MEGA-STAGE MK IV

DESIGN BRIEF

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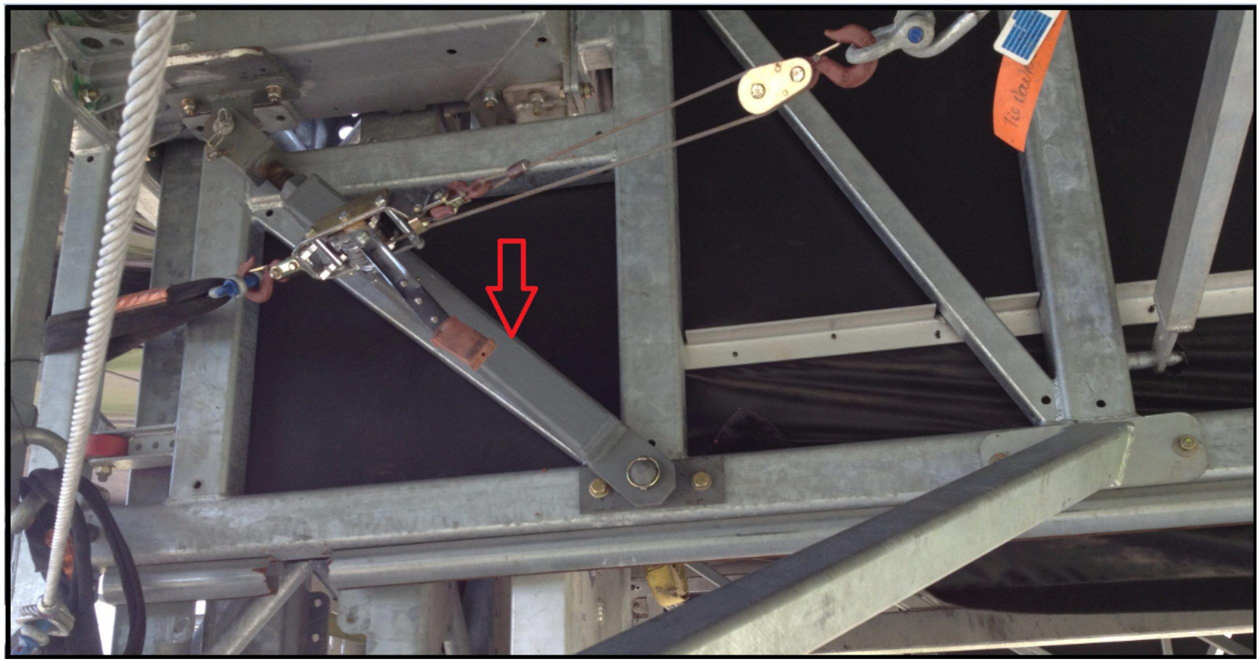
1. Introduction

This document is a brief description of the Mega Stage MK IV (double bay) model structural analysis and its results. The aim of the analysis is to verify the stability and the resistance of the structure under combinations of applied loads.

Mega Stage MK IV is an upgrade of Mega Stage MK III that was done in order to increase the maximum applied wind speed from 80 km/Hr (50 mph) to 120 km/Hr (75 mph) three seconds gust.

Mega Stage MK IV is a steel and aluminium structure that contains the following components:

- A roof that consists of sixteen secondary aluminium trusses and seven secondary steel trusses supported by two main steel trusses. The depth of a secondary truss is variable from 4 ft at the front of the stage to 3 ft at the rear of the stage and is bolted to vertical members of steel trusses; aluminium and steel secondary trusses are 52 ft 9 inches long approximately and they are braced together with aluminium bracings, **additional diagonal steel reinforcing braces are added to both ends of each of the two secondary steel trusses linking the main stage sleeves**.



The height of the front main steel truss is 6 ft 4 inches. The height of the rear main steel truss is 7 ft 4 inches. Main steel trusses are 141 ft long approximately (total length) and the middle span is 77’ long and has **one 32 ft span from each side (the bays)**. Aluminium I beams that can slides inside the top members of the aluminium and steel secondary trusses are also used as cantilevers to extend the roof 10 ft, these aluminium beams are braced to the bottom of the secondary beam.

- **Eight steel masts, Internal steel masts are braced with 5/8 inch steel cables above the stage deck level;**

External steel masts (bays) are braced with ½ inch cables. Internal steel masts are 22 inches x 22 inches 3D steel trusses of 5 ft or 10 ft sections bolted together with four 1-1/8 inches diameter bolts (grade 8), external steel masts (outside corners of the bays) are 20 inches x 20 inches 3D steel trusses of 5 ft or 10 ft sections bolted together with four 1-1/8 inches diameter bolts (grade 8), internal masts are 45 ft high above the deck level, external masts are 45 ft high above the ground. Two stabilizers per each one of the masts are installed under the stage deck to both sides; Masts are also bolted to the deck contour beam directly below the corners of the stage deck. The bottoms of the four central main steel masts are linked together with horizontal steel HSS 10x10x5/16 (or 12x12 Fraco steel masts) in the perpendicular direction of the stage facade.

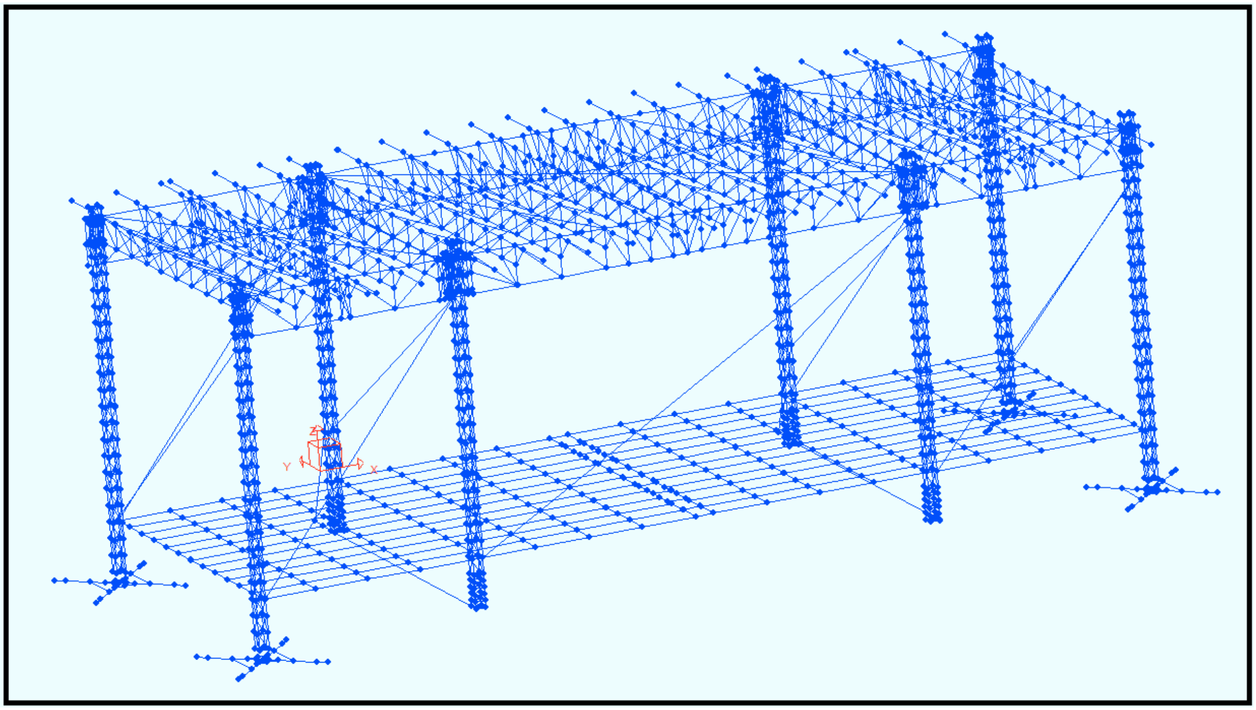
At the bottom of each column (only 4 central columns), a special steel base is installed; this base is supported by jacks and wood cribbing, and is connected to the column by a special bracket at the bottom of the column from one side and with a ½ inch cable from the other side to the column at 15 ft height from ground level, these bases are destined to support a 43200 lb (each) counterweights in order to stabilize the structure.

