	-0.00307
3	3	0.00000	-0.13619	0.00000	0.00052	0.00000	0.00000
4	3	0.00000	-0.09598	0.00000	0.00084	0.00000	0.00307
5	3	0.00000	0.00000	0.00000	0.00116	0.00000	0.00435
6	3	0.00000	-0.01376	0.00000	0.00086	0.00000	-0.00425
7	3	0.00000	-0.14203	0.00000	0.00035	0.00000	0.00000
8	3	0.00000	-0.01376	0.00000	0.00086	0.00000	0.00425
9	3	0.00000	-0.02027	0.00000	0.00000	0.00000	-0.00414
10	3	0.00000	-0.10984	0.00000	0.00000	0.00000	-0.00278
11	3	0.00000	-0.14433	0.00000	0.00000	0.00000	0.00000
12	3	0.00000	-0.10984	0.00000	0.00000	0.00000	0.00278
13	3	0.00000	-0.02027	0.00000	0.00000	0.00000	0.00414
14	3	0.00000	-0.01376	0.00000	-0.00086	0.00000	-0.00425
15	3	0.00000	-0.14203	0.00000	-0.00035	0.00000	0.00000
16	3	0.00000	-0.01376	0.00000	-0.00086	0.00000	0.00425
17	3	0.00000	0.00000	0.00000	-0.00116	0.00000	-0.00435
18	3	0.00000	-0.09598	0.00000	-0.00084	0.00000	-0.00307
19	3	0.00000	-0.13619	0.00000	-0.00052	0.00000	0.00000
20	3	0.00000	-0.09598	0.00000	-0.00084	0.00000	0.00307
21	3	0.00000	0.00000	0.00000	-0.00116	0.00000	0.00435
22	3	0.00000	-0.05201	0.00000	0.00100	0.00000	-0.00401
23	3	0.00000	-0.12549	0.00000	0.00068	0.00000	-0.00168
24	3	0.00000	-0.12549	0.00000	0.00068	0.00000	0.00168
25	3	0.00000	-0.05201	0.00000	0.00100	0.00000	0.00401
26	3	0.00000	-0.06942	0.00000	0.00000	0.00000	-0.00376
27	3	0.00000	-0.13569	0.00000	0.00000	0.00000	-0.00144
28	3	0.00000	-0.13569	0.00000	0.00000	0.00000	0.00144
29	3	0.00000	-0.06942	0.00000	0.00000	0.00000	0.00376
30	3	0.00000	-0.05201	0.00000	-0.00100	0.00000	-0.00401
31	3	0.00000	-0.12549	0.00000	-0.00068	0.00000	-0.00168
32	3	0.00000	-0.12549	0.00000	-0.00068	0.00000	0.00168
33	3	0.00000	-0.05201	0.00000	-0.00100	0.00000	0.00401

************** END OF LATEST ANALYSIS RESULT **************

63. PRINT MEMBER FORCES

MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KIP INCH (LOCAL)

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
1	3	1	0.00	0.54	0.00	0.15	0.00	0.09
		22	0.00	-0.48	0.00	-0.15	0.00	6.16
2	3	2	0.00	0.35	0.00	0.15	0.00	-11.11
		23	0.00	-0.22	0.00	-0.15	0.00	14.52
3	3	3	0.00	-0.15	0.00	-0.15	0.00	-16.63
		24	0.00	0.22	0.00	0.15	0.00	14.52
4	3	4	0.00	-0.35	0.00	-0.15	0.00	-11.11
		25	0.00	0.48	0.00	0.15	0.00	6.16
5	3	5	0.00	0.50	0.00	0.09	0.00	0.15
		8	0.00	-0.43	0.00	-0.09	0.00	5.57
6	3	8	0.00	0.43	0.00	0.09	0.00	-5.57
		13	0.00	-0.37	0.00	-0.09	0.00	10.24
7	3	13	0.00	-0.37	0.00	-0.09	0.00	-10.24
		16	0.00	0.43	0.00	0.09	0.00	5.57
8	3	16	0.00	-0.43	0.00	-0.09	0.00	-5.57
		21	0.00	0.50	0.00	0.09	0.00	-0.15
9	3	21	0.00	0.54	0.00	0.15	0.00	0.09
		33	0.00	-0.48	0.00	-0.15	0.00	6.16
10	3	20	0.00	0.35	0.00	0.15	0.00	-11.11
		32	0.00	-0.22	0.00	-0.15	0.00	14.52
11	3	19	0.00	-0.15	0.00	-0.15	0.00	-16.63
		31	0.00	0.22	0.00	0.15	0.00	14.52
12	3	18	0.00	-0.35	0.00	-0.15	0.00	-11.11
		30	0.00	0.48	0.00	0.15	0.00	6.16
13	3	17	0.00	0.50	0.00	0.09	0.00	0.15
		14	0.00	-0.43	0.00	-0.09	0.00	5.57
14	3	14	0.00	0.43	0.00	0.09	0.00	-5.57
		9	0.00	-0.37	0.00	-0.09	0.00	10.24
15	3	9	0.00	-0.37	0.00	-0.09	0.00	-10.24
		6	0.00	0.43	0.00	0.09	0.00	5.57

MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KIP INCH (LOCAL)

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
16	3	6	0.00	-0.43	0.00	-0.09	0.00	-5.57
		1	0.00	0.50	0.00	0.09	0.00	-0.15
17	3	9	0.00	0.73	0.00	0.00	0.00	-0.19
		26	0.00	-0.60	0.00	0.00	0.00	8.44
18	3	10	0.00	0.35	0.00	0.00	0.00	-14.12
		27	0.00	-0.09	0.00	0.00	0.00	16.72
19	3	11	0.00	0.04	0.00	0.00	0.00	-16.75
		28	0.00	0.09	0.00	0.00	0.00	16.72
20	3	12	0.00	-0.35	0.00	0.00	0.00	-14.12
		29	0.00	0.60	0.00	0.00	0.00	8.44
21	3	3	0.00	0.31	0.00	0.00	0.00	-0.30
		7	0.00	-0.17	0.00	0.00	0.00	3.42
22	3	7	0.00	0.17	0.00	0.00	0.00	-3.42
		11	0.00	-0.04	0.00	0.00	0.00	4.47
23	3	11	0.00	-0.04	0.00	0.00	0.00	-4.47
		15	0.00	0.17	0.00	0.00	0.00	3.42
24	3	15	0.00	-0.17	0.00	0.00	0.00	-3.42
		19	0.00	0.31	0.00	0.00	0.00	0.30
25	3	22	0.00	0.48	0.00	0.15	0.00	-6.16
		2	0.00	-0.35	0.00	-0.15	0.00	11.11
26	3	23	0.00	0.22	0.00	0.15	0.00	-14.52
		3	0.00	-0.15	0.00	-0.15	0.00	16.63
27	3	24	0.00	-0.22	0.00	-0.15	0.00	-14.52
		4	0.00	0.35	0.00	0.15	0.00	11.11
28	3	25	0.00	-0.48	0.00	-0.15	0.00	-6.16
		5	0.00	0.54	0.00	0.15	0.00	-0.09
29	3	26	0.00	0.60	0.00	0.00	0.00	-8.44
		10	0.00	-0.35	0.00	0.00	0.00	14.12
30	3	27	0.00	0.09	0.00	0.00	0.00	-16.72
		11	0.00	0.04	0.00	0.00	0.00	16.75
31	3	28	0.00	-0.09	0.00	0.00	0.00	-16.72
		12	0.00	0.35	0.00	0.00	0.00	14.12

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MEMBER END FORCES STRUCTURE TYPE = SPACE

ALL UNITS ARE -- KIP INCH (LOCAL)

MEMBER	LOAD	JT	AXIAL	SHEAR-Y	SHEAR-Z	TORSION	MOM-Y	MOM-Z
32	3	29	0.00	-0.60	0.00	0.00	0.00	-8.44
		13	0.00	0.73	0.00	0.00	0.00	0.19
33	3	30	0.00	-0.48	0.00	-0.15	0.00	-6.16
		17	0.00	0.54	0.00	0.15	0.00	-0.09
34	3	31	0.00	-0.22	0.00	-0.15	0.00	-14.52
		18	0.00	0.35	0.00	0.15	0.00	11.11
35	3	32	0.00	0.22	0.00	0.15	0.00	-14.52
		19	0.00	-0.15	0.00	-0.15	0.00	16.63
36	3	33	0.00	0.48	0.00	0.15	0.00	-6.16
		20	0.00	-0.35	0.00	-0.15	0.00	11.11

