CLARK REDER
ENGINEERING

STRUCTURAL SUBMITTAL PACKAGE

for

Reliable Design

Audio Towers (Indoor)

Project Location: US

Event Dates: Varies

Submittal Date: 12/19/2019

Clark Reder Project Number: 19.418.17

Prepared by:

STATE OF TEXAS
12/19/2019
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LICENSED
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Clark Reder Engineering
F-12154

Jeffrey M. Reder, P.E.
TX Registration #: 124100

Table of Contents for Structural Submittal Package

Project Information

Project Summary

The project referenced by this submittal consists of indoor steel framed audio support towers. The towers will be used at a maximum height of 30 feet for indoor use only.

Scope of Review

Clark Reder Engineering reviewed the capacity of the steel framed towers, headblock and base members. The audio tower was reviewed for a maximum cantilever load of 5,200 pounds per the Client. Rear ballast is required for this installation and can be found in Appendix A.

Conclusions

Our review has concluded that the Audio Tower meets the structural requirements of the 2015 International Building Code, ASCE 7-10, AND ASCE 37-14. The structure shall be used indoors only for a maximum height of 30 feet.

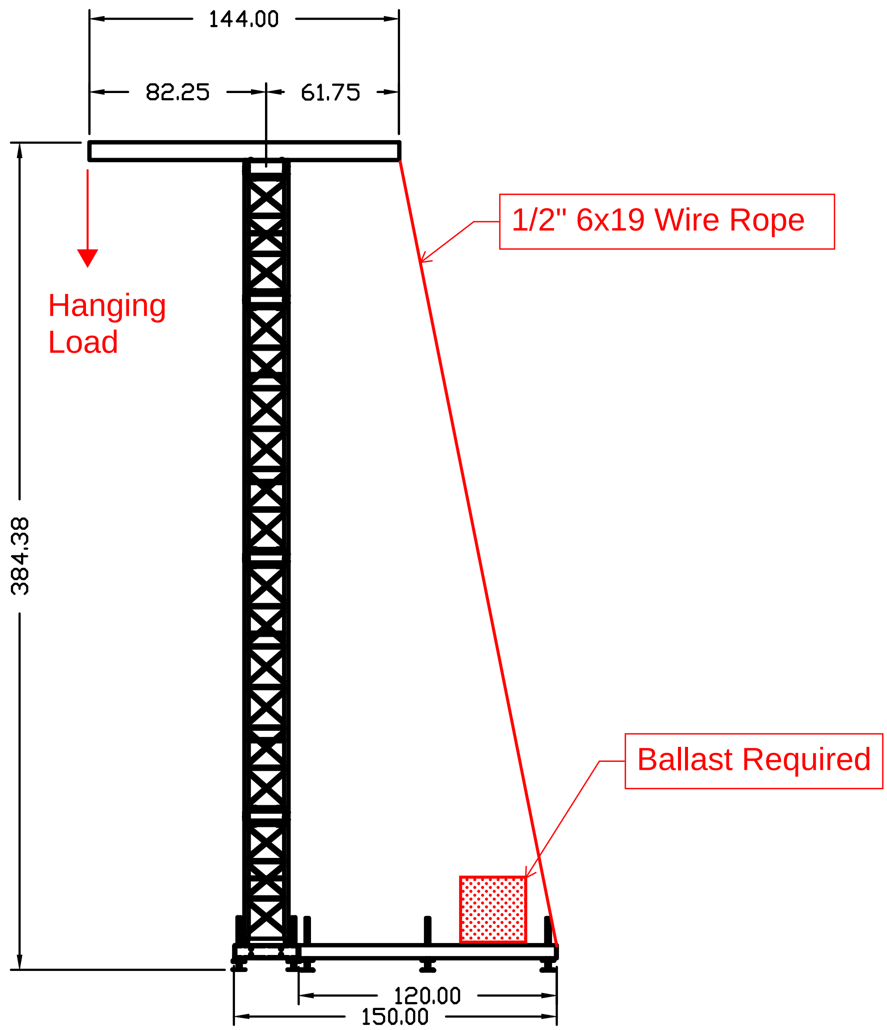
Limitations and Exceptions

The scope of review for this submittal is limited to the items listed above. All other temporary or permanent structures on site not specifically referenced above under “Scope of Review” are the responsibility of others.

Where the items covered by this submittal are attached to existing structures, it is the responsibility of the engineer of record for those existing structures to review the impact of the elements referenced in this submittal.

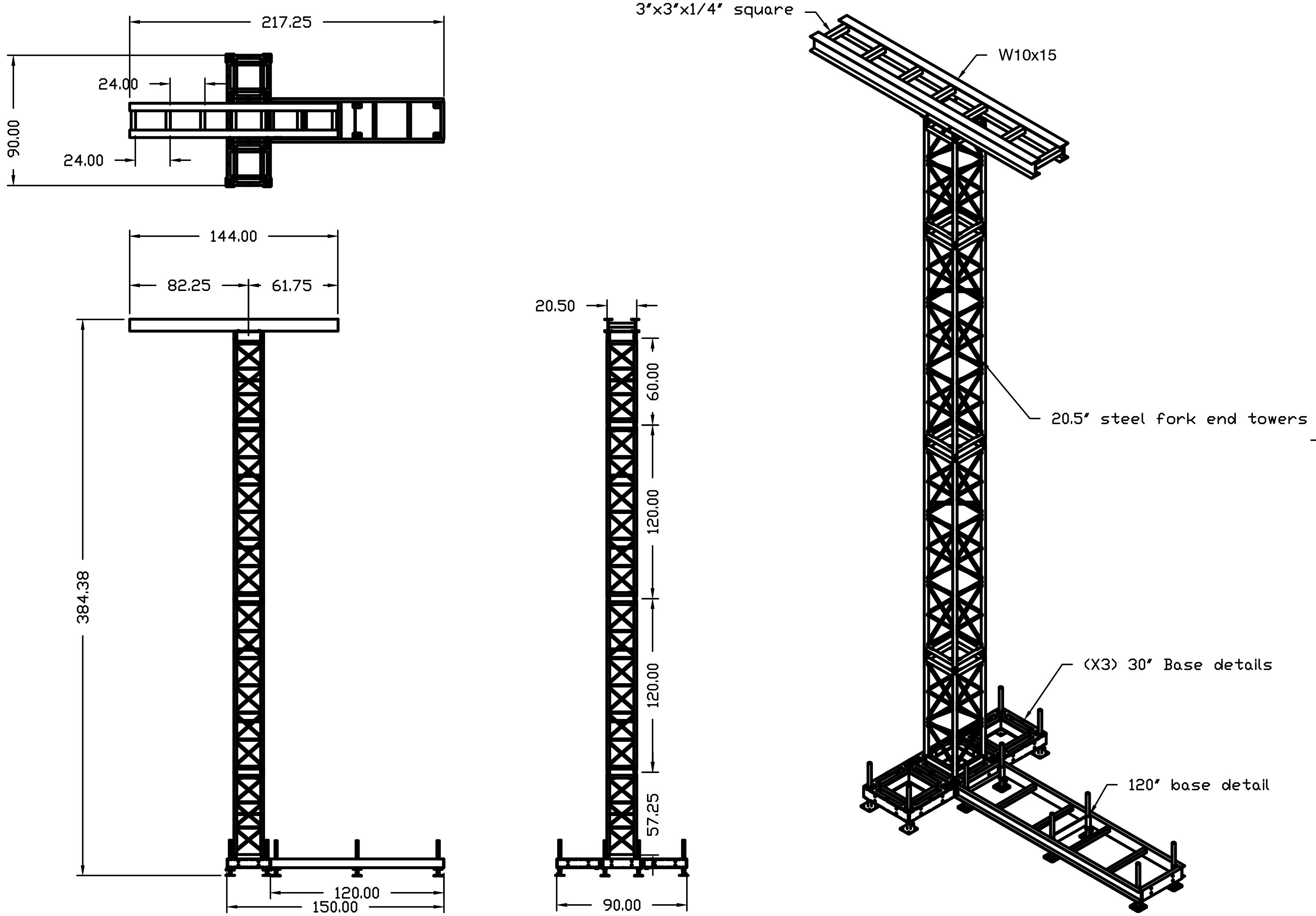
APPENDIX A

Hanging Load (lb)	Ballast Required (lb)
1000	525
2000	1450
3000	2375
4000	3300
5200	4400



Ballast shown is for indoor use only

RevNo	Revision note	Date



ALL DIMENSIONS IN INCHES

RELIABLE DESIGN

(972) 584-0551 **WWW.RDS-WEB.COM**

12111901.1

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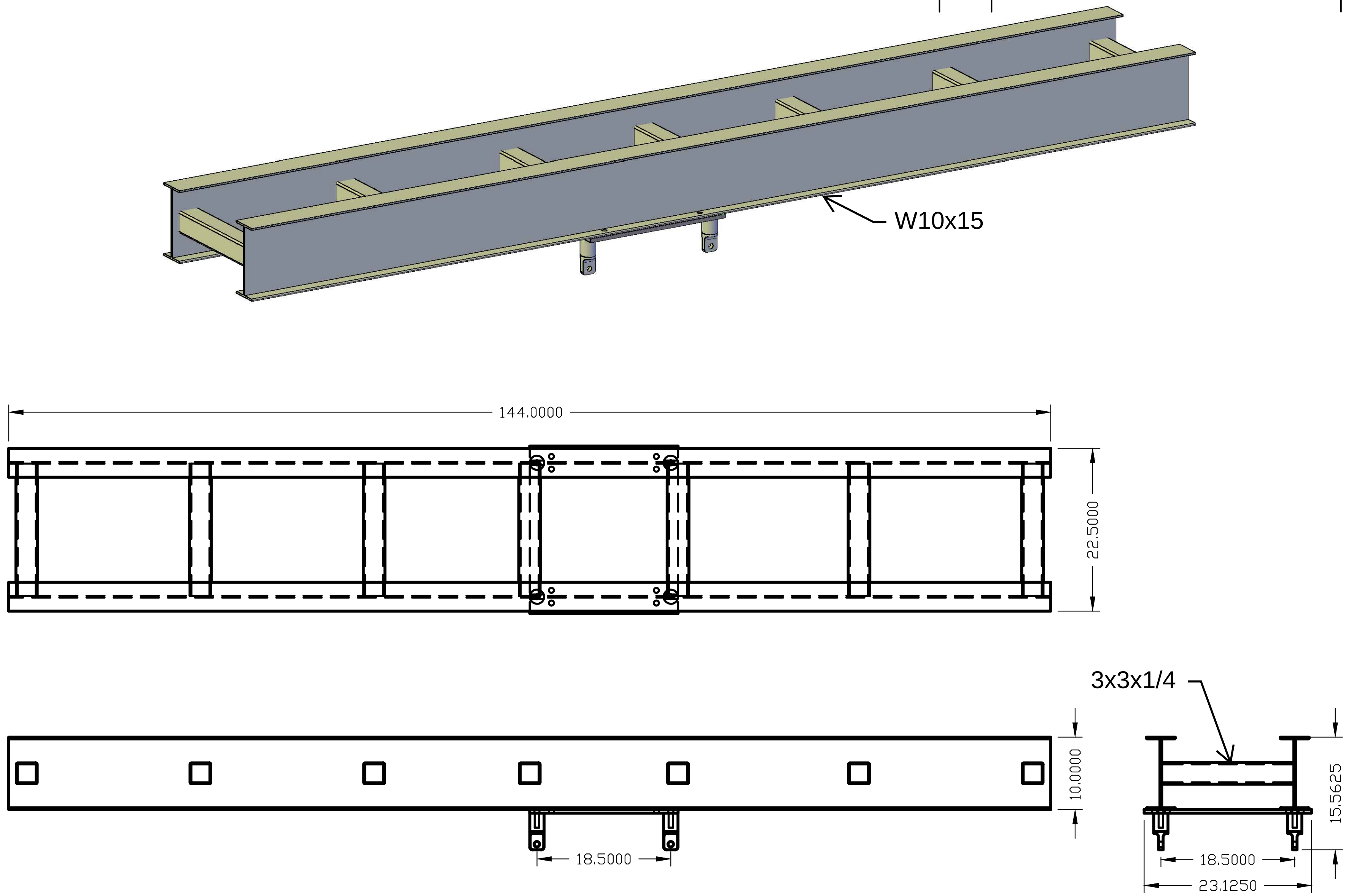
Date 12/11/19	Scale NTS	Job#:	Sheet 1/1