External columns (bays corners) are bolted to Mega-stage free-standing bases with 24000 lb counterweight on each base (6000 lb per jack).

- **Eight steel sliding frames (sleeves)** surround the internal and external steel masts tightly and transfer the loads from the roof to the masts; these sliding frames can slide over the whole length of masts in order to lift the roof to the top after the installation of the roof trusses is done. The sliding frame is bolted to the main steel truss of the roof and could be pinned to the top of the mast to prevent any vertical movement in both directions (up and down).

- A Stage Deck consisting mainly of aluminium tubes supporting a plywood deck, a flat bed trailer truck makes part of the stage deck structure, the trailer is destined to transport the whole stage structure when dismantled to parts, when the stage structure is installed the trailer act as the central beams supporting the deck and as a counterweight to prevent the sliding of the structure. Aluminum rectangular tubes transfer the loads to main beams (parallel to the trailer long side), shoring jacks supporting the main beams transfer the load to ground below the deck. The deck is at 5 ft to 8 ft from ground level.

Solid fabrics will be covering the entire roof area as a weather protection of the stage and installed equipments. A solid tarp fabric will be covering left and right sides of the stage, wire mesh fabric with 40% porosity cover the rear side of the stage. All the vertical fabric covering the walls of the stage will be secured to the stage structure in order to distribute the wind load evenly at the edges. Even if the structure is able to resist a 120 km/Hr wind speed while the fabric walls are still installed, it is recommended to detach these fabric walls from the structure before having a gusting wind speed of 80 km/Hr (50 mph) since the manipulation of the walls becomes very difficult if wind speed is higher than this limit.
The rear wire mesh fabric wall should be attached to the roof structure at the secondary trusses levels (never below).



The model of the structure was created and analysed using S-Frame software, the verification was done based on S16-01 for steel and CAN3-S157-M83 for Aluminium.

Important modifications (upgrades) to the structure are written in bold in the above structural components listing.

2. Materials

Steel 44 W for plates and iron angles- Fy=44 ksi

Steel A500-C for structural tubes – Fy = 50 Ksi

Aluminium 6061-T6, Welded – Fy = 16 Ksi

Aluminium 6061-T6, Not welded – Fy = 35 ksi

3. Load Cases

3.1 Dead Load

Dead load is the own weight of the structure and is represented by the load case 1- SW (approximately 150 000 lb total weight)

3.2 Wind Load

Wind load is calculated based on a 120 km/Hr (75 mph) and 160 km/Hr (100 mph) three seconds gust measured on site.

For 120 km/Hr, since the wind speed is measured on site directly and it is not a calculation based on the hourly average wind speed measured at a 10 m height, we can consider that q Ce Cg = 15 psf.

Based on figures I-8 and I-9 from the NBC2005 structural commentaries-part 4 of division B – Low-rise buildings, |CgCp| = 2 maximum for walls and 1.5 maximum for roof (the area is higher than 100 m^2). By taking Cg = 2, Cp = 1 for wind on the walls and Cp = -0.75 for the roof. Pressure resulting from this wind speed is P = 15 psf on the solid walls, perpendicular to the vertical faces of the stage, and an uplift of 11.25 psf perpendicular to the roof of the stage.

Pressure on the rear side is used as 12 psf (20% reduction) since mesh's void is 40 %.

An hourly average wind speed of 80 km/Hr is equivalent to 120 km/Hr three second gust based on Durst Curve (ASCE7-05 Figure C6-4), these values (80 km/Hr hourly average or 120 km/Hr gust) could be used to monitor the wind speed in the weather news to detach the fabric walls if higher winds are expected.

Fabrics transfer the wind loads in all the directions (Figure 1)

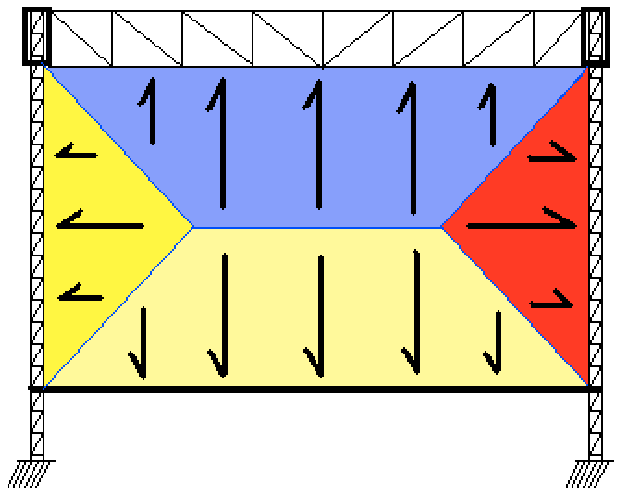


Figure 1

By applying the same calculations for 160 km/Hr, P = 26.7 psf for horizontal surfaces and 20 psf uplift for the roof.

Wind loads were represented in two load cases:

- Load case 2 – Wind Y: wind load perpendicular to the rear side of the stage (15 psf pushing the stage toward its front side).
- Load case 3 – Wind Y Uplift: 11.25 psf uplift wind perpendicular to the roof distributed over the complete area of the roof.
- Load case 4 – Wind X: wind load perpendicular to the left side of the stage (15 psf pushing the stage toward its right side).
- Load case 5 – Wind X Uplift: 11.25 psf uplift wind perpendicular to the roof covering its left half.

3.3 Live Load

Equipment and loads are attached to the lower nodes of trusses. Two Live load cases were created, the total live load for both of them is 228 000 lb distributed over the roof structure by two different ways:

- Load case 6 - Main trusses: concentrated loads are distributed uniformly under the main steel trusses. For the middle span the distributed load is 50 000 lb, for side spans (Bays) 32 000 lb concentrated loads are distributed uniformly.
- Load case 7 – Secondary steel trusses: concentrated loads are distributed uniformly under the secondary aluminium and steel trusses. 10 000 lb uniformly distributed under each aluminium secondary truss and 20 000 lb uniformly distributed under each steel secondary truss.

4. Load Combinations

The structural model was verified for the following load combinations:

- Combination 1 – Stability Y: 1.0 x (LC 1) + 1.4 x (LC2) + 1.4 x (LC3)
- Combination 2 – Stability X: 1.0 x (LC1) + 1.4 x (LC4) + 1.4 x (LC5)
- Combination 3 – Resistance Y 1: 2.0 x (LC 1) + 1.5 x (LC6) + 1.4 x (LC2)
- Combination 4 – Resistance Y 2: 2.0 x (LC1) + 1.5 x (LC7) + 1.4 x (LC2)
- Combination 5 – Resistance X 1: 2.0 x (LC 1) + 1.5 x (LC6) + 1.4 x (LC4)
- Combination 6 – Resistance X 2: 2.0 x (LC 1) + 1.5 x (LC7) + 1.4 x (LC4)
- Combination 7 – Deflection Y: 1.0 x (LC 1) + 1.0 x (LC2) + 1.0 x (LC3)
- Combination 8 – Deflection X: 1.0 x (LC1) + 1.0 x (LC4) + 1.0 x (LC5)
- Combination 9 – Stability Y - without walls: 1.0 x (LC 1) + 1.25 x 0.3 x 1.78 x (LC2) + 1.25 x 1.78 x (LC3) where 0.3 represents the reduction factor of vertical solid surfaces when walls are removed, and 1.78 represents the increase of pressure if wind is 160 km/Hr instead of 120 km/Hr with 1.25 safety factor, these same factors are used also for combination 10.
- Combination 10 – Stability X – without walls: 1.0 x (LC1) + 1.25 x 0.3 x 1.78 x (LC4) + 1.25 x 1.78 x (LC5)