************** END OF LATEST ANALYSIS RESULT **************

64. PRINT MEMBER STRESSES

MEMBER STRESSES

ALL UNITS ARE KIP /SQ INCH

MEMB	LD	SECT	AXIAL	BEND-Y	BEND-Z	COMBINED	SHEAR-Y	SHEAR-Z
1	3	.0	0.0	0.0	0.1	0.1	1.1	0.0
		1.00	0.0	0.0	6.2	6.2	1.0	0.0
2	3	.0	0.0	0.0	11.2	11.2	0.7	0.0
		1.00	0.0	0.0	14.6	14.6	0.5	0.0
3	3	.0	0.0	0.0	16.7	16.7	0.3	0.0
		1.00	0.0	0.0	14.6	14.6	0.5	0.0
4	3	.0	0.0	0.0	11.2	11.2	0.7	0.0
		1.00	0.0	0.0	6.2	6.2	1.0	0.0
5	3	.0	0.0	0.0	0.1	0.1	1.0	0.0
		1.00	0.0	0.0	5.6	5.6	0.9	0.0
6	3	.0	0.0	0.0	5.6	5.6	0.9	0.0
		1.00	0.0	0.0	10.3	10.3	0.8	0.0
7	3	.0	0.0	0.0	10.3	10.3	0.8	0.0
		1.00	0.0	0.0	5.6	5.6	0.9	0.0
8	3	.0	0.0	0.0	5.6	5.6	0.9	0.0
		1.00	0.0	0.0	0.1	0.1	1.0	0.0
9	3	.0	0.0	0.0	0.1	0.1	1.1	0.0
		1.00	0.0	0.0	6.2	6.2	1.0	0.0
10	3	.0	0.0	0.0	11.2	11.2	0.7	0.0
		1.00	0.0	0.0	14.6	14.6	0.5	0.0
11	3	.0	0.0	0.0	16.7	16.7	0.3	0.0
		1.00	0.0	0.0	14.6	14.6	0.5	0.0
12	3	.0	0.0	0.0	11.2	11.2	0.7	0.0
		1.00	0.0	0.0	6.2	6.2	1.0	0.0
13	3	.0	0.0	0.0	0.1	0.1	1.0	0.0
		1.00	0.0	0.0	5.6	5.6	0.9	0.0
14	3	.0	0.0	0.0	5.6	5.6	0.9	0.0
		1.00	0.0	0.0	10.3	10.3	0.8	0.0
15	3	.0	0.0	0.0	10.3	10.3	0.8	0.0
		1.00	0.0	0.0	5.6	5.6	0.9	0.0
16	3	.0	0.0	0.0	5.6	5.6	0.9	0.0
		1.00	0.0	0.0	0.1	0.1	1.0	0.0
17	3	.0	0.0	0.0	0.2	0.2	1.0	0.0
		1.00	0.0	0.0	7.9	7.9	0.8	0.0