RevNo	Revision note	Date

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DTHB-20X12-6.2K

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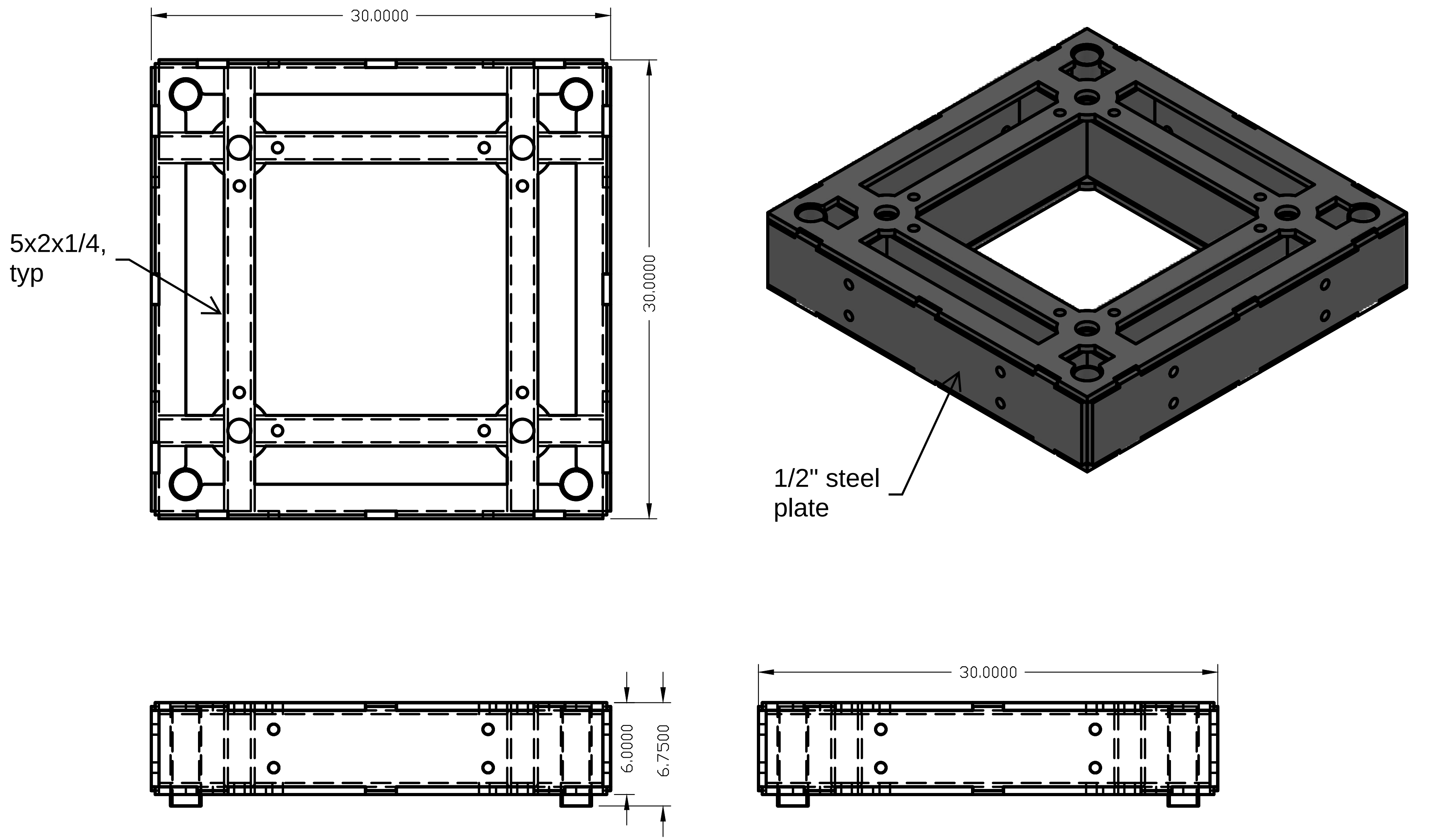

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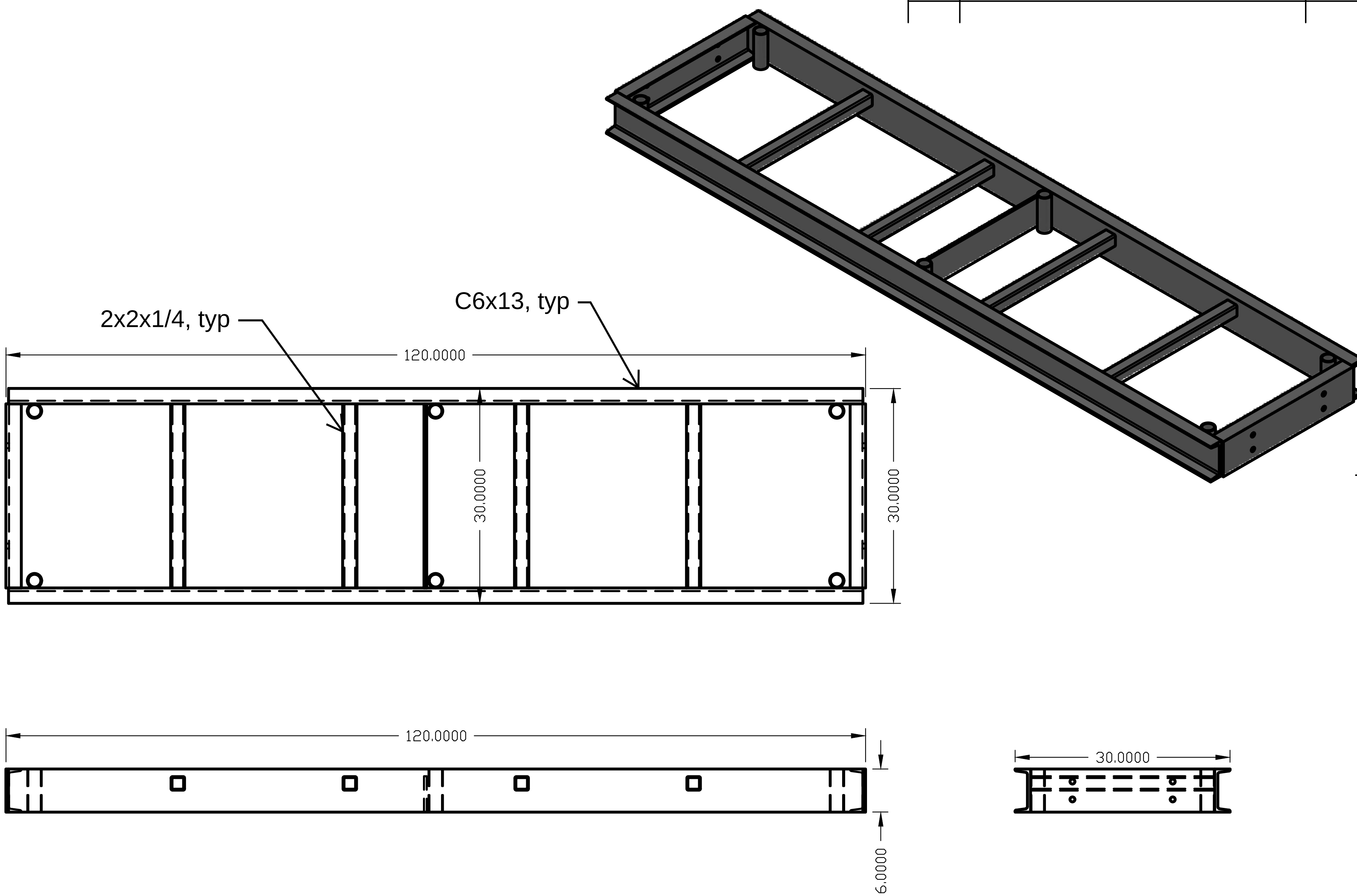
RevNo	Revision note	Date



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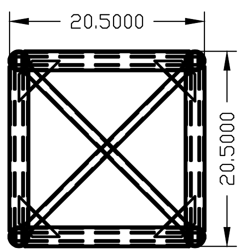
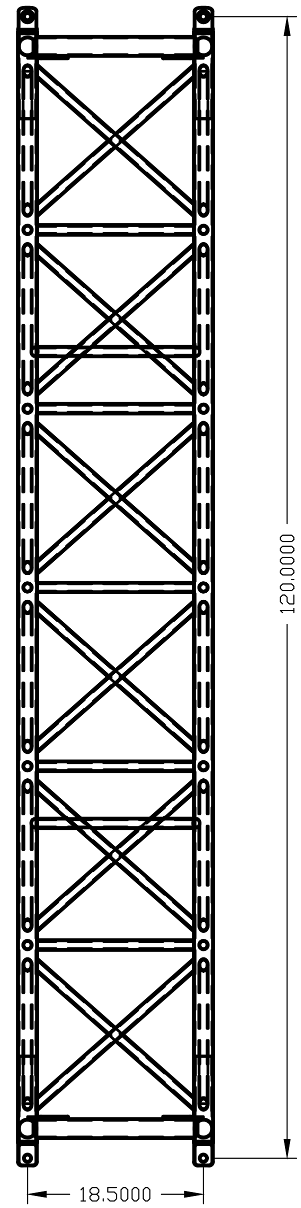
BSDTL-120-V2

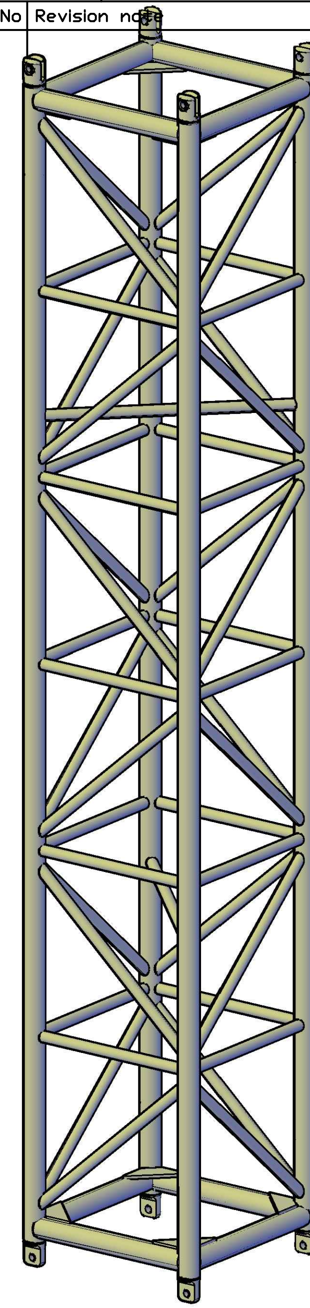
Client:			Project:	
....			.....	
Date 00/00/00	Scale NTS		Job#:	Sheet 1/1

Main Chords are 2.00" O.D. x 11 GA DOM steel tube
Diagonals are: 1.00" O.D. x 11GA DOM steel tube
Forks are 12L14 using a $\frac{3}{4}$" clevis pin

RevNo	Revision note	Date







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20.5X20.5X120FE-S120

Client:			Project:	
Date 00/00/00	Scale NTS		Job#:	Sheet 1/1

RevNo	Revision note	Date

Main Chords are 2.00" O.D. x 11 GA DOM steel tube

Diagonals are: 1.00" O.D. x 11GA DOM steel tube

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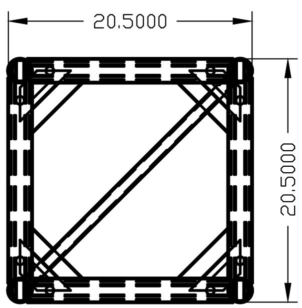