5. Deformation

5.1 Columns

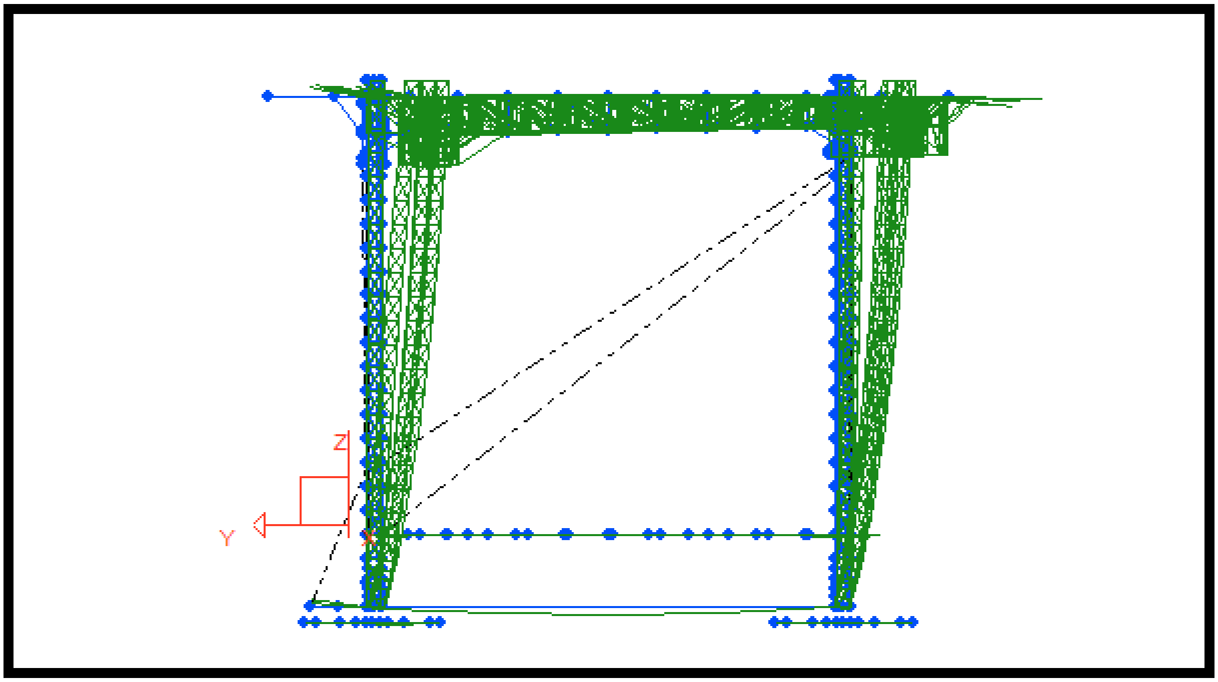


Figure 2

For combination 7 the maximum horizontal displacement of columns is 6’’ in Y direction – see figure 2.

For combination 8 the maximum horizontal displacement of columns is 1.8’’ in X direction – see figure 3 (below).

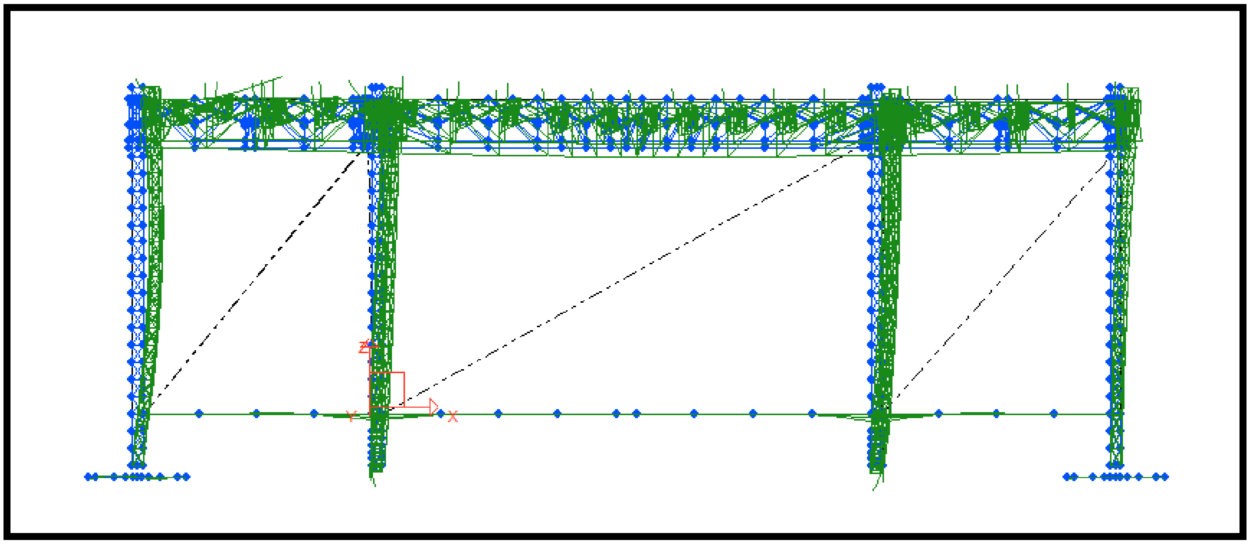


Figure 3

5.2 Secondary trusses

The maximum vertical deflection of the steel secondary truss is 0.89''.

The maximum vertical deflection of the aluminium secondary truss is 1.05'' (figure 4), the length of the truss is 53 ft, L/360= 1.76'' > 1.05'' O.K.

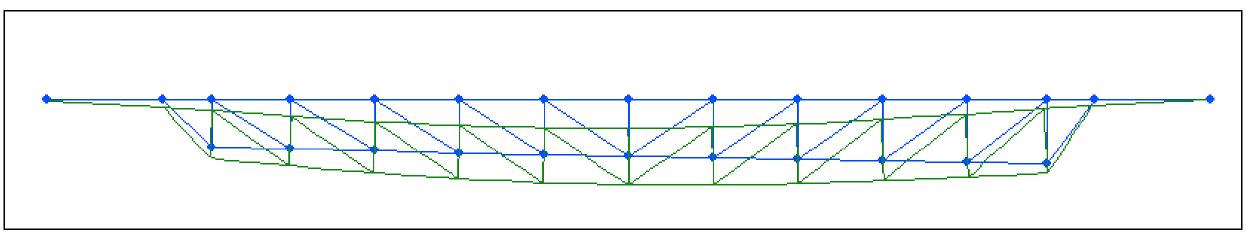


Figure 4

5.3 Main Steel Trusses

The maximum vertical deflection of the steel truss is 1.14'' (figure 5), the length of the steel trusses between the columns is 77 ft, L/360= 2.57'' > 1.14'' O.K.

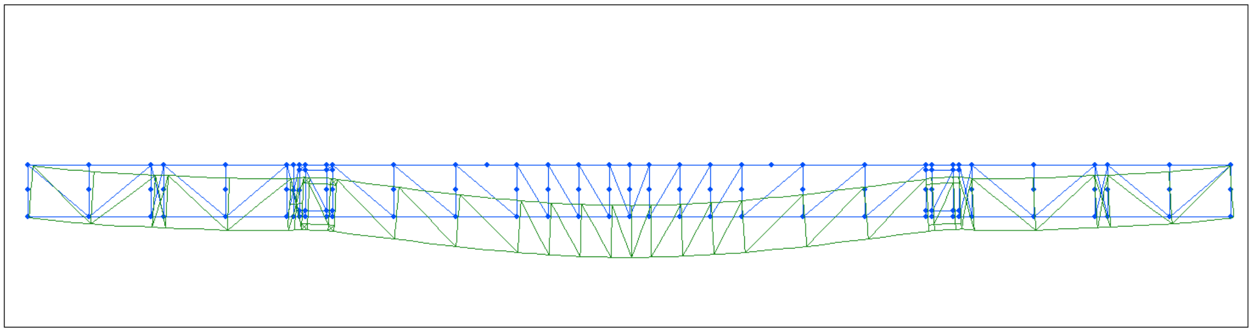


Figure 5

6. Stability

For load combinations 1, 2, 9 and 10 (dead loads with counterweights and factored wind load), all the supports reactions were positive, which means that there was no uplift for any support and the lowest reaction was 5.2 kips for a 120 km/Hr wind. With low reactions values like this case, friction is not enough to resist the sliding of this support, but in our case the bottoms of the columns are linked together in order to transmit the horizontal force to supports having very high compression values which will insure enough friction to resist sliding of the structure.
The total horizontal factored load in Y direction is 141’x53’x12psfx1.4/1000 = 125.5 kips, if we assume a 0.5 friction factor, this means that we will need a minimum of 125.5 kips/0.5 = 251 kips to be able to resist the sliding, the sum of reactions for this load combination is: 256.6 kips.
Maximum factored tension in a cable is 24 kips (safety factor for wind is 1.4), if we increase the safety factor to 2.0 the limit becomes 34 kips. **Cable and turnbuckle resistance should be 34 kips minimum**.