MEMBER STRESSES

ALL UNITS ARE KIP /SQ INCH

MEMB	LD	SECT	AXIAL	BEND-Y	BEND-Z	COMBINED	SHEAR-Y	SHEAR-Z
18	3	.0	0.0	0.0	13.3	13.3	0.5	0.0
		1.00	0.0	0.0	15.7	15.7	0.1	0.0
19	3	.0	0.0	0.0	15.7	15.7	0.1	0.0
		1.00	0.0	0.0	15.7	15.7	0.1	0.0
20	3	.0	0.0	0.0	13.3	13.3	0.5	0.0
		1.00	0.0	0.0	7.9	7.9	0.8	0.0
21	3	.0	0.0	0.0	0.3	0.3	0.4	0.0
		1.00	0.0	0.0	3.2	3.2	0.2	0.0
22	3	.0	0.0	0.0	3.2	3.2	0.2	0.0
		1.00	0.0	0.0	4.2	4.2	0.1	0.0
23	3	.0	0.0	0.0	4.2	4.2	0.1	0.0
		1.00	0.0	0.0	3.2	3.2	0.2	0.0
24	3	.0	0.0	0.0	3.2	3.2	0.2	0.0
		1.00	0.0	0.0	0.3	0.3	0.4	0.0
25	3	.0	0.0	0.0	6.2	6.2	1.0	0.0
		1.00	0.0	0.0	11.2	11.2	0.7	0.0
26	3	.0	0.0	0.0	14.6	14.6	0.5	0.0
		1.00	0.0	0.0	16.7	16.7	0.3	0.0
27	3	.0	0.0	0.0	14.6	14.6	0.5	0.0
		1.00	0.0	0.0	11.2	11.2	0.7	0.0
28	3	.0	0.0	0.0	6.2	6.2	1.0	0.0
		1.00	0.0	0.0	0.1	0.1	1.1	0.0
29	3	.0	0.0	0.0	7.9	7.9	0.8	0.0
		1.00	0.0	0.0	13.3	13.3	0.5	0.0
30	3	.0	0.0	0.0	15.7	15.7	0.1	0.0
		1.00	0.0	0.0	15.7	15.7	0.1	0.0
31	3	.0	0.0	0.0	15.7	15.7	0.1	0.0
		1.00	0.0	0.0	13.3	13.3	0.5	0.0
32	3	.0	0.0	0.0	7.9	7.9	0.8	0.0
		1.00	0.0	0.0	0.2	0.2	1.0	0.0
33	3	.0	0.0	0.0	6.2	6.2	1.0	0.0
		1.00	0.0	0.0	0.1	0.1	1.1	0.0
34	3	.0	0.0	0.0	14.6	14.6	0.5	0.0
		1.00	0.0	0.0	11.2	11.2	0.7	0.0

MEMBER STRESSES

ALL UNITS ARE KIP /SQ INCH

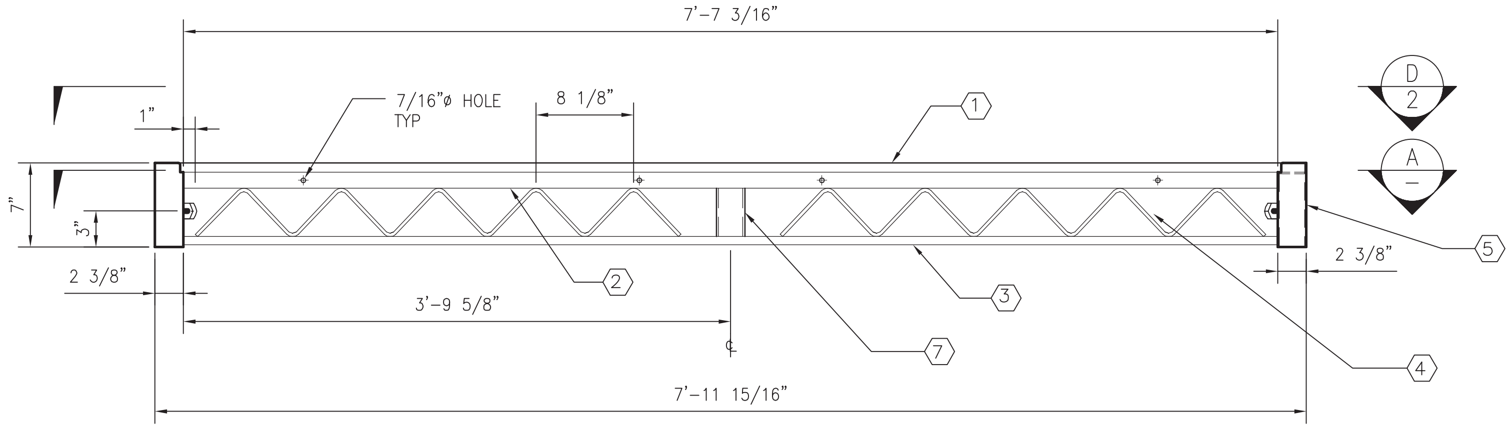
MEMB	LD	SECT	AXIAL	BEND-Y	BEND-Z	COMBINED	SHEAR-Y	SHEAR-Z
35	3	.0	0.0	0.0	14.6	14.6	0.5	0.0
		1.00	0.0	0.0	16.7	16.7	0.3	0.0
36	3	.0	0.0	0.0	6.2	6.2	1.0	0.0
		1.00	0.0	0.0	11.2	11.2	0.7	0.0