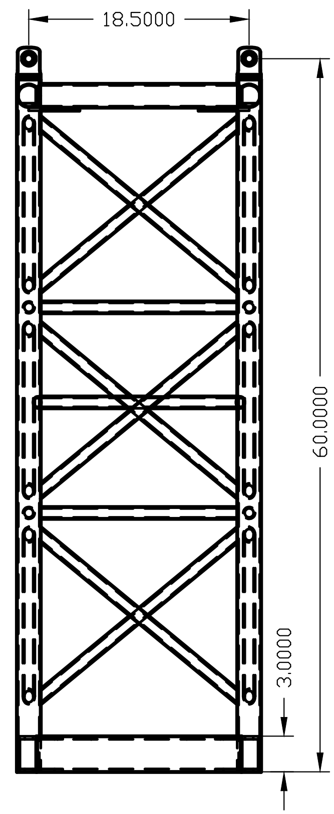

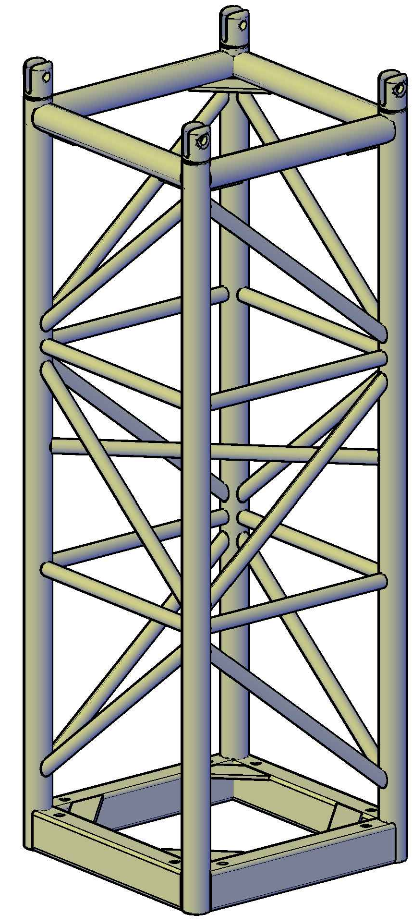

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20X60-TWRBS				
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APPENDIX B

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Audio Towers

Event Date & Location: Anywhere USA, Indoors, Temporary

Codes and Referenced Standards

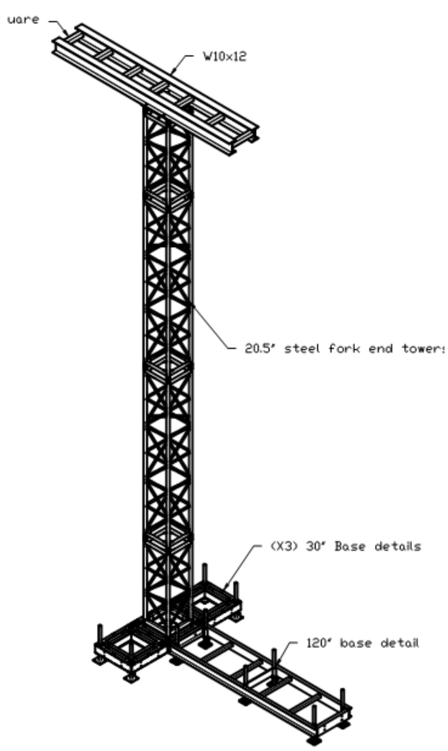
- 2015 International Building Code
- Aluminum Design Manual, 2015 ed.
- American Institute of Steel Construction, Steel Construction Manual 15th Edition
- American Society of Civil Engineers 7-10 (ASCE 7-10) *"Minimum Design Loads for Buildings and Other Structures"*
- American Society of Civil Engineers 37-14 (ASCE 37-14) *"Design Loads on Structures During Construction"*
- ANSI E 1.21-2013 *"Temporary Structures Used for Technical Production of Outdoor Entertainment Events"*
- ANSI E 1.2-2012 *"Manufacture and Use of Aluminum Trusses and Towers"*
- ICC 300-2012, *"Bleachers, Folding and Telescopic Seating, and Grandstands"*

Project Description

The indoor style delay towers will be used to support up to a maximum speaker weight of 5,200 pounds. The tower will be at a maximum height of 30 feet with a 6 ft cantilever to support the audio.

Analysis Assumptions/Design Criteria

Structure is indoors.
Structure will require a rear ballast and guy wire.



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Gravity Loads

Maximum weight of speaker array: $P_{speaker\_max} := 5200 \cdot lbf$

Incremental weight of speakers: $P_{speaker\_1} := 1000 \cdot lbf$ $P_{speaker\_3} := 3000 \cdot lbf$

$P_{speaker\_2} := 2000 \cdot lbf$ $P_{speaker\_4} := 4000 \cdot lbf$

Weight of tower section: $wt_{10\_tower} := 242 \cdot lbf \cdot 3 = 726.00\,lbf$

Weight of 30"x30" base:

$$wt_{base\_30} := 195.0962 \cdot in^3 \cdot \gamma_s \cdot 2 + 73.26 \cdot in^3 \cdot \gamma_s \cdot 4 + 52.74 \cdot in^3 \cdot \gamma_s \cdot 2 \ldots = 296.06\,lbf$$
$$+ 90.93 \cdot in^3 \cdot \gamma_s \cdot 2 + 13.58 \cdot in^3 \cdot \gamma_s \cdot 4 + 4.79 \cdot in^3 \cdot (\gamma_s \cdot 4)$$

Weight of 10 ft base:

$$wt_{base\_10} := 467.81 \cdot in^3 \cdot \gamma_s \cdot 2 + 43.57 \cdot in^3 \cdot \gamma_s \cdot 4 + 100.1 \cdot in^3 \cdot \gamma_s \cdot 2 \ldots = 392.39\,lbf$$
$$+ 48.16 \cdot in^3 \cdot \gamma_s \cdot 1 + 4.25 \cdot in^3 \cdot \gamma_s \cdot 6$$

Weight of head block: $wt_{head\_block} := 144 \cdot in \cdot 2 \cdot 15 \cdot plf + 49.26 \cdot in^3 \cdot \gamma_s \cdot 7 = 457.78\,lbf$

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General Overturning

Assume load of array is at tip of tower.

Resisting moment of assemly for tipping caused by audio eccentricity:

$$M_{Resist} := wt_{10\_tower} \cdot \frac{18.5 \cdot in}{2} + wt_{base\_30} \cdot 3 \cdot \frac{18.5 \cdot in}{2} + wt_{base\_10} \cdot 84.25 \cdot in = 3999.13 \cdot ft \cdot lbf$$

Overturning due to various loads:

$$M_{OT\_max} := P_{speaker\_max} \cdot 70.5 \cdot in = 30.55 \cdot ft \cdot kip$$

$$M_{OT\_1} := P_{speaker\_1} \cdot 70.5 \cdot in = 5.88 \cdot ft \cdot kip$$

$$M_{OT\_2} := P_{speaker\_2} \cdot 70.5 \cdot in = 11.75 \cdot ft \cdot kip$$

$$M_{OT\_3} := P_{speaker\_3} \cdot 70.5 \cdot in = 17.62 \cdot ft \cdot kip$$

$$M_{OT\_4} := P_{speaker\_4} \cdot 70.5 \cdot in = 23.50 \cdot ft \cdot kip$$

Ballast will be located centered in the rear between the (4) screw jacks:

$$ballast\_max := \frac{1.5 \cdot M_{OT\_max} - M_{Resist}}{114.0625 \cdot in} = 4400.31\ lbf$$

$$ballast\_1 := \frac{1.5 \cdot M_{OT\_1} - M_{Resist}}{114.0625 \cdot in} = 506.39\ lbf$$

$$ballast\_2 := \frac{1.5 \cdot M_{OT\_2} - M_{Resist}}{114.0625 \cdot in} = 1433.52\ lbf$$

$$ballast\_3 := \frac{1.5 \cdot M_{OT\_3} - M_{Resist}}{114.0625 \cdot in} = 2360.64\ lbf$$

$$ballast\_4 := \frac{1.5 \cdot M_{OT\_4} - M_{Resist}}{114.0625 \cdot in} = 3287.76\ lbf$$

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Consider Lateral Sway - Side to Side

Use 5% of the audio weight.

$$M_{OT\_maxside} := 5\% \cdot P_{speaker\_max} \cdot 30 \cdot ft = 7.80 \cdot ft \cdot kip$$

$$M_{OT\_1side} := 5\% \cdot P_{speaker\_1} \cdot 30 \cdot ft = 1.50 \cdot ft \cdot kip$$

$$M_{OT\_2side} := 5\% \cdot P_{speaker\_2} \cdot 30 \cdot ft = 3.00 \cdot ft \cdot kip$$

$$M_{OT\_3side} := 5\% \cdot P_{speaker\_3} \cdot 30 \cdot ft = 4.50 \cdot ft \cdot kip$$

$$M_{OT\_4side} := 5\% \cdot P_{speaker\_4} \cdot 30 \cdot ft = 6.00 \cdot ft \cdot kip$$

Resisting moment neglecting rear 10' base

$$M_{Resist\_side} := wt_{10\_tower} \cdot \frac{85.5 \cdot in}{2} + wt_{base\_30} \cdot 3 \cdot \frac{85.5 \cdot in}{2} = 5750.48 \cdot ft \cdot lbf$$

Ballast will be located centered on the 30" base:

$$ballast\_max\_side := \frac{1.5 \cdot M_{OT\_maxside} - M_{Resist\_side} - P_{speaker\_max} \cdot 42.75 \cdot in}{72.75 \cdot in} = -2074.31\ lbf$$

$$ballast\_1\_side := \frac{1.5 \cdot M_{OT\_1side} - M_{Resist\_side} - P_{speaker\_1} \cdot 42.75 \cdot in}{72.75 \cdot in} = -1165.03\ lbf$$

$$ballast\_2\_side := \frac{1.5 \cdot M_{OT\_2side} - M_{Resist\_side} - P_{speaker\_2} \cdot 42.75 \cdot in}{72.75 \cdot in} = -1381.52\ lbf$$

$$ballast\_3\_side := \frac{1.5 \cdot M_{OT\_3side} - M_{Resist\_side} - P_{speaker\_3} \cdot 42.75 \cdot in}{72.75 \cdot in} = -1598.02\ lbf$$

$$ballast\_4\_side := \frac{1.5 \cdot M_{OT\_4side} - M_{Resist\_side} - P_{speaker\_4} \cdot 42.75 \cdot in}{72.75 \cdot in} = -1814.51\ lbf$$

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Audio Towers - 20.5x20.5

Safety Factors Used in Design

Bending:	$\Omega_b := 1.67$	Bearing:	$\Omega_{brg} := 2.0$	Pin shear:	$\Omega_v := 2.0$	Weld :	$\Omega_{weld} := 2.0$
Tension:	$\Omega_t := 1.67$	Tensile rupture:	$\Omega_{tr} := 2.0$	Shear yielding:	$\Omega_{sy} := 1.5$		
Compression:	$\Omega_c := 1.67$	Shear rupture:	$\Omega_{sr} := 2.0$	Block shear:	$\Omega_{bs} := 2.0$		

Chord member properties - 2"x11 GA (DOM):

Pipe Properties

Pipe diameter: $OD_{chord} := 2in$

Pipe thickness: $t_{chord} := 0.12in$

Pipe yield strength: $Fy_{chord} := 75 \cdot ksi$

Pipe tensile strength: $Fu_{chord} := 85 \cdot ksi$

Radius: $R_{chord} := \frac{OD_{chord}}{2} = 1 \cdot in$

Inner diameter: $ID_{chord} := OD_{chord} - 2 \cdot t_{chord} = 1.76 \cdot in$

Moment of inertia: $I_{chord} := \frac{\pi \cdot \left(OD_{chord}^{\ 4} - ID_{chord}^{\ 4}\right)}{64} = 0.314 \cdot in^4$

Area: $A_{chord} := \frac{\pi \cdot \left(OD_{chord}^{\ 2} - ID_{chord}^{\ 2}\right)}{4} = 0.709 \cdot in^2$

Radius of gyration: $r_{chord} := \sqrt{\frac{I_{chord}}{A_{chord}}} = 0.666 \cdot in$

Elastic section modulus: $S_{chord} := \frac{\pi \cdot \left(OD_{chord}^{\ 4} - ID_{chord}^{\ 4}\right)}{32 \cdot OD_{chord}} = 0.314 \cdot in^3$

Plastic section moduls: $Z_{chord} := \frac{\left(OD_{chord}^{\ 3} - ID_{chord}^{\ 3}\right)}{6} = 0.425 \cdot in^3$

D over t: $D\_t_{chord} := \frac{OD_{chord}}{t_{chord}} = 16.667$

Weight of chords: $wt_{chord} := \gamma_s \cdot A_{chord}$ $wt_{chord} = 2.412 \cdot plf$

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Diagonal Member Properties - 1"x11GA (DOM)