7. Members resistance

The analysis of the steel components was done with S-steel, which is a module of S-Frame that verifies the resistance of steel members.
To verify the resistance of aluminium components, we will use the maximum values of axial forces calculated in S-Frame and compare them to resistance of members.

7.1 Steel masts

The highest stresses occur in columns of the back face of the stage (figure 7).

Figure 8 shows the values of the ratio described in clause 13.8.3-S16-01 (should be less than one) where members with high stresses exist, based on load combinations described above. Calculation details of member having the highest ratios in this column are also shown in the annex in the end of this report.

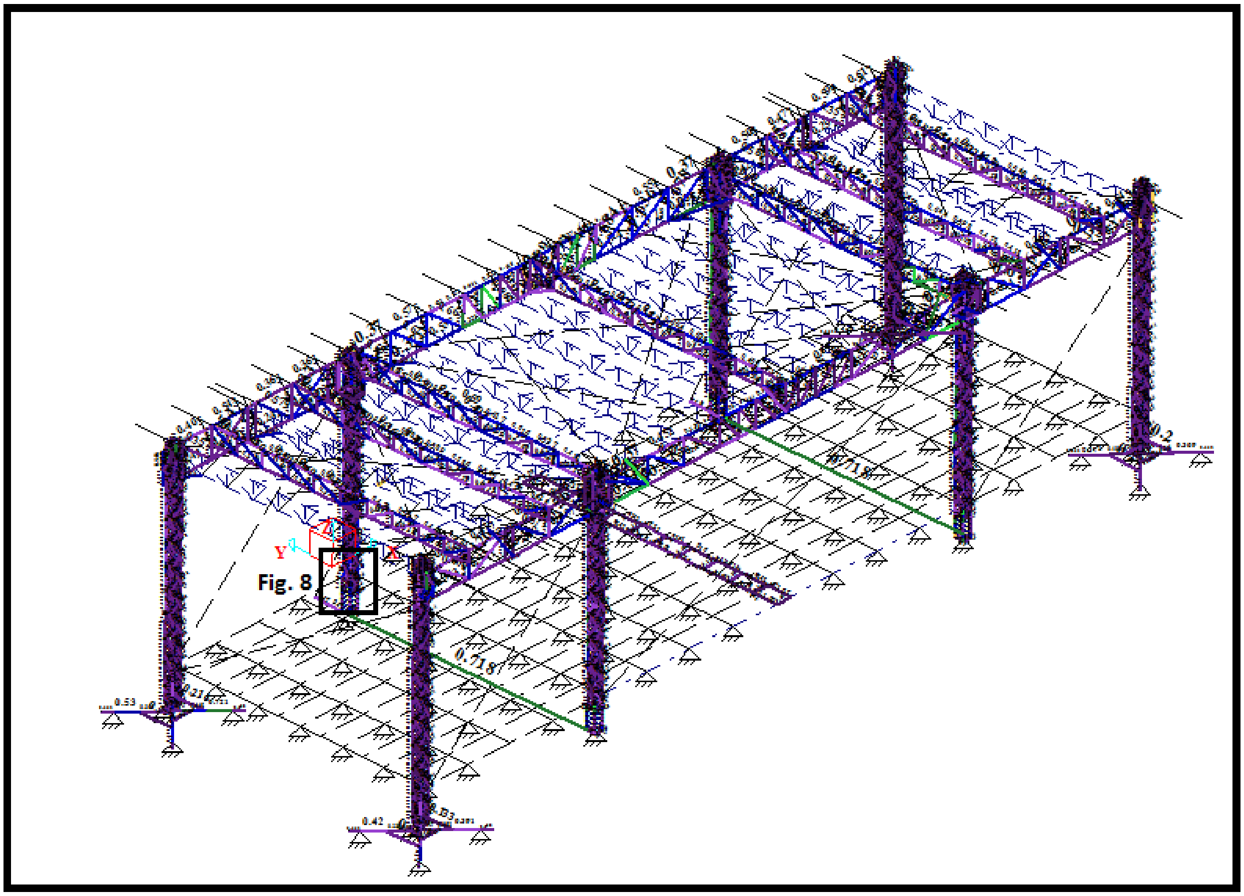


Figure 7

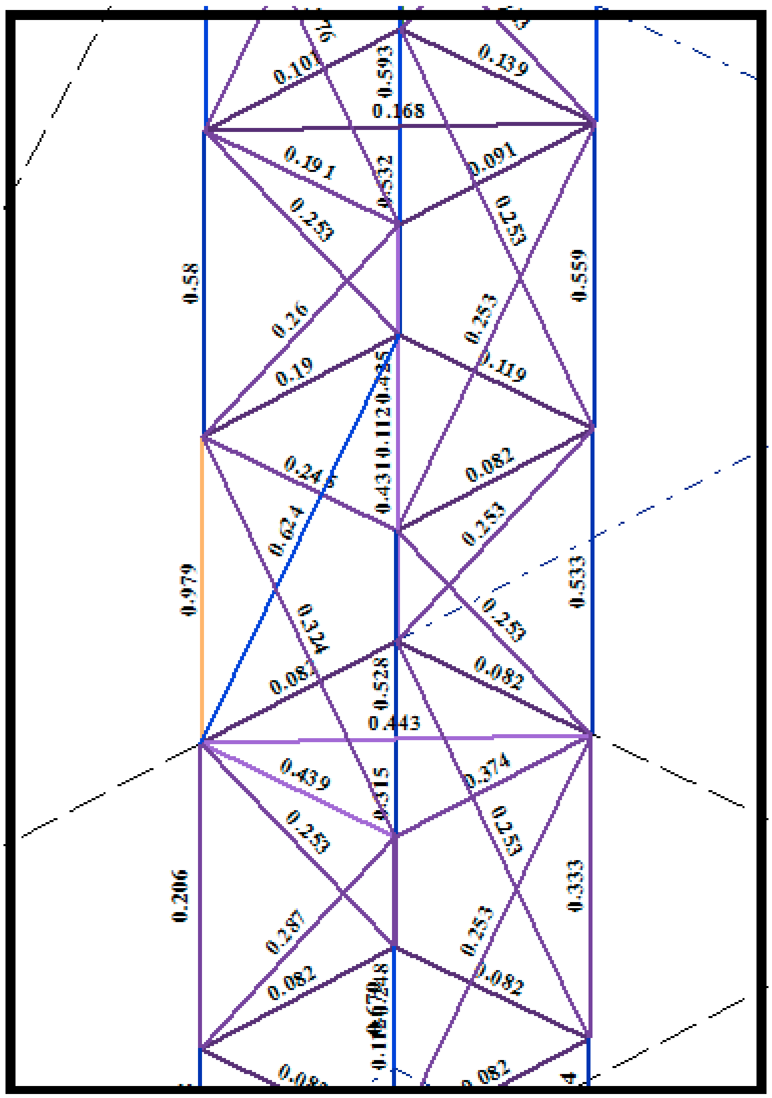


Figure 8

7.2 Main steel Trusses

Figure 9 shows the values of the ratio described in clause 13.8.3-S16-01 (should be less than one) based on load combinations described above, the highest value is 0.998.

Calculation details of member having the highest ratios in this steel truss are also shown in the annex in the end of this report.