************** END OF LATEST ANALYSIS RESULT **************

65. FINISH

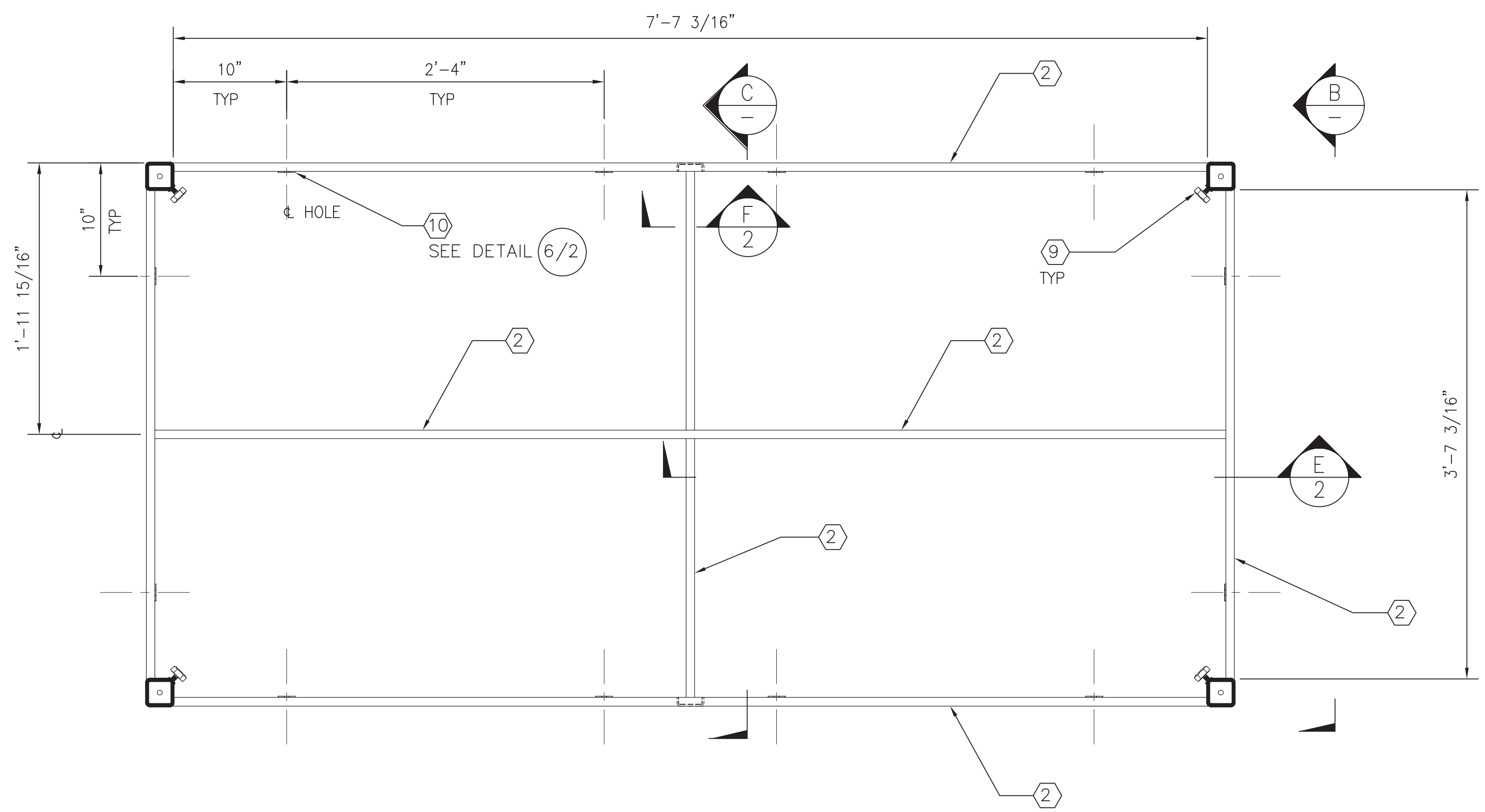
*********** END OF THE STAAD.Pro RUN ***********