Pipe diameter: $OD_{diag} := 1in$

Pipe thickness: $t_{diag} := 0.125in$

Pipe yield strength: $Fy_{diag} := 75 \cdot ksi$

Pipe tensile strength: $Fu_{diag} := 85 \cdot ksi$

Radius: $R_{diag} := \frac{OD_{diag}}{2} = 0.5 \cdot in$

Inner diameter: $ID_{diag} := OD_{diag} - 2 \cdot t_{diag} = 0.75 \cdot in$

Moment of inertia: $I_{diag} := \frac{\pi \cdot \left(OD_{diag}^{\ 4} - ID_{diag}^{\ 4}\right)}{64} = 0.034 \cdot in^4$

Area: $A_{diag} := \frac{\pi \cdot \left(OD_{diag}^{\ 2} - ID_{diag}^{\ 2}\right)}{4} = 0.344 \cdot in^2$

Radius of gyration: $r_{diag} := \sqrt{\frac{I_{diag}}{A_{diag}}} = 0.313 \cdot in$

Elastic section modulus: $S_{diag} := \frac{\pi \cdot \left(OD_{diag}^{\ 4} - ID_{diag}^{\ 4}\right)}{32 \cdot OD_{diag}} = 0.067 \cdot in^3$

Plastic section moduls: $Z_{diag} := \frac{\left(OD_{diag}^{\ 3} - ID_{diag}^{\ 3}\right)}{6} = 0.096 \cdot in^3$

D over t: $D\_t_{diag} := \frac{OD_{diag}}{t_{diag}} = 8$

Weight of diagonals: $wt_{diag} := \gamma_s \cdot A_{diag}$ $wt_{chord} = 2.412 \cdot plf$

Horizontal Member Properties - 2"x11 GA (DOM)

Pipe diameter: $OD_{horz} := 2in$

Pipe thickness: $t_{horz} := 0.125in$

Pipe yield strength: $Fy_{horz} := 75 \cdot ksi$

Pipe tensile strength: $Fu_{horz} := 85 \cdot ksi$

Radius: $R_{horz} := \frac{OD_{horz}}{2} = 1 \cdot in$

Inner diameter: $ID_{horz} := OD_{horz} - 2 \cdot t_{horz} = 1.75 \cdot in$

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Moment of inertia: $I_{horz} := \dfrac{\pi \cdot \left(OD_{horz}^{\,4} - ID_{horz}^{\,4}\right)}{64} = 0.325 \cdot in^4$

Area: $A_{horz} := \dfrac{\pi \cdot \left(OD_{horz}^{\,2} - ID_{horz}^{\,2}\right)}{4} = 0.736 \cdot in^2$

Radius of gyration: $r_{horz} := \sqrt{\dfrac{I_{horz}}{A_{horz}}} = 0.664 \cdot in$

Elastic section modulus: $S_{horz} := \dfrac{\pi \cdot \left(OD_{horz}^{\,4} - ID_{horz}^{\,4}\right)}{32 \cdot OD_{horz}} = 0.325 \cdot in^3$

Plastic section moduls: $Z_{horz} := \dfrac{\left(OD_{horz}^{\,3} - ID_{horz}^{\,3}\right)}{6} = 0.44 \cdot in^3$

D over t: $D\_t_{horz} := \dfrac{OD_{horz}}{t_{horz}} = 16$

Weight of diagonals: $wt_{horz} := \gamma_s \cdot A_{horz}$ $\qquad wt_{horz} = 2.506 \cdot plf$

Global Column Section Properties

Truss chord centerline to centerline: $d_{column} := 18 \cdot in - OD_{chord} = 16 \cdot in$

Area of column: $A_{column} := 4 \cdot A_{chord}$ $\qquad A_{column} = 2.835 \cdot in^2$

Moment of inertia of column: $I_{column} := 4 \cdot I_{chord} + 4 \cdot A_{chord} \cdot \left(\dfrac{d_{column}}{2}\right)^2$ $\qquad I_{column} = 182.696 \cdot in^4$

Radius of gyration of column: $r_{column} := \sqrt{\dfrac{I_{column}}{A_{column}}}$ $\qquad r_{column} = 8.028 \cdot in$

Weight of typical tower: $wt_{tower\_mid} := A_{chord} \cdot \gamma_s \cdot 4 \cdot 116.75 \cdot in + A_{diag} \cdot \gamma_s \cdot 24 \cdot in \cdot 24 + A_{diag} \cdot \gamma_s \cdot 20 \cdot 16.5 \cdot in \ldots$
$+ 4 \cdot lbf \cdot 8 + A_{horz} \cdot \gamma_s \cdot 8 \cdot 16.5 \cdot in$

$wt_{tower\_mid} = 241.693\ lbf$

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Column Chord Analysis - 2"x11GA (ERW A53 steel)

Built Up Member Capacity - E.6

Distance between connectors (diagonals): $a_i := 19 \cdot in$

Effective length factor: $K_i := 0.86$ for all other case

Minimum radius of gyration on individual component: $r_i := r_{chord} = 0.666 \cdot in$

Ratio of a/r: $ratio\_a\_r := \dfrac{a_i}{r_i} = 28.527$

Slenderness ratio of built up member = E6-2a.

Column Global Axial Capacity

The column will be checked for the full length capacity of the global section. Use Table 4-22 of the AISC code to determine the available critical stress.

K factor: $k_{column} := 1.25$ vertical diagonal X-bracing

Column length: $L_{column} := 50 \cdot ft$

KL/r
: $kL\_r_{column} := \dfrac{k_{column} \cdot L_{column}}{r_{column}}$ $kL\_r_{column} = 93.427$

$$F_e := \frac{\pi^2 \cdot E_s}{\left(kL\_r_{column}\right)^2} = 32.791 \cdot ksi$$

$$F_{cr\_column} := \begin{vmatrix} 0.658^{\left(\frac{Fy_{chord}}{F_e}\right)} \cdot Fy_{chord} \ \text{ if } kL\_r_{column} < 4.71 \cdot \sqrt{\dfrac{E_s}{Fy_{chord}}} \\ 0.877 \cdot F_e \ \text{ if } kL\_r_{column} > 4.71 \cdot \sqrt{\dfrac{E_s}{Fy_{chord}}} \end{vmatrix} = 28.758 \cdot ksi$$

Allowable global axial load: $P_{a\_column} := F_{cr\_column} \cdot A_{column}$ $P_{a\_column} = 81.527 \cdot kip$

Column Local Axial Capacity

The chords are braced by diagonals on all sides. The local axial load capacity of each chord will be determined and compared to the global capacity.

K factor: $k_{local} := 1.0$

Column length: $L_{local} := 18 \cdot in$

KL/r: $kL\_r_{local} := \dfrac{k_{local} \cdot L_{local}}{r_{chord}}$ $\qquad kL\_r_{local} = 27.026$

$$F_e := \frac{\pi^2 \cdot E_s}{\left(\dfrac{k_{local} \cdot L_{local}}{r_{chord}}\right)^2} = 391.872 \cdot ksi$$

$$F_{cr\_local} := \begin{cases} 0.658^{\left(\frac{Fy_{chord}}{F_e}\right)} \cdot Fy_{chord} & \text{if } \dfrac{k_{local} \cdot L_{local}}{r_{chord}} < 4.71 \cdot \sqrt{\dfrac{E_s}{Fy_{chord}}} \\ 0.877 \cdot F_e & \text{if } \dfrac{k_{local} \cdot L_{local}}{r_{chord}} > 4.71 \cdot \sqrt{\dfrac{E_s}{Fy_{chord}}} \end{cases} = 69.226 \cdot ksi$$

Allowable global axial load: $P_{a\_local} := F_{cr\_local} \cdot A_{chord} \cdot 4$ $\qquad P_{a\_local} = 196.255 \cdot kip$

Per E.6.2 the effective slenderness ratio of the component shapes between fasteners shall not exceed 3/4 x the governing slenderness ratio of the built up member.

$kL\_r_{local} = 27.026$ less than $kL\_r_{column} \cdot 0.75 = 70.07$ Acceptable

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Allowable Concentrated Force On Chord Member

Check Flexural Capacity Chord Between Panel Points

Determine the maximum concentrated load that the chord can resist.

Check limiting width to thickness ratios for compression elements using Table B4.1.

$Dt\_flex := 0.11 \cdot \frac{E_s}{Fy_{chord}}$ $\quad Dt\_flex = 42.533$ $\quad >$ $\quad D\_t_{chord} = 16.667$ compact

$Dt\_comp := 0.07 \cdot \frac{E_s}{Fy_{chord}}$ $\quad Dt\_comp = 27.067$ $\quad >$ $\quad D\_t_{chord} = 16.667$ compac
t

$Dt\_F8 := 0.45 \cdot \frac{E_s}{Fy_{chord}}$ $\quad Dt\_comp = 27.067$ $\quad >$ $\quad D\_t_{chord} = 16.667$

Local buckling does not apply as section is compact. Yielding controls the flexural strength of the member.

Nominal moment capacity: $M_n := Fy_{chord} \cdot Z_{chord}$ $\quad M_n = 2.654 \cdot ft \cdot kip$

Allowable moment capacity: $M_{a\_chord} := \frac{M_n}{\Omega_b}$ $\quad M_{a\_chord} = 1.589 \cdot ft \cdot kip$

Allowable concentrated load: $P_{conc\_chord} := \frac{8 \cdot M_{a\_chord}}{L_{local}}$ $\quad P_{conc\_chord} = 8.477 \cdot kip$

asssumes load applied at midspan between diagonals and fixed end beam condition

Allowable Axial Tension Capacity

Tensile yielding on gross section: $P_{n\_y} := Fy_{chord} \cdot A_{chord}$ $\quad P_{n\_y} = 53.156 \cdot kip$

Allowable tension capacity: $P_{ta\_chord} := 4 \cdot \frac{P_{n\_y}}{\Omega_t}$ $\quad P_{ta\_chord} = 127.319 \cdot kip$

Diagonal Analysis - 1"x1/8" (ERW A53)

Diagonal Axial Capacity (Compression)

Per AISC Chapter E, Section 6 the lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 percent of the available compressive strength of the member.

K factor: $k_{diag} := 1.0$

Diagonal length: $L_{diag} := 19.8 \cdot in$

KL/r: $kL\_r_{diag} := \dfrac{k_{diag} \cdot L_{diag}}{r_{diag}}$ $kL\_r_{diag} = 63.36$ less than 140 - OK

Critical stress from Table 4-22: $F_{cr\_diag} := 16.5 \cdot ksi$

Allowable diagonal axial load: $P_{a\_diag} := F_{cr\_diag} \cdot A_{diag}$ $P_{a\_diag} = 5.67 \cdot kip$

Required lacing force per AISC: $P_{req\_diag} := 0.02 \cdot \left(F_{cr\_local} \cdot A_{chord}\right)$ $P_{req\_diag} = 981.275 \cdot lbf$

By inspection of the loads, the lacing meets the minimum requirements of the AISC code.

Forked End Capacity

The towers are connected using forked end connectors. The material strength of the pins and forks are as follows 12L14 steel connected with (2) spirol pins to the actual truss.

Pin yield stress: $Fy_{pin} := 60 \cdot ksi = 60 \cdot ksi$

Pin tensile stress: $Fu_{pin} := 78 \cdot ksi = 78 \cdot ksi$

The typical tower truss is connected using 0.75 diameter 12L14 steel pins. The pin material is not referenced of AISC 13th edition. However, use Table J3.2 to determine allowable solution in lieu of other information.

Table J3.2 (sim to A325): $F_{nv\_pin} := Fu_{pin} \cdot 0.5$ $\quad F_{nv\_pin} = 39 \cdot ksi$

Pin diameter: $d_{pin} := 0.75 \cdot in$

Area of pin: $A_{pin} := \dfrac{d_{pin}^{\ 2} \cdot \pi}{4}$ $\quad A_{pin} = 0.442 \cdot in^2$

Allowable pin shear at fork: $R_{n\_over\_\Omega.bolt} := \dfrac{F_{nv\_pin} \cdot A_{pin} \cdot 2}{2.0}$ $\quad R_{n\_over\_\Omega.bolt} = 17.23 \cdot kip$ bolt in double shear

Total fork bearing capacity: $R_{brg\_max} := 4 \cdot R_{n\_over\_\Omega.bolt}$ $\quad R_{brg\_max} = 68.919 \cdot kip$

Forked Properties

1.59" diameter 12L14 material solid round. End machined to a double clevis configuration or a single clevis. The forked end does not meet the dimensional requirements of a pin connected member per AISC. Therefore consider it a bolted connection.