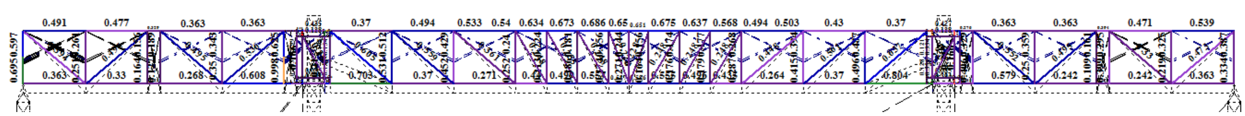


Figure 9

7.3 Secondary steel trusses

Figure 10 shows the values of the ratio described in clause 13.8.3-S16-01 (should be less than one) based on load combinations described above, the highest value is 0.663.

Calculation details of member having the highest ratios in this steel truss are also shown in the annex in the end of this report.

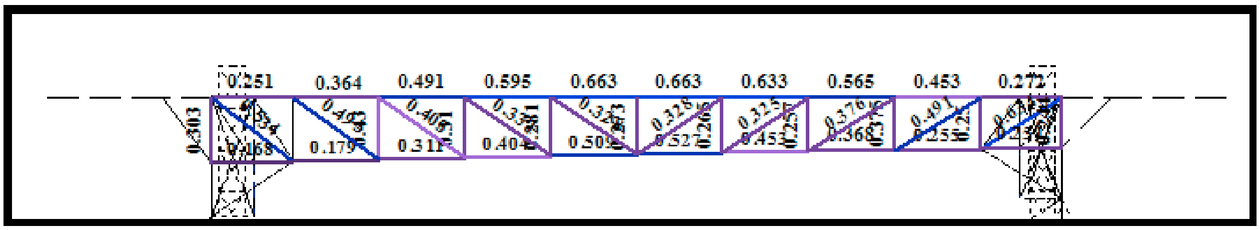


Figure 10

7.4 Secondary aluminium trusses

Horizontal members

Highest axial forces in horizontal members are generated by combination 6 (Figure 11)

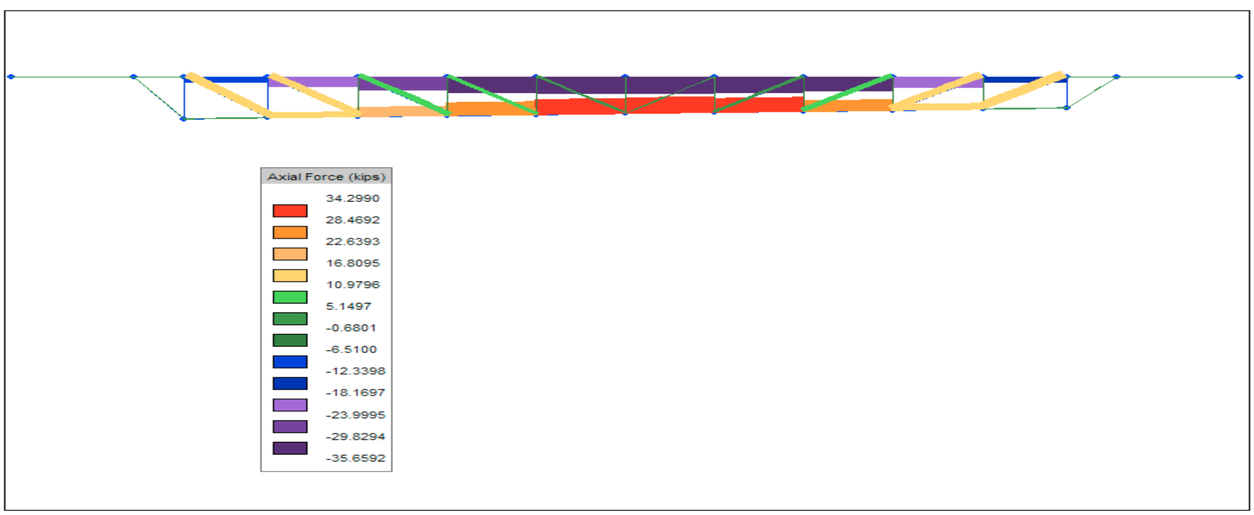


Figure 11

Since the weakest point in the horizontal members is joints due to welding at these places, we will take Cr = Tr = Ø Fy A, and we will use Fy = 16 ksi, Fy in the middle of the member is much higher (Fy = 35 ksi) since the aluminium in the middle is not influenced by welding, this will make Cr at the middle of the member higher than Cr at the edges even with the buckling effect.

For a 4''x3''x1/4'' tube A =3.08 in^2 => Cr = Tr = 0.9 x 16 x 3.08 = 44.35 kips > Cf = 35.66 kips

Vertical and diagonal members

Highest axial forces in vertical and diagonal members are generated by combination 6 (Figure 11)

Verification will be done also at joints with Fy = 16 ksi.

2''x2''x1/8'' tube A =0.9 in^2 => Tr = 0.9 x 16 x 0.9 = 13 kips ~ Tf = 13.09 kips

ANNEX

Detail calculation example of members with high

Internal forces – Main steel truss and column

(Software output)

Ghais Semaan

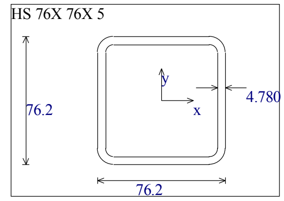
3860 MAUPASSANT
BROSSARD, QC
514-699-3231

Code Details

Project: MKIV
Structure: Main steel truss member
Filename: C:\USERS\GHAISS\DOCUMENTS\BERGER\MKIV\MODEL 5.TEL
Engineer:

Date:21/01/2012

Member: 2342 S-FRAME Section is HS 76X 76X 5
Member is part of group: Section 13
Note: Neglecting: axial<0.0 kips, shear<0.0 kips, moment<0.0 k-ft
Note: Member in braced frame(s).



Load Combination 3 Resistance Y 1 (Bending + Compression)

Section classification (f_y=350 MPa); Section Class = **1**

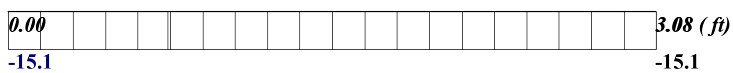
Clause 11

Governing geometrical slenderness ratio

k_x=1.00; k_y=1.00; k_yL/r_y=32.7; $\frac{k_x\,L/r_x}{200} = \frac{33}{200} =$ **0.164**

Clause 10.4.2.1

Axial Load - (kips)

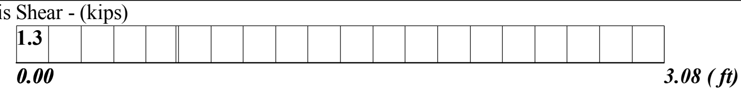


Factored Compressive Resistance Check

n=1.34; λ_x=0.436; Class C; $\frac{C_f}{C_{rx}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\,(324\text{ MPa})} = \frac{15}{86} =$ **0.176**

Clause 13.3.1

Strong Axis Shear - (kips)

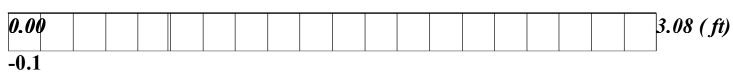


Strong axis shear strength check

A_w= 728 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w\, 0.66F_y} = \frac{1}{34} =$ **0.038**

Clause 13.4.1.1(a)

Weak Axis Shear - (kips)

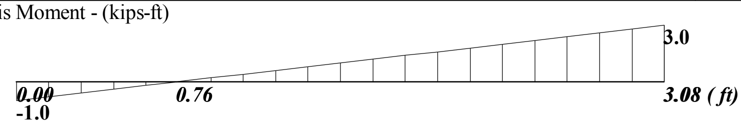


Weak axis shear strength check

A_w= 728 mm^2; $\frac{V_{fx}}{\phi A_w F_s} = \frac{V_{fx}}{\phi A_w\, 0.66F_y} = \frac{0.08}{34.05} =$ **0.002**

Clause 13.4.1.1(a)

Strong Axis Moment - (kips-ft)