**** DATE= JAN 22,2007 TIME= 16:52:22 ****

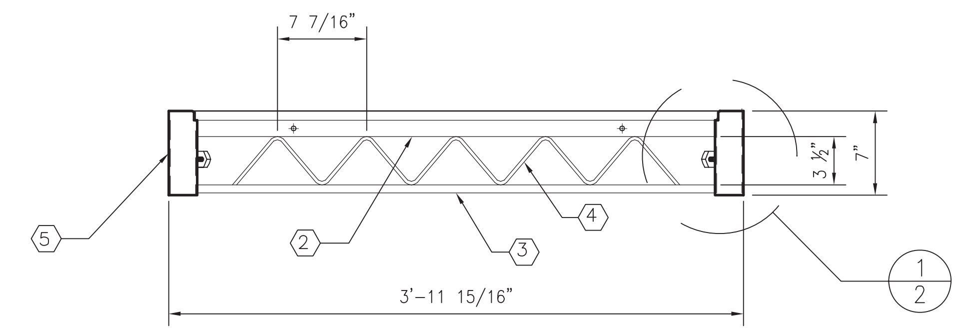


ELEVATION
3/4"=1'-0"

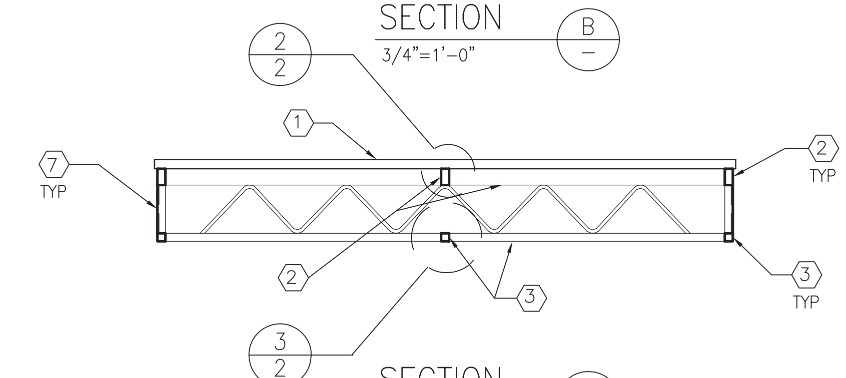
LEGEND	
DESIG	DESCRIPTION
1	3/4" VENEER CORE PHENOLIC GLUE ABB MDO PLYWOOD
2	3/4"x 1 1/2"x 16 GA ASTM A-513 STEEL TUBE
3	3/4"x 3/4"x 14 GA ASTM A-513 STEEL TUBE
4	5/16"ø A-36 STEEL ROD
5	2 3/8"x 2 3/8"x 0.158" THK x 7" LONG ASTM A-513 STEEL TUBE
6	#14x 1 1/2" FLATHEAD SHEET METAL SCREWS
7	2 3/8"x 3/4"x 16 GA COLD FORMED CHANNEL
8	2.05"x 2.05"x 11 GA PL ASTM A-36
9	60 MM HAND KNOB , M12x 1.75x 40 MM
10	1 1/2" SQRx 13 GA PL