Outside diameter of spigot: $OD_{fork} := 1.6in$

Wall thickness: $t_{fork} := 0.35 \cdot in$

Inside diameter: $ID_{fork} := OD_{fork} - 2 \cdot t_{fork}$ $\quad ID_{fork} = 0.9 \cdot in$

Area of spigot at inserted section: $A_{fork} := \dfrac{\pi \cdot \left(OD_{fork}^{\ 2} - ID_{fork}^{\ 2}\right)}{4}$ $\quad A_{fork} = 1.374 \cdot in^2$

Moment of inertia at inserted section: $I_{fork} := \dfrac{\pi \cdot \left(OD_{fork}^{\ 4} - ID_{fork}^{\ 4}\right)}{64}$ $\quad I_{fork} = 0.289 \cdot in^4$

Section modulus: $S_{fork} := \dfrac{\pi \cdot \left(OD_{fork}^{\ 4} - ID_{fork}^{\ 4}\right)}{325 \cdot OD_{fork}}$ $\quad S_{fork} = 0.036 \cdot in^3$

Male Plate Bending:

Plate Tension Capacity (AISC D5):

Safety factor for tensile yielding: $\Omega_{ty} := 1.67$

Safety factor for tensile rupture $\Omega_{tr} := 2.0$

Safety factor for shear yielding: $\Omega_{sy} := 1.5$

Safety factor for shear rupture: $\Omega_{sr} := 2$

Safety factor for weld: $\Omega_{weld} := 2.0$

Safety factor for flexure: $\Omega_b := 1.67$

Safety factor for tension: $\Omega_t := 1.67$

Safety factor for bearing: $\Omega_{tr} := 2.0$

Safety factor for compression: $\Omega_c := 1.67$

Safety factor for bearing: $\Omega_{brg} := 2.0$

spigot plate properties

Width of plate: $b := 1.97in$

Thickness of plate: $t := 0.625in$

Span of plate: $L_p := 0.96in$

area of plate: $A_p := b \cdot t$ $\qquad A_p = 1.231 \cdot in^2$

Plastic Section modulus: $Z_x := \dfrac{t \cdot b^2}{4}$ $\qquad Z_x = 0.606 \cdot in^3$ $\qquad Z_y := \dfrac{b \cdot t^2}{4}$ $\qquad Z_y = 0.192 \cdot in^3$

Allowable moments: $M_{y\_allow} := \dfrac{Fy_{pin} \cdot Z_y}{\Omega_b}$ $\qquad M_{y\_allow} = 0.576 \cdot kip \cdot ft$

$M_{x\_allow} := \dfrac{Fy_{pin} \cdot Z_x}{\Omega_b}$ $\qquad M_{x\_allow} = 1.816 \cdot kip \cdot ft$

Allowable Axial Compression

Fork plate

moment of inertia weak axis: $I_y := \dfrac{b \cdot t^3}{12}$ $\qquad I_y = 0.04 \cdot in^4$

radius of gyration weak axis: $r_y := \sqrt{\dfrac{I_y}{A_p}}$ $\qquad r_y = 0.18 \cdot in$

Effective length factor: $K_{eff\_sec} := 2$

Slenderness factor: $kl\_r_{sec} := \dfrac{K_{eff\_sec} \cdot L_p}{r_y}$ $\qquad kl\_r_{sec} = 10.642$

Elastic critical buckling value: $F_{e.sec} := \dfrac{\pi^2 \cdot 29000 \cdot ksi}{kl\_r_{sec}{}^2}$ $\quad F_{e.sec} = 2527.401 \cdot ksi$

Flexural buckling stress: $F_{cr.sec} := \text{if}\left[kl\_r_{sec} > 4.71 \cdot \sqrt{\dfrac{29000 \cdot ksi}{Fy_{pin}}}, 0.877 \cdot F_{e.sec}, \left(0.658^{\frac{Fy_{pin}}{F_{e.sec}}}\right) \cdot F\right]$ $\quad F_{cr.sec} = 59.407 \cdot ksi$

Allowable axial load: $P_{ca.sec} := \dfrac{F_{cr.sec} \cdot A_p}{\Omega_c}$ $\quad P_{ca.sec} = 43.799 \cdot kip$

Allowable axial compression load $P_{ca} := 4 \cdot P_{ca.sec}$ $\quad P_{ca} = 175.197 \cdot kip$

Fork Tension Capacity

The pin diameter is 0.75" and the hole size is slightly bigger. The difference is less than 1/32". Per AISC the the diameter of the pin hole shall shall not be more than 1/32" greater than the pin. Use section D5 for pin connected members.

Bearing strength at bolt holes (AISC J7)

Projected bearing area: $A_{pb} := t \cdot d_{pin}$

Available bearing strength: $R_{a\_bearing} := \dfrac{1.8 \cdot Fy_{pin} \cdot A_{pb}}{\Omega_{brg}}$ $\quad R_{a\_bearing} = 25.313 \cdot kip$

Plate Tension Capacity (AISC J4):

Tensile yielding on gross section J4-1: $P_{ny\_plate} := \dfrac{Fy_{pin} \cdot b \cdot t}{\Omega_{ty}}$ $\quad P_{ny\_plate} = 44.237 \cdot kip$

Tensile rupture of connecting elements (AISC J4-2): $P_{n\_tr} := \dfrac{Fu_{pin} \cdot \left[0.85 \cdot \left(b - \frac{13}{16} \cdot in\right) \cdot t\right]}{\Omega_{tr}}$

$P_{n\_tr} = 23.982 \cdot kip$

Shear yielding (AISC J4-3): $P_{sy\_plate} := \dfrac{0.6 \cdot Fy_{pin} \cdot (b \cdot t)}{\Omega_{sy}}$ $\quad P_{sy\_plate} = 29.55 \cdot kip$

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Shear rupture (AISC J4-4): $P_{sr\_plate} := \dfrac{0.6 \cdot Fu_{pin} \cdot \left[2 \cdot t \cdot \left(1 \cdot in + \dfrac{d_{pin}}{2}\right)\right]}{\Omega_{sr}}$ $P_{sr\_plate} = 40.219 \cdot kip$

The forked connection is controlled by the pin shear.

Allowable axial tension due to forked connection: $P_{a\_fork} := \left(4 \cdot R_{n\_over\_\Omega.bolt}\right)$ $P_{a\_fork} = 68.919 \cdot kip$

Spirol Coil Spring Pins

The forks are connected to the aluminum with Spirol Pins. The Spirol pins are coiled 3/8" diameter pins located 1" apart with a 1/2" edge distance to the edge of steel. Use a factor of safety of 2 on the coil pins.

Capacity of coiled spirol pin in double shear: $P_{allow\_coil\_pin} := 12.6 \cdot kip$

There are two pins therefore allowable load is $P_{allow\_coil} := \dfrac{2 \cdot P_{allow\_coil\_pin}}{2}$ $P_{allow\_coil} = 12.6 \cdot kip$

There are (4) chords $P_{allow\_coil} \cdot 4 = 50400\ lbf$

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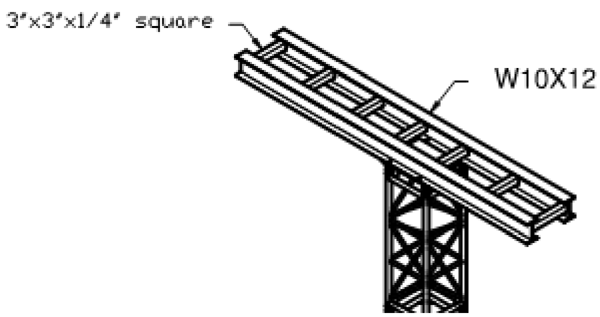
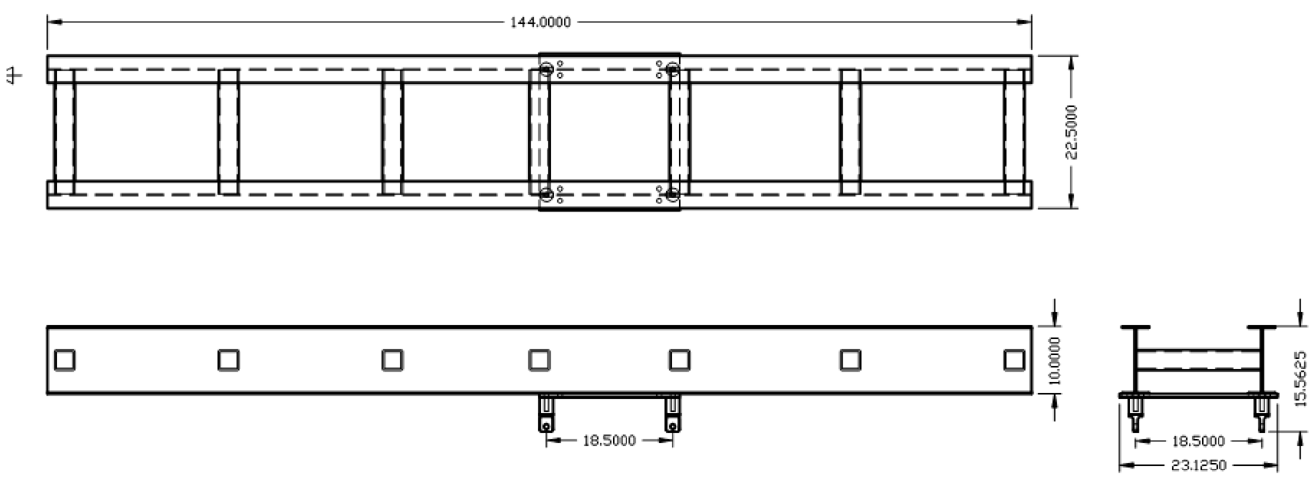
Audio Headblock Assembly





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Headblock Beam

Standard Beam AISC Parameters:

$E := 29000 \cdot ksi$ $K_x := 1.0$ $K_y := 1.0$ $C_b := 1.0$ $C_{mx} := 1.0$ $C_{my} := 1.0$ $Ratio := 1.0$

Beam $B := 1$ $span := 144 \cdot in$ $bm_{wt} := 0 \cdot plf$

$$p_x := \begin{pmatrix} 0 \cdot kip \\ 0 \cdot kip \\ 0 \cdot kip \\ 0 \cdot kip \end{pmatrix} \quad d_{px} := \begin{pmatrix} 0.5 \cdot span \\ 20 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix} \quad w_{1x} := \begin{pmatrix} bm_{wt} \\ 0 \cdot klf \\ 0 \cdot klf \\ 0 \cdot klf \end{pmatrix} \quad d_{w1x} := \begin{pmatrix} 0 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix} \quad w_{2x} := \begin{pmatrix} bm_{wt} \\ 0 \cdot klf \\ 0 \cdot klf \\ 0 \cdot klf \end{pmatrix} \quad d_{w2x} := \begin{pmatrix} span \\ span \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix}$$

$$\begin{pmatrix} RL_{x_B} \\ RR_{x_B} \\ M_{x_B} \end{pmatrix} := \begin{pmatrix} R_L(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \\ R_R(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \\ Mom(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \end{pmatrix} \quad RL_{x_B} = 0 \cdot kip \quad RR_{x_B} = 0 \cdot kip \quad M_{x_B} = 0 \cdot ft \cdot kip$$

Analysis Loads: $M_x := \frac{5.2 \cdot kip}{2} \cdot 72 \cdot in$ $M_y := 0 \cdot ft \cdot kip$ $P_{ax} := 0 \cdot kip$ $V_x := Maximum(RR_{x_B}, RL_{x_B})$

AISC Design Parameters: $F_y := 50 \cdot ksi$ $L_x := span$ $L_y := span$ $L_b := 72 \cdot in$

Member Specification: $PROF := "W10X15"$

$$A := Results(PROF, L_x, K_x, L_y, K_y, L_b, C_b, C_{mx}, C_{my}, M_x, M_y, P_{ax}, V_x, Ratio, F_y, E)$$

A =

"W10X15"	" PASS"	"fa_comp (ksi) ="	0	"Fa_comp (ksi) ="	4.725	"Ix (in^4) ="	68.9
"IntActRatio ="	0.55	"fbx (ksi) ="	13.565	"Fbx (ksi) ="	24.656	"Iy (in^4) ="	2.89
"Shear Ratio ="	0	"fby (ksi) ="	0	"Fby (ksi) ="	37.5	"Fy (ksi) ="	50