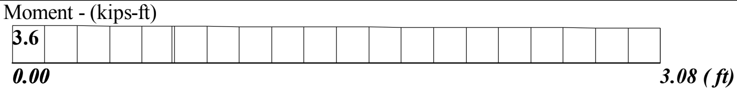


Strong axis section capacity in bending

$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi F_y Z_x} = \frac{2.98}{7.99} =$ **0.372**

Clause 13.5(a)

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending

$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi F_y Z_y} = \frac{3.59}{7.99} =$ **0.450**

Clause 13.5(a)

Axial Compression and Bending cross-sectional Strength Check

ω_{1x}=0.47; ω_{1y}=0.97; U_{1x}=1.00; U_{1y}=1.00; $\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =$ **0.985**

Clause 13.8.3(a)

Design Code: CAN/CSA S16-09
Steel Table : Canadian (CISC)
Analysis Program: S-FRAME (Linear static analysis)

S-STEEL
Version Version 10.00

Axial Compression and Bending overall member Strength Check

ω_{1x}=0.47; ω_{1y}=0.97; U_{1x}=0.48; U_{1y}=1.00; $\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi Z_x F_y} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$ **0.805** — Clause 13.8.3(b)

Axial Compression and Bending lateral torsional buckling strength check

ω_{1x}=0.47; ω_{1y}=0.97; U_{1x}=1.00; U_{1y}=1.00; $\mathbf{\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =}$ **0.998** — **Clause 13.8.3(c)**

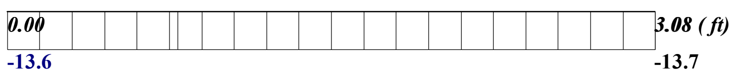
Load Combination 4 Resistance Y 2 (Bending + Compression)

Section classification (f_y=350 MPa); Section Class = **1** — Clause 11

Governing geometrical slenderness ratio — Clause 10.4.2.1

k_x=1.00; k_y=1.00; $k_y L/r_y$=32.7; $\frac{k_x L/r_x}{200} = \frac{33}{200} =$ **0.164**

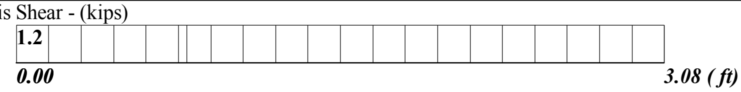
Axial Load - (kips)



Factored Compressive Resistance Check — Clause 13.3.1

n=1.34; λ_x=0.436; Class C; $\frac{C_f}{C_{rx}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A \text{ (324 MPa)}} = \frac{14}{86} =$ **0.159**

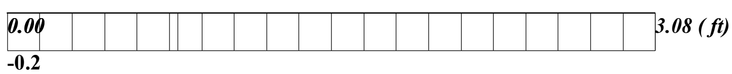
Strong Axis Shear - (kips)



Strong axis shear strength check — Clause 13.4.1.1(a)

A_w= 728 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66 F_y} = \frac{1}{34} =$ **0.035**

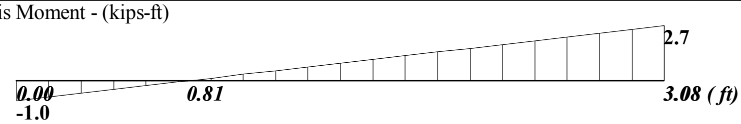
Weak Axis Shear - (kips)



Weak axis shear strength check — Clause 13.4.1.1(a)

A_w= 728 mm^2; $\frac{V_{fx}}{\phi A_w F_s} = \frac{V_{fx}}{\phi A_w 0.66 F_y} = \frac{0.21}{34.05} =$ **0.006**

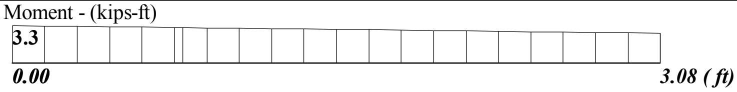
Strong Axis Moment - (kips-ft)



Strong axis section capacity in bending — Clause 13.5(a)

$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi F_y Z_x} = \frac{2.71}{7.99} =$ **0.339**

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending — Clause 13.5(a)

$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi F_y Z_y} = \frac{3.26}{7.99} =$ **0.407**

Axial Compression and Bending cross-sectional Strength Check

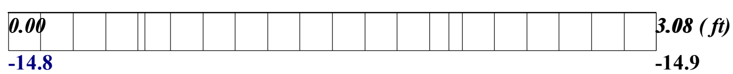
ω_{1x}=0.46; ω_{1y}=0.92; U_{1x}=1.00; U_{1y}=1.00; $\mathbf{\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =}$ **0.894** — **Clause 13.8.3(a)**

Axial Compression and Bending overall member Strength Check ω_{1x}=0.46; ω_{1y}=0.92; U_{1x}=0.47; U_{1y}=0.94;	$\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi Z_x F_y} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$	**0.702**	Clause 13.8.3(b)
Axial Compression and Bending lateral torsional buckling strength check ω_{1x}=0.46; ω_{1y}=0.92; U_{1x}=1.00; U_{1y}=0.94;	$\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$	**0.882**	Clause 13.8.3(c)

Load Combination 5 Resistance X 1 (Bending + Compression)

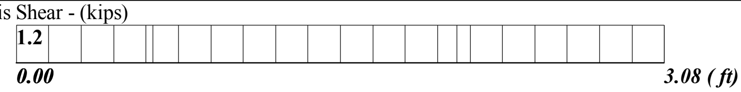
Section classification (f_y=350 MPa);	Section Class =	**1**	Clause 11
Governing geometrical slenderness ratio k_x=1.00; k_y=1.00; k_yL/r_y=32.7;	$\frac{k_x L/r_x}{200} = \frac{33}{200} =$	**0.164**	Clause 10.4.2.1

Axial Load - (kips)



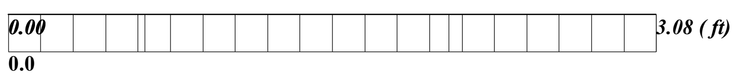
Factored Compressive Resistance Check n=1.34; λ_x=0.436; Class C;	$\frac{C_f}{C_{rx}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A \text{ (324 MPa)}} = \frac{15}{86} =$	**0.173**	Clause 13.3.1

Strong Axis Shear - (kips)



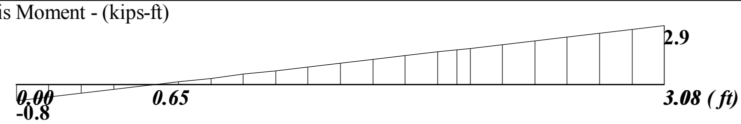
Strong axis shear strength check A_w= 728 mm^2;	$\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66 F_y} = \frac{1}{34} =$	**0.035**	Clause 13.4.1.1(a)

Weak Axis Shear - (kips)



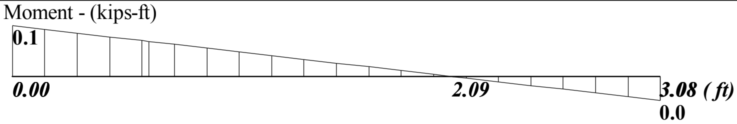
Weak axis shear strength check A_w= 728 mm^2;	$\frac{V_{fx}}{\phi A_w F_s} = \frac{V_{fx}}{\phi A_w 0.66 F_y} = \frac{0.03}{34.05} =$	**0.001**	Clause 13.4.1.1(a)

Strong Axis Moment - (kips-ft)



Strong axis section capacity in bending	$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi F_y Z_x} = \frac{2.86}{7.99} =$	**0.358**	Clause 13.5(a)

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending	$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi F_y Z_y} = \frac{0.06}{7.99} =$	**0.007**	Clause 13.5(a)
Axial Compression and Bending cross-sectional Strength Check ω_{1x}=0.49; ω_{1y}=1.00; U_{1x}=1.00; U_{1y}=1.03;	$\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =$	**0.526**	Clause 13.8.3(a)