SECTION
3/4"=1'-0"
A



SECTION
3/4"=1'-0"
B



SECTION
3/4"=1'-0"
C

SteelDeck® - Drawing I

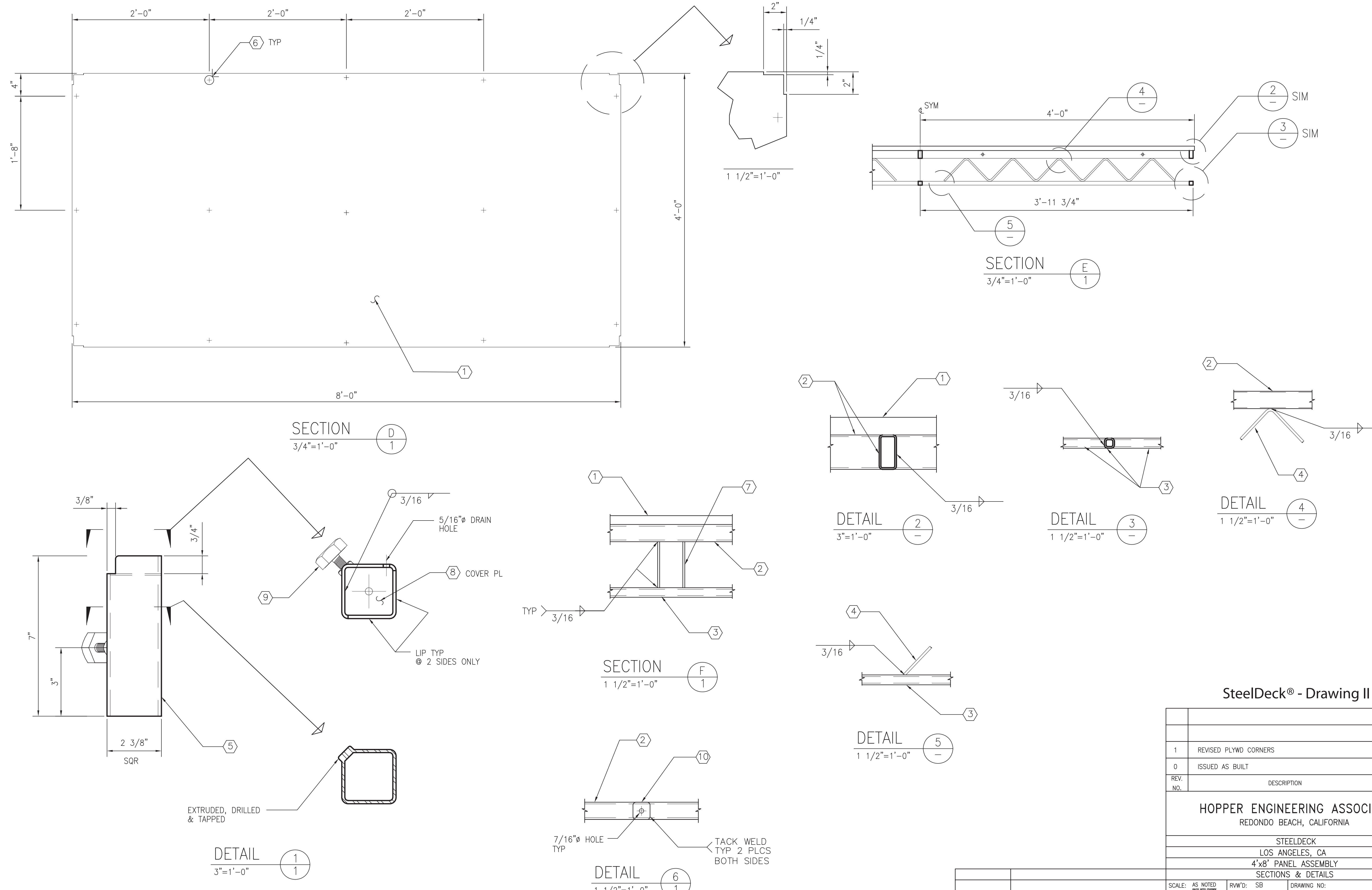
REV. NO.	DESCRIPTION	BY DATE	CHK'D APPRV
0	ISSUED AS BUILT	DD 4/17/07	WB

HOPPER ENGINEERING ASSOCIATES
REDONDO BEACH, CALIFORNIA

STEELDECK
LOS ANGELES, CA
4'x8' PANEL ASSEMBLY
ELEVATION, SECTIONS & LEGEND

DWG. NO.	REFERENCE DRAWINGS

SCALE: AS NOTED	RVW'D: SB	DRAWING NO:	SHEET NO.	REV.
DATE: 04/17/07		HEAMSI06103	1/2	0
DR. DD	JOB NO: PR0196			



SteelDeck® - Drawing II

REV. NO.	DESCRIPTION	BY DATE	CHK'D APPRV
1	REVISED PLYWD CORNERS	DD 4/30/07	WB
0	ISSUED AS BUILT	DD 4/17/07	WB

HOPPER ENGINEERING ASSOCIATES
REDONDO BEACH, CALIFORNIA

STEELDECK
LOS ANGELES, CA
4'x8' PANEL ASSEMBLY
SECTIONS & DETAILS

DWG. NO.	REFERENCE DRAWINGS

SCALE: AS NOTED	RVW'D: SB	DRAWING NO:	SHEET NO.	REV.
DATE: 04/17/07		HEAMSI06103	2/2	1
DR. DD	JOB NO: PR0196			