The W10X15 can safely support the weight of the array (5200#/2)

3x3x1/4

Beam $B := 1$ $span := 18.5 \cdot in$ $bm_{wt} := 0 \cdot plf$

$$p_x := \begin{pmatrix} 5.2 \cdot kip \\ 0 \cdot kip \\ 0 \cdot kip \\ 0 \cdot kip \end{pmatrix} \quad d_{px} := \begin{pmatrix} 0.5 \cdot span \\ 20 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix} \quad w_{1x} := \begin{pmatrix} bm_{wt} \\ 0 \cdot klf \\ 0 \cdot klf \\ 0 \cdot klf \end{pmatrix} \quad d_{w1x} := \begin{pmatrix} 0 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix} \quad w_{2x} := \begin{pmatrix} bm_{wt} \\ 0 \cdot klf \\ 0 \cdot klf \\ 0 \cdot klf \end{pmatrix} \quad d_{w2x} := \begin{pmatrix} span \\ span \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix}$$

$$\begin{pmatrix} RL_{x_B} \\ RR_{x_B} \\ M_{x_B} \end{pmatrix} := \begin{pmatrix} R_L(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \\ R_R(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \\ Mom(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \end{pmatrix} \quad RL_{x_B} = 2.6 \cdot kip \quad RR_{x_B} = 2.6 \cdot kip \quad M_{x_B} = 2.004 \cdot ft \cdot kip$$

Analysis Loads: $M_x := (M_{x_B})$ $M_y := 0 \cdot ft \cdot kip$ $P_{ax} := 0 \cdot kip$ $V_x := \text{Maximum}(RR_{x_B}, RL_{x_B})$

AISC Design Parameters: $F_y := 46 \cdot ksi$ $L_x := span$ $L_y := span$ $L_b := 72 \cdot in$

Member Specification: PROF := "TS3X3X1/4"

$$A := \text{Results}(PROF, L_x, K_x, L_y, K_y, L_b, C_b, C_{mx}, C_{my}, M_x, M_y, P_{ax}, V_x, Ratio, F_y, E)$$

A =

"TS3X3X1/4"	" PASS"	"fa_comp (ksi) ="	0	"Fa_comp (ksi) ="	26.398	"Ix (in^4) ="	3.16
"IntActRatio ="	0.377	"fbx (ksi) ="	11.452	"Fbx (ksi) ="	30.36	"Iy (in^4) ="	3.16
"Shear Ratio ="	0.094	"fby (ksi) ="	0	"Fby (ksi) ="	30.36	"Fy (ksi) ="	46

The HSS3x3x1/4 can support a load of 5200#.

Weld to W10: $P_{weld} := 3 \cdot 0.92 \cdot \frac{kip}{in} \cdot 12 \cdot in = 33120\ lbf$ The weld is adequate

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Plate and Bolted Connections

Check 1/2" aluminum plate as well as connection bolts.

Bar Properties

Bar Depth: $d := 2.75 \cdot in + 1.25 \cdot in \cdot 2 = 5.25\,in$

Bar thickness: $t := 0.5in$

Aluminum alloy: Alloy := 6061-T6

Unbraced length (include K-factor): $L_{bx} := 6in$ $L_{by} := 6in$

Pipe Cross Section Properties

Area: $Area := d \cdot t = 2.625 \cdot in^2$

Elastic section modulus: $S_x := \frac{t \cdot d^2}{6} = 2.297 \cdot in^3$ $S_y := \frac{t^2 \cdot d}{6} = 0.219 \cdot in^3$

Moment of inertia: $I_x := \frac{t \cdot d^3}{12} = 6.029 \cdot in^4$ $I_y := \frac{t^3 \cdot d}{12} = 0.055 \cdot in^4$

Radius of gyration: $r_x := \sqrt{\frac{I_x}{Area}} = 1.516 \cdot in$ $r_y := \sqrt{\frac{I_y}{Area}} = 0.144 \cdot in$

Alloy Properties

Safety Factors

Buckling Constants

	Unwelded	Welded
Ultimate tensile strength:	$F_{tu} = 42 \cdot ksi$	$F_{tu} = 42 \cdot ksi$
Tensile yield strength:	$F_{ty} = 35 \cdot ksi$	$F_{ty} = 35 \cdot ksi$
Compressive yield strength:	$F_{cy} = 35 \cdot ksi$	$F_{cy} = 35 \cdot ksi$
Shear yield strength:	$F_{sy} = 21 \cdot ksi$	$F_{sy} = 21 \cdot ksi$
Modulus of elasticity:	$E_a = 10100 \cdot ksi$	

Flexure Capacity

Section F.8.1.2:

Welded or unwelded: $Weld_{F812} :=$ Unwelded

Moment capacity:

Tensile yielding: $F_{b\_F812_1} := \frac{1}{\Omega_{by}} \cdot \begin{vmatrix} 1.3 \cdot F_{ty} & \text{if } Weld_{F812} = 1 \\ 1.3 \cdot F_{tyw} & \text{otherwise} \end{vmatrix}$ $\quad F_{b\_F812_1} = 27.576 \cdot ksi$

Tensile rupture: $F_{b\_F812_2} := \frac{1}{\Omega_{bu}} \cdot \begin{vmatrix} 1.42 \cdot \frac{F_{tu}}{k_t} & \text{if } Weld_{F812} = 1 \\ 1.42 \cdot \frac{F_{tuw}}{k_t} & \text{otherwise} \end{vmatrix}$ $\quad F_{b\_F812_2} = 30.585 \cdot ksi$

Allowable flexural capacity:

$M_{n\_F812x} := \min(F_{b\_F812}) \cdot S_x$ $\quad M_{n\_F812x} = 63.34 \cdot kip \cdot in$

$M_{n\_F812y} := \min(F_{b\_F812}) \cdot S_y$ $\quad M_{n\_F812y} = 6.03 \cdot kip \cdot in$

Section F.4.1:

Welded or unwelded: $Weld_{F41} :=$ Unwelded

Moment capacity:

Compressive yielding: $F_{b\_F41_1} := \frac{1}{\Omega_{by}} \cdot \begin{vmatrix} 1.3 \cdot F_{cy} & \text{if } Weld_{F41} = 1 \\ 1.3 \cdot F_{cyw} & \text{otherwise} \end{vmatrix}$ $\quad F_{b\_F41_1} = 27.576 \cdot ksi$

Tensile yielding:

$$F_{b\_F41_2} := \frac{1}{\Omega_{by}} \cdot \begin{vmatrix} 1.3 \cdot F_{ty} & \text{if } Weld_{F41} = 1 \\ 1.3 \cdot F_{tyw} & \text{otherwise} \end{vmatrix}$$

$F_{b\_F41_2} = 27.576 \cdot ksi$

Tensile rupture:

$$F_{b\_F41_3} := \frac{1}{\Omega_{bu}} \cdot \begin{vmatrix} 1.42 \cdot \frac{F_{tu}}{k_t} & \text{if } Weld_{F41} = 1 \\ 1.42 \cdot \frac{F_{tuw}}{k_t} & \text{otherwise} \end{vmatrix}$$

$F_{b\_F41_3} = 30.585 \cdot ksi$

Allowable flexural capacity:

$M_{n\_F41x} := \min(F_{b\_F41}) \cdot S_x$ $\qquad M_{n\_F41x} = 63.34 \cdot kip \cdot in$

$M_{n\_F41y} := \min(F_{b\_F41}) \cdot S_y$ $\qquad M_{n\_F41y} = 6.03 \cdot kip \cdot in$

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Section F.4.2:

Welded or unwelded: $Weld_{F42} :=$ Unwelded

Slenderness limits:

$$S_{2\_F42} := \begin{cases} \dfrac{C_{br}}{2.3} & \text{if } Weld_{F42} = 1 \\ \dfrac{C_{brw}}{2.3} & \text{otherwise} \end{cases} = 29$$

Slenderness:

$$S_{F42x} := \frac{d}{t}\cdot\sqrt{\frac{L_{bx}}{d}} = 11.225 \qquad S_{F42y} := \frac{t}{d}\cdot\sqrt{\frac{L_{by}}{t}} = 0.33$$

Allowable flexural stress:

$$F_{b\_F42x} := \frac{1}{\Omega_{by}}\cdot \begin{cases} \dfrac{\pi^2\cdot E_a}{5.29\cdot S_{F42x}^{\ 2}} & \text{if } S_{F42x} > S_{2\_F42} \\ \text{otherwise} \\ \quad \begin{cases} B_{br} - 2.3\cdot D_{br}\cdot S_{F42x} & \text{if } Weld_{F42} = 1 \\ B_{brw} - 2.3\cdot D_{brw}\cdot S_{F42x} & \text{otherwise} \end{cases} \end{cases} \qquad F_{b\_F42x} = 30.083\cdot ksi$$

$$F_{b\_F42y} := \frac{1}{\Omega_{by}}\cdot \begin{cases} \dfrac{\pi^2\cdot E_a}{5.29\cdot S_{F42y}^{\ 2}} & \text{if } S_{F42y} > S_{2\_F42} \\ \text{otherwise} \\ \quad \begin{cases} B_{br} - 2.3\cdot D_{br}\cdot S_{F42y} & \text{if } Weld_{F42} = 1 \\ B_{brw} - 2.3\cdot D_{brw}\cdot S_{F42y} & \text{otherwise} \end{cases} \end{cases} \qquad F_{b\_F42y} = 40.193\cdot ksi$$

Allowable flexural capacity:

$$M_{n\_F42x} := F_{b\_F42x}\cdot S_x \qquad M_{n\_F42x} = 69097\cdot lbf\cdot in$$

$$M_{n\_F42y} := F_{b\_F42y}\cdot S_y \qquad M_{n\_F42y} = 8792\cdot lbf\cdot in$$

Flexural Summary:

Controlling flexural capacity:

$$M_{nx\_bar} := \min\left(M_{n\_F812x}, M_{n\_F41x}, M_{n\_F42x}\right) \qquad M_{nx\_bar} = 63338\cdot lbf\cdot in$$

$$M_{ny\_bar} := \min\left(M_{n\_F812y}, M_{n\_F41y}, M_{n\_F42y}\right) \qquad M_{ny\_bar} = 6032\cdot lbf\cdot in$$

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Controlling flexural stress: $F_{bx\_bar} := \dfrac{M_{nx\_bar}}{S_x}$ $\quad F_{bx\_bar} = 27.576 \cdot ksi$

$F_{by\_bar} := \dfrac{M_{ny\_bar}}{S_y}$ $\quad F_{by\_bar} = 27.576 \cdot ksi$

Shear capacity

Section G.2:

Welded or unwelded: $Weld_{G2} :=$ Unwelded

Slenderness limits:

$$S_{1\_G2} := \begin{cases} \dfrac{B_s - F_{sy}}{1.25 \cdot D_s} & \text{if } Weld_{G2} = 1 \\ \dfrac{B_{sw} - F_{syw}}{1.25 \cdot D_{sw}} & \text{otherwise} \end{cases} = 35$$

$$S_{2\_G2} := \begin{cases} \dfrac{C_s}{1.25} & \text{if } Weld_{G2} = 1 \\ \dfrac{C_{sw}}{1.25} & \text{otherwise} \end{cases} = 63$$

d/t: $dt := \dfrac{d}{t} = 10.5$

Allowable shear stress:

$$F_{v\_G2} := \frac{1}{\Omega_v} \cdot \begin{cases} \text{if } Weld_{G2} = 1 \\ \quad \begin{cases} F_{sy} & \text{if } dt < S_{1\_G2} \\ \dfrac{1.3 \cdot \pi^2 \cdot E_a}{(1.25 \cdot dt)^2} & \text{if } dt > S_{2\_G2} \\ B_s - 1.25 \cdot D_s \cdot dt & \text{otherwise} \end{cases} \\ \text{otherwise} \\ \quad \begin{cases} F_{syw} & \text{if } dt < S_{1\_G2} \\ \dfrac{1.3 \cdot \pi^2 \cdot E_a}{(1.25 \cdot dt)^2} & \text{if } dt > S_{2\_G2} \\ B_{sw} - 1.25 \cdot D_{sw} \cdot dt & \text{otherwise} \end{cases} \end{cases}$$