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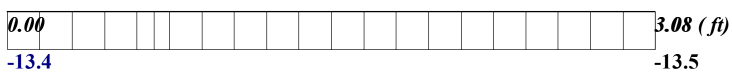
Axial Compression and Bending overall member Strength Check ω_{1x}=0.49; ω_{1y}=1.00; U_{1x}=0.51; U_{1y}=1.03;	$\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi Z_x F_y} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$	0.362	Clause 13.8.3(b)
Axial Compression and Bending lateral torsional buckling strength check **ω_{1x}=0.49; ω_{1y}=1.00; U_{1x}=1.00; U_{1y}=1.03;**	$\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$	**0.539**	**Clause 13.8.3(c)**

Load Combination 6 Resistance X 2 (Bending + Compression)

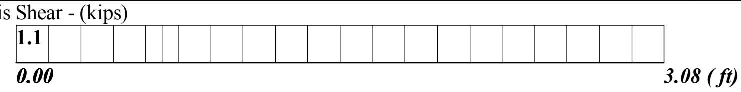
Section classification (f_y=350 MPa);	Section Class =	1	Clause 11
Governing geometrical slenderness ratio k_x=1.00; k_y=1.00; k_yL/r_y=32.7;	$\frac{k_x L/r_x}{200} = \frac{33}{200} =$	0.164	Clause 10.4.2.1

Axial Load - (kips)



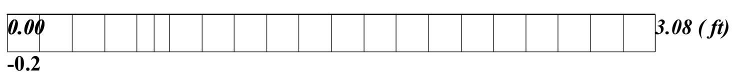
Factored Compressive Resistance Check n=1.34; λ_x=0.436; Class C;	$\frac{C_f}{C_{rx}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A \text{ (324 MPa)}} = \frac{13}{86} =$	0.157	Clause 13.3.1

Strong Axis Shear - (kips)



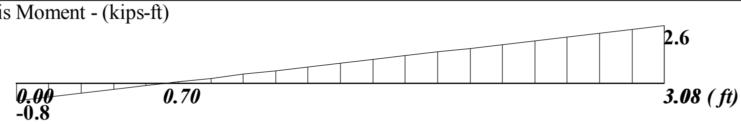
Strong axis shear strength check A_w= 728 mm^2;	$\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66 F_y} = \frac{1}{34} =$	0.032	Clause 13.4.1.1(a)

Weak Axis Shear - (kips)



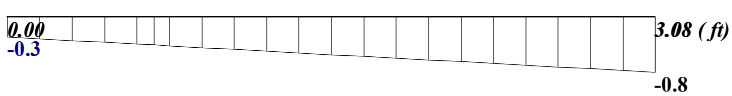
Weak axis shear strength check A_w= 728 mm^2;	$\frac{V_{fx}}{\phi A_w F_s} = \frac{V_{fx}}{\phi A_w 0.66 F_y} = \frac{0.16}{34.05} =$	0.005	Clause 13.4.1.1(a)

Strong Axis Moment - (kips-ft)



Strong axis section capacity in bending	$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi F_y Z_x} = \frac{2.59}{7.99} =$	0.325	Clause 13.5(a)

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending	$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi F_y Z_y} = \frac{0.77}{7.99} =$	0.097	Clause 13.5(a)
Axial Compression and Bending cross-sectional Strength Check ω_{1x}=0.48; ω_{1y}=1.00; U_{1x}=1.00; U_{1y}=1.03;	$\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =$	0.569	Clause 13.8.3(a)

Axial Compression and Bending overall member Strength Check ω_{1x}=0.48; ω_{1y}=1.00; U_{1x}=0.50; U_{1y}=1.03; $\frac{C_f}{C_{rx}} + \frac{U_{1x} M_{fx}}{\phi Z_x F_y} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$	**0.417**	Clause 13.8.3(b)
Axial Compression and Bending lateral torsional buckling strength check **ω_{1x}=0.48; ω_{1y}=1.00; U_{1x}=1.00; U_{1y}=1.03;** $\frac{C_f}{C_{ry}} + \frac{U_{1x} M_{fx}}{M_{rx}} + \frac{U_{1y} M_{fy}}{\phi Z_y F_y} =$	**0.581**	**Clause 13.8.3(c)**

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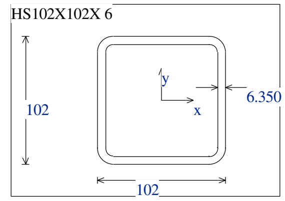
Code Details

Project: MKIV
Structure: Steel Column - vert.
Filename: C:\USERS\GHAISS\DOCUMENTS\BERGER\MKIV\MODEL 9 (WITH CORNER SLEEVES).TEL
Engineer:

Date:09/05/2012

Member: 3517 S-FRAME Section is HS102X102X 6
Member is part of group: Section 19
Note: Neglecting: axial<0.0 kips, shear<0.0 kips, moment<0.0 k-ft
Note: Member in braced frame(s).

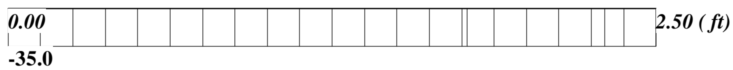


Load Combination 3 Resistance Y 1 (Bending + Compression)

Section classification (f_y=350 MPa); Section Class = **1** — Clause 11

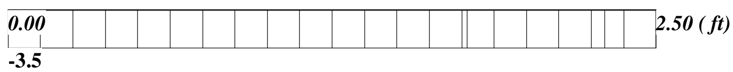
Governing geometrical slenderness ratio — Clause 10.4.2.1
k_x=1.00; k_y=1.00; k_yL/r_y=19.8; $\frac{k_x L/r_x}{200} = \frac{20}{200} =$ **0.099**

Axial Load - (kips)



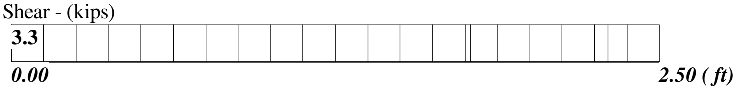
Factored Compressive Resistance Check — Clause 13.3.1
n=1.34; λ_x=0.264; Class C; $\frac{C_f}{C_{rx}} = \frac{C_f}{\phi A F_y (1+\lambda^{2n})^{-1/n}} = \frac{C_f}{\phi A\ (343\ \text{MPa})} = \frac{35}{161} =$ **0.218**

Strong Axis Shear - (kips)



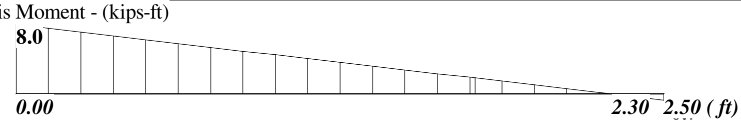
Strong axis shear strength check — Clause 13.4.1.1(a)
A_w= 1290 mm^2; $\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66F_y} = \frac{3}{60} =$ **0.058**

Weak Axis Shear - (kips)



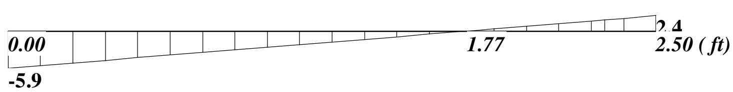
Weak axis shear strength check — Clause 13.4.1.1(a)
A_w= 1290 mm^2; $\frac{V_{fx}}{\phi A_w F_s} = \frac{V_{fx}}{\phi A_w 0.66F_y} = \frac{3}{60} =$ **0.055**

Strong Axis Moment - (kips-ft)



Strong axis section capacity in bending — Clause 13.5(a)
$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi F_y Z_x} = \frac{8}{19} =$ **0.425**

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending — Clause 13.5(a)
$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi F_y Z_y} = \frac{6}{19} =$ **0.313**

Axial Compression and Bending cross-sectional Strength Check — **Clause 13.8.3(a)**
ω_{1x}=0.57; ω_{1y}=0.44; U_{1x}=1.00; U_{1y}=1.00; $\frac{C_f}{\phi A F_y} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =$ **0.952**