$F_{v\_G2} = 12.727 \cdot ksi$

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CLARK REDER ENGINEERING

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Allowable shear capacity: $V_{n\_G2} := F_{v\_G2} \cdot d \cdot t$ $\qquad V_{n\_G2} = 33.409 \cdot kip$

Shear Summary:

Controlling shear capacity: $V_{n\_bar} := V_{n\_G2}$ $\qquad V_{n\_bar} = 33.409 \cdot kip$

Controlling shear stress: $F_{v\_bar} := \frac{V_{n\_bar}}{Area}$ $\qquad F_{v\_bar} = 12.727 \cdot ksi$

Component summary:

	Allowable Force	Allowable Stress
Moment capacity:	$M_{nx\_bar} = 63.338 \cdot kip \cdot in$	$F_{bx\_bar} = 27.576 \cdot ksi$
	$M_{ny\_bar} = 6.032 \cdot kip \cdot in$	$F_{by\_bar} = 27.576 \cdot ksi$
Shear capacity:	$V_{n\_bar} = 33.409 \cdot kip$	$F_{v\_bar} = 12.727 \cdot ksi$

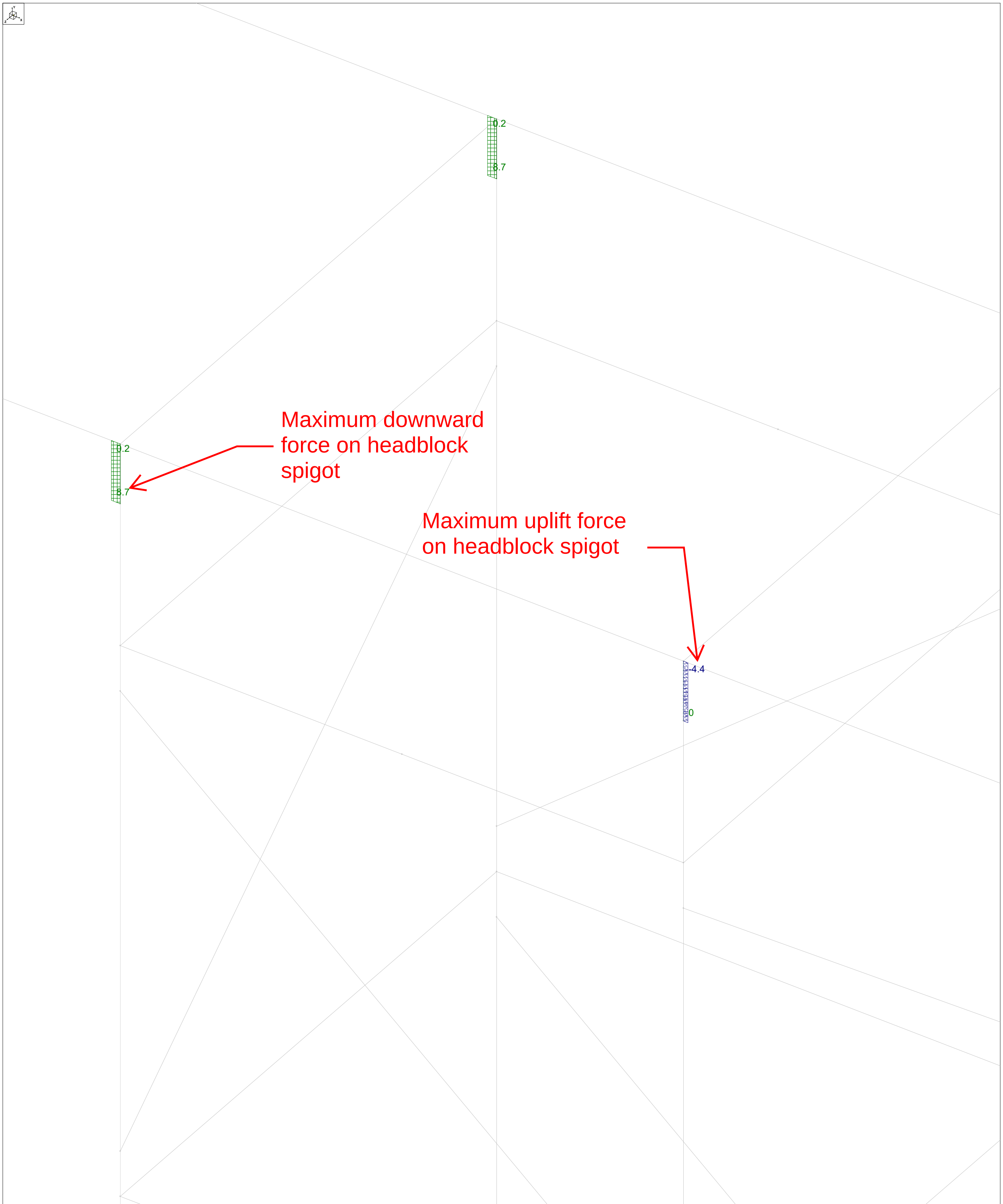
Allowable uplift load: $uplift := \frac{M_{ny\_bar}}{1.375 \cdot in} = 4387.052\ lbf$ The plate can safely resist the uplift.

Consider bolts

Capacity of 5/8" grade 8 bolts: $V_{allow\_58} := 10.4 \cdot kip$

$T_{allow\_58} := 17.3 \cdot kip$

The maximum uplift is 4.4 kips. The maximum shear is minimal. The bolts are adequate.



Member Axial Forces (k) (Enveloped)
Y-direction Reaction Units are k and k-ft (Enveloped)

Clark Reder Engineering		SK - 13
JMR	Indoor Tower	Dec 17, 2019 at 6:48 PM
		30 ft Tower Indoors.r3d

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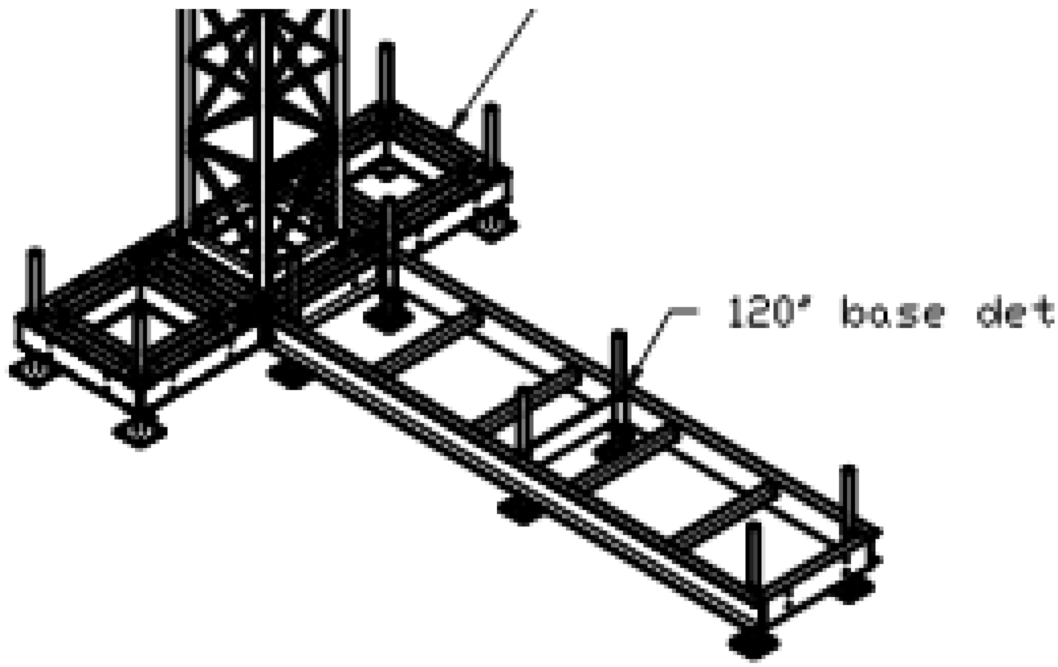
Audio Tower Base



10 foot Base

Standard Beam AISC Parameters:

$E := 29000 \cdot ksi$ $K_x := 1.0$ $K_y := 1.0$ $C_b := 1.0$ $C_{mx} := 1.0$ $C_{my} := 1.0$ $Ratio := 1.0$

Beam $B := 1$ $span := 56 \cdot in$ $bm_{wt} := 13 \cdot plf$

$$p_x := \begin{pmatrix} 5 \cdot kip \\ 0 \cdot kip \\ 0 \cdot kip \\ 0 \cdot kip \end{pmatrix} \quad d_{px} := \begin{pmatrix} 0.5 \cdot span \\ 20 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix} \quad w_{1x} := \begin{pmatrix} bm_{wt} \\ 0 \cdot klf \\ 0 \cdot klf \\ 0 \cdot klf \end{pmatrix} \quad d_{w1x} := \begin{pmatrix} 0 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix} \quad w_{2x} := \begin{pmatrix} bm_{wt} \\ 0 \cdot klf \\ 0 \cdot klf \\ 0 \cdot klf \end{pmatrix} \quad d_{w2x} := \begin{pmatrix} span \\ span \\ 0 \cdot ft \\ 0 \cdot ft \end{pmatrix}$$

$$\begin{pmatrix} RL_{x_B} \\ RR_{x_B} \\ M_{x_B} \end{pmatrix} := \begin{pmatrix} R_L(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \\ R_R(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \\ Mom(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span) \end{pmatrix} \quad RL_{x_B} = 2.53 \cdot kip \quad RR_{x_B} = 2.53 \cdot kip \quad M_{x_B} = 5.869 \cdot ft \cdot kip$$

Analysis Loads: $M_x := M_{x_B}$ $M_y := 0 \cdot ft \cdot kip$ $P_{ax} := 0 \cdot kip$ $V_x := Maximum(RR_{x_B}, RL_{x_B})$

AISC Design Parameters: $F_y := 50 \cdot ksi$ $L_x := span$ $L_y := span$ $L_b := 2 \cdot ft$

Member Specification: $PROF := \text{"C6X13"}$

$$A := Results(PROF, L_x, K_x, L_y, K_y, L_b, C_b, C_{mx}, C_{my}, M_x, M_y, P_{ax}, V_x, Ratio, F_y, E)$$

A =

"C6X13"	" PASS"	"fa_comp (ksi) ="	0	"Fa_comp (ksi) ="	13.124	"Ix (in^4) ="	17.4
"IntActRatio ="	0.405	"fbx (ksi) ="	12.142	"Fbx (ksi) ="	30	"Iy (in^4) ="	1.05
"Shear Ratio ="	0.048	"fby (ksi) ="	0	"Fby (ksi) ="	30	"Fy (ksi) ="	50

$$\delta_{location} := \frac{span}{2}$$

$$\delta := Defl\left(p_x, d_{px}, w_{1x}, w_{2x}, d_{w1x}, d_{w2x}, span, E, A_{1,8} \cdot in^4, \delta_{location}\right)$$

$\delta = 0.037 \cdot in$

$$\frac{span}{\delta} = 1533.078$$

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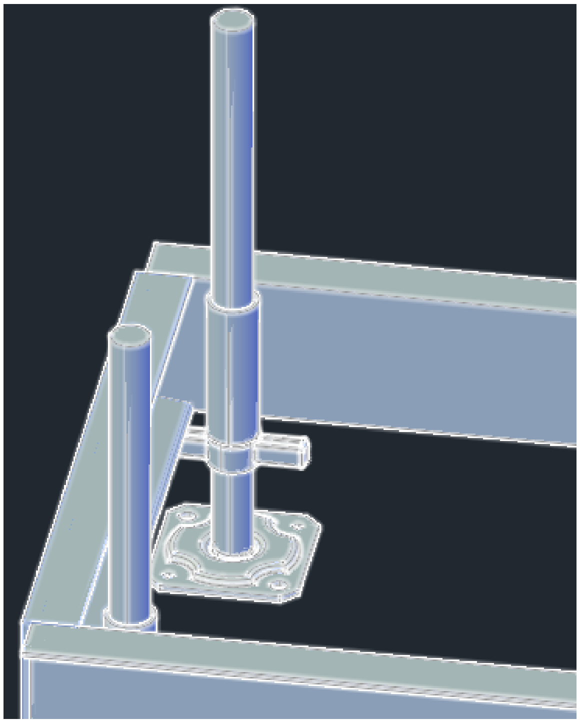
Vertical weld capacity of screw jack support



Determine the weld capacity of the vertical support:

Weld strength: $P_{weld} := 3 \cdot 0.92 \cdot \frac{kip}{in} \cdot 6 \cdot in \cdot 2 = 33.12 \cdot kip$

The screw jack has a miniumum capacity of 12,500#.