Design Code: CAN/CSA S16-09
Steel Table : Canadian (CISC)
Analysis Program: S-FRAME (Linear static analysis)

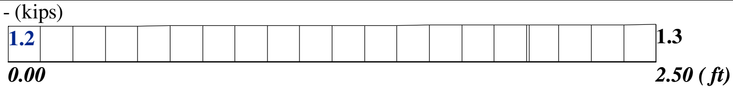
S-STEEL
Version Version 10.00

Axial Compression and Bending overall member Strength Check ω_{1x}=0.57; ω_{1y}=0.44; U_{1x}=0.57; U_{1y}=0.44;	$\frac{C_f}{C_{rx}} + \frac{U_{1x}\,M_{fx}}{\phi\,Z_x\,F_y} + \frac{U_{1y}\,M_{fy}}{\phi\,Z_y\,F_y} =$	**0.600**	Clause 13.8.3(b)
Axial Compression and Bending overall member Strength Check ω_{1x}=0.56; ω_{1y}=0.44; U_{1x}=0.57; U_{1y}=0.44;	$\frac{C_f}{C_{rx}} + \frac{U_{1x}\,M_{fx}}{\phi\,Z_x\,F_y} + \frac{U_{1y}\,M_{fy}}{\phi\,Z_y\,F_y} =$	**0.618**	Clause 13.8.3(b)
Axial Compression and Bending lateral torsional buckling strength check ω_{1x}=0.56; ω_{1y}=0.44; U_{1x}=1.00; U_{1y}=0.44;	$\frac{C_f}{C_{ry}} + \frac{U_{1x}\,M_{fx}}{M_{rx}} + \frac{U_{1y}\,M_{fy}}{\phi\,Z_y\,F_y} =$	**0.810**	Clause 13.8.3(c)

Load Combination 5 Resistance X 1 (Bending + Tension)

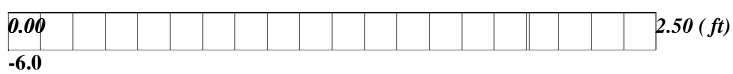
Section classification (f_y=350 MPa);	Section Class =	**1**	Clause 11
Governing geometrical slenderness ratio	$\frac{L/r_x}{300} = \frac{20}{300} =$	**0.066**	Clause 10.4.2.2

Axial Load - (kips)



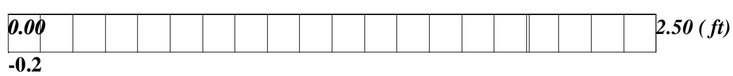
Factored Tensile Resistance Check A_g=2320 mm^2;	$\frac{T_f}{T_r} = \frac{T_f}{\phi A_g F_y} = \frac{1}{164} =$	**0.008**	Clause 13.2 a (i)

Strong Axis Shear - (kips)



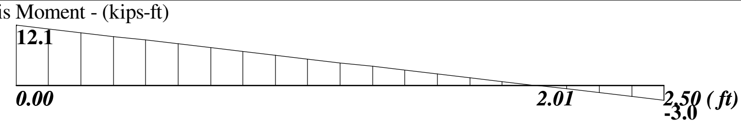
Strong axis shear strength check A_w= 1290 mm^2;	$\frac{V_{fy}}{\phi\,A_w\,F_s} = \frac{V_{fy}}{\phi\,A_w\,0.66F_y} = \frac{6}{60} =$	**0.100**	Clause 13.4.1.1(a)

Weak Axis Shear - (kips)



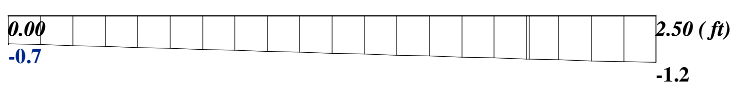
Weak axis shear strength check A_w= 1290 mm^2;	$\frac{V_{fx}}{\phi\,A_w\,F_s} = \frac{V_{fx}}{\phi\,A_w\,0.66F_y} = \frac{0.20}{60.31} =$	**0.003**	Clause 13.4.1.1(a)

Strong Axis Moment - (kips-ft)



Strong axis section capacity in bending	$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi\,F_y\,Z_x} = \frac{12}{19} =$	**0.643**	Clause 13.5(a)

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending	$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi\,F_y\,Z_y} = \frac{1}{19} =$	**0.066**	Clause 13.5(a)

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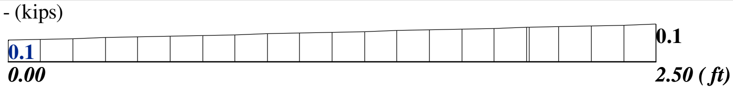
Axial Tension and Bending Strength Check	$\frac{T_f}{T_r} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =$	**0.716**	**Clause 13.9(a)**
Axial Tension and Bending Stability Check	$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} - \frac{T_f Z_x}{M_{rx} A} =$	**0.700**	Clause 13.9(b)
Note: Torsion is beyond scope and is not checked	Torsion (kips-ft) =	**1.0**	

Load Combination 6 Resistance X 2 (Bending + Tension)

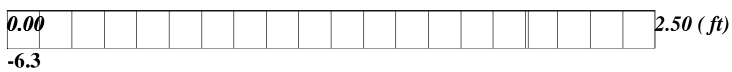
Section classification (f_y=350 MPa);	Section Class =	**1**	Clause 11
Governing geometrical slenderness ratio	$\frac{L/r_x}{300} = \frac{20}{300} =$	**0.066**	Clause 10.4.2.2

Axial Load - (kips)



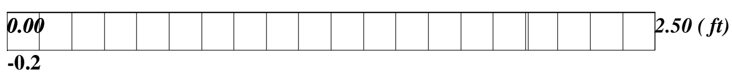
Factored Tensile Resistance Check A_g=2320 mm^2;	$\frac{T_f}{T_r} = \frac{T_f}{\phi A_g F_y} = \frac{0.13}{164.29} =$	**0.001**	Clause 13.2 a (i)

Strong Axis Shear - (kips)



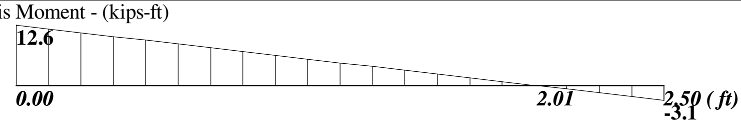
Strong axis shear strength check A_w= 1290 mm^2;	$\frac{V_{fy}}{\phi A_w F_s} = \frac{V_{fy}}{\phi A_w 0.66F_y} = \frac{6}{60} =$	**0.104**	Clause 13.4.1.1(a)

Weak Axis Shear - (kips)



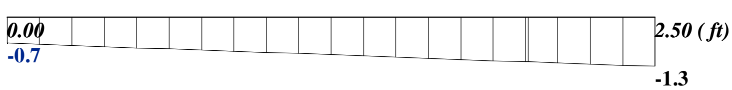
Weak axis shear strength check A_w= 1290 mm^2;	$\frac{V_{fx}}{\phi A_w F_s} = \frac{V_{fx}}{\phi A_w 0.66F_y} = \frac{0.24}{60.31} =$	**0.004**	Clause 13.4.1.1(a)

Strong Axis Moment - (kips-ft)



Strong axis section capacity in bending	$\frac{M_{fx}}{M_{rx}} = \frac{M_{fx}}{\phi F_y Z_x} = \frac{13}{19} =$	**0.666**	Clause 13.5(a)

Weak Axis Moment - (kips-ft)



Weak axis section capacity in bending	$\frac{M_{fy}}{M_{ry}} = \frac{M_{fy}}{\phi F_y Z_y} = \frac{1}{19} =$	**0.068**	Clause 13.5(a)

Axial Tension and Bending Strength Check $$\frac{T_f}{T_r} + \frac{M_{fx}}{\phi Z_x F_y} + \frac{M_{fy}}{\phi Z_y F_y} =$$	**0.734**	**Clause 13.9(a)**
Axial Tension and Bending Stability Check $$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} - \frac{T_f Z_x}{M_{rx} A} =$$	**0.733**	Clause 13.9(b)