Moment and Shear Capacities of C6x13

Area of C6x13: $A_{c6} := 3.82 \cdot in^2$

Plastic section modulus of C6: $Zxc6 := 7.29 \cdot in^3$ $Zyc6 := 1.35 \cdot in^3$

Section modulus of C6: $Sxc6 := 5.78 \cdot in^3$ $Syc6 := 0.638 \cdot in^3$

Moment of inertia of C6: $Ixc6 := 17.3 \cdot in^4$ $Iyc6 := 1.05 \cdot in^4$

Radius of gyration: $rxc6 := 2.13 \cdot in$ $ryc6 := 0.524 \cdot in$

Depth of C6: $d_{c6} := 6 \cdot in$ $t_f := 0.343 \cdot in$ $t_w := 0.437 \cdot in$ $b_f := 2.16 \cdot in$

Material properties: $F_y := 50 \cdot ksi$

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Yielding, F2.1: $M_n := F_y \cdot Zxc6 = 30.375 \cdot ft \cdot kip$

Lateral torsional buckling, F2.2:

Length between braced points: $L_b := 2 \cdot ft$

Limiting lateral unbraced length: $L_p := 1.76 \cdot ryc6 \cdot \sqrt{\frac{E_s}{F_y}} = 22.21 \cdot in$

Lateral torsional buckling does not control.

Allowable moment in C6: $M_{a\_c6} := \frac{M_n}{1.67} = 18.189 \cdot ft \cdot kip$

Weak Axis Bending

$$M_{ay\_c6} := \frac{\min(F_y \cdot Zyc6, 1.6 \cdot F_y \cdot Syc6)}{1.67} = 2.547 \cdot ft \cdot kip$$

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Connection Tab

Safety factors used in analysis per AISC:

Bending: $\Omega_b := 1.67$ Bearing: $\Omega_{brg} := 2.0$ Pin shear: $\Omega_v := 2.0$

Tension: $\Omega_t := 1.67$ Tensile rupture: $\Omega_{tr} := 2.0$ Shear yielding: $\Omega_{sy} := 1.5$

Compression: $\Omega_c := 1.67$ Shear rupture: $\Omega_{sr} := 2.0$ Block shear: $\Omega_{bs} := 2.0$

Plate height: $h_{pl} := 5 \cdot in$

Plate thickness: $t_{pl} := 0.5 \cdot in$

Plate length: $l_{pl} := 4 \cdot in$

Hole Diameter: $d_{hole} := 0.3438 \cdot in \cdot 2 = 0.688 \cdot in$

Yield strength of plate: $Fy_{plate} := 36 \cdot ksi$

Tensile strength of plate: $Fu_{plate} := 58 \cdot ksi$

Diameter of pin: $d_{pin} := 0.625 \cdot in$

Pin yield strength: $Fy_{pin} := 120 \cdot ksi$

Pin tensile strength: $Fu_{pin} := 150 \cdot ksi$

Pin shear capacity:

The pin will be in double shear. Use Table J3.2 for nominal stress of fasteners.

Nominal shear stress: $F_{nv\_pin} := 0.5 \cdot Fu_{pin}$ $F_{nv\_pin} = 75 \cdot ksi$

Area of pin: $A_{pin} := \dfrac{\pi \cdot d_{pin}^{\,2}}{4}$ $A_{pin} = 0.307 \cdot in^2$

Available double shear capacity of pin: $V_{a\_pin} := \dfrac{F_{nv\_pin} \cdot A_{pin}}{\Omega_v}$ $V_{a\_pin} = 11.505 \cdot kip$

Bearing at Pin Hole

Clear distance between edge of hole and end of plate: $L_c := 1 \cdot in - \dfrac{d_{hole}}{2}$ $L_c = 0.656 \cdot in$

Available bearing strength:

$$Ra_{brg} := \frac{\text{if}\left(1.2 \cdot L_c \cdot t_{pl} \cdot Fu_{plate} \le 2.4 \cdot d_{pin} \cdot t_{pl} \cdot Fu_{plate}, 1.2 \cdot L_c \cdot t_{pl} \cdot Fu_{plate}, 2.4 \cdot d_{pin} \cdot t_{pl} \cdot Fu_{plate}\right)}{\Omega_{brg}}$$

$Ra_{brg} = 11.418 \cdot kip$

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Plate tension capacity

The plate will be checked for tensile yielding on gross section, tensile rupture on net section, shear yielding and shear rupture per section J4 of AISC.

Tensile yielding: $R_{ty} := \dfrac{Fy_{plate} \cdot t_{pl} \cdot h_{pl}}{\Omega_t}$ $\qquad R_{ty} = 53.892 \cdot kip$

Effective area: $A_e := \text{if}\left[t_{pl} \cdot \left[h_{pl} - \left(d_{hole} + \dfrac{1}{16} \cdot in\right)\right] \le 0.85 \cdot (t_{pl} \cdot h_{pl}), t_{pl} \cdot \left[h_{pl} - \left(d_{hole} + \dfrac{1}{16} \cdot in\right)\right], 0.85 \cdot (t_{pl} \cdot h_{pl})\right]$

Tensile rupture: $R_{tr} := \dfrac{Fu_{plate} \cdot A_e}{\Omega_{tr}}$ $\qquad R_{tr} = 61.624 \cdot kip$

Plate compression capacity

The nominal compressive strength shall be determined based on the limit state of flexural buckling.

Plate area: $A_{pl} := t_{pl} \cdot h_{pl}$

Moment of inertia of plate: $Iy_{pl} := \dfrac{1}{12} \cdot h_{pl} \cdot t_{pl}^{3}$

Radius of gyration of plate: $r_{y\_pl} := \sqrt{\dfrac{Iy_{pl}}{A_{pl}}}$ $\qquad r_{y\_pl} = 0.144 \cdot in$

Effective buckling length of female plate: $L_{eff\_pl} := 3 \cdot in$

Effective length factor: $K_{pl} := 2$

Slenderness: $kl\_r_{pl} := \dfrac{K_{pl} \cdot L_{eff\_pl}}{r_{y\_pl}}$ $\qquad kl\_r_{pl} = 41.569$

Elastic critical buckling stress: $Fe_{pl} := \dfrac{\pi^2 \cdot E_s}{kl\_r_{pl}^{2}}$ $\qquad Fe_{pl} = 165.636 \cdot ksi$

Flexural buckling stress: $Fcr_{plate} := \begin{cases} \left(0.658^{\frac{Fy_{plate}}{Fe_{pl}}} \cdot Fy_{plate}\right) & \text{if } kl\_r_{pl} \le 4.71\cdot\sqrt{\frac{E_s}{Fy_{plate}}} \\ (0.877Fe_{pl}) & \text{if } kl\_r_{pl} > 4.71\cdot\sqrt{\frac{E_s}{Fy_{plate}}} \end{cases}$

Available axial compressive strength: $P_{ca\_plate} := \frac{Fcr_{plate}\cdot A_{pl}}{\Omega_c}$ $P_{ca\_plate} = 49.206\cdot kip$

Plate Flexural Capacity - AISC F11

Plastic section modulus: $Zx_{pl} := \frac{t_{pl}\cdot h_{pl}^2}{4}$ $Zx_{pl} = 3.125\cdot in^3$

$Zy_{pl} := \frac{t_{pl}^2\cdot h_{pl}}{4}$ $Zy_{pl} = 0.313\cdot in^3$

Elastic section modulus: $Sx_{pl} := \frac{t_{pl}\cdot h_{pl}^2}{6}$ $Sx_{pl} = 2.083\cdot in^3$

$Sy_{pl} := \frac{t_{pl}^2\cdot h_{pl}}{6}$ $Sy_{pl} = 0.208\cdot in^3$

Unbraced length of plate: $L_b := 3\cdot in$

Vertical flexural capacity:

Eqn F11-1: $M_{px} := \text{if}\left(Fy_{plate}\cdot Zx_{pl} \le 1.6\cdot Sx_{pl}\cdot Fy_{plate}, Fy_{plate}\cdot Zx_{pl}, 1.6\cdot Sx_{pl}\cdot Fy_{plate}\right)$ $M_{px} = 112.5\cdot in\cdot kip$

$C_b := 1.0$

Eqn F11-2: $M_{n\_ltb} := C_b\cdot\left[1.52 - 0.274\cdot\left(\frac{L_b\cdot h_{pl}}{t_{pl}^2}\right)\cdot\frac{Fy_{plate}}{E_s}\right]\cdot Sx_{pl}\cdot Fy_{plate}$

$M_{nx\_ltb} := \text{if}\left(M_{n\_ltb} \le M_{px}, M_{n\_ltb}, M_{px}\right)$ $M_{nx\_ltb} = 112.469\cdot in\cdot kip$

$$F_{cr} := \frac{1.9 \cdot E_s \cdot C_b}{\left(\frac{L_b \cdot h_{pl}}{t_{pl}^{\,2}}\right)}$$

Eqn F11-3: $M_{nx\_s} := \text{if}\left(F_{cr} \cdot Sx_{pl} \le M_{px}, F_{cr} \cdot Sx_{pl}, M_{px}\right)$ $M_{nx\_s} = 112.5 \cdot in \cdot kip$

Plate available flexural strength:

$$M_{nx\_pl} := \begin{vmatrix} M_{px} \ \text{if} \ \frac{L_b \cdot h_{pl}}{t_{pl}^{\,2}} \le \frac{0.08 \cdot E_s}{Fy_{plate}} \\ M_{nx\_ltb} \ \text{if} \ \frac{L_b \cdot h_{pl}}{t_{pl}^{\,2}} > \frac{0.08 \cdot E_s}{Fy_{plate}} \wedge \frac{L_b \cdot h_{pl}}{t_{pl}^{\,2}} \le \frac{1.9 \cdot E_s}{Fy_{plate}} \\ M_{nx\_s} \ \text{if} \ \frac{L_b \cdot h_{pl}}{t_{pl}^{\,2}} \le \frac{1.9 \cdot E_s}{Fy_{plate}} \end{vmatrix}$$

$M_{nx\_pl} = 112.5 \cdot in \cdot kip$

Available flexural strength: $M_{ax\_pl} := \frac{M_{nx\_pl}}{\Omega_b}$ $M_{ax\_pl} = 5.614 \cdot ft \cdot kip$

Allowable vertical shear: $P_{vert\_pl} := \frac{M_{ax\_pl}}{3 \cdot in}$ $P_{vert\_pl} = 22.455 \cdot kip$

Horizontal flexural capacity

For rectangular bars bent about their rectangular axis, the limit state of lateral torsional buckling does not apply.

Eqn F11-1: $M_{py} := \text{if}\left(Fy_{plate} \cdot Zy_{pl} \le 1.6 \cdot Sy_{pl} \cdot Fy_{plate}, Fy_{plate} \cdot Zy_{pl}, 1.6 \cdot Sy_{pl} \cdot Fy_{plate}\right)$ $M_{py} = 11.25 \cdot in \cdot kip$

Available flexural strength: $M_{ay\_pl} := \frac{M_{py}}{\Omega_b}$ $M_{ay\_pl} = 6.737 \cdot in \cdot kip$

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30"x30" Base

Moment and Shear Capacities of Base Channel (built up member)

Area of built up C: $A_{bub} := 4.5 \cdot in^2$

Plastic section modulus of channel: $Zxbub := 8.625 \cdot in^3$ $Zybub := 1.78 \cdot in^3$

Section modulus of channel: $Sxbub := 6.792 \cdot in^3$ $Sybub := 0.9485 \cdot in^3$

Moment of inertia of channel: $Ixbub := 20.38 \cdot in^4$ $Iybub := 1.344 \cdot in^4$

Radius of gyration: $rxbub := 2.128 \cdot in$ $rybub := 0.5465 \cdot in$

Depth of channel: $d_{bub} := 6 \cdot in$ $t_f := 0.5 \cdot in$ $t_w := 0.5 \cdot in$ $b_f := 2 \cdot in$

Material properties: $F_y := 36 \cdot ksi$

Yielding, F2.1: $M_n := F_y \cdot Zxbub = 25.875 \cdot ft \cdot kip$

Lateral torsional buckling, F2.2:

Length between braced points: $L_b := 1.33 \cdot ft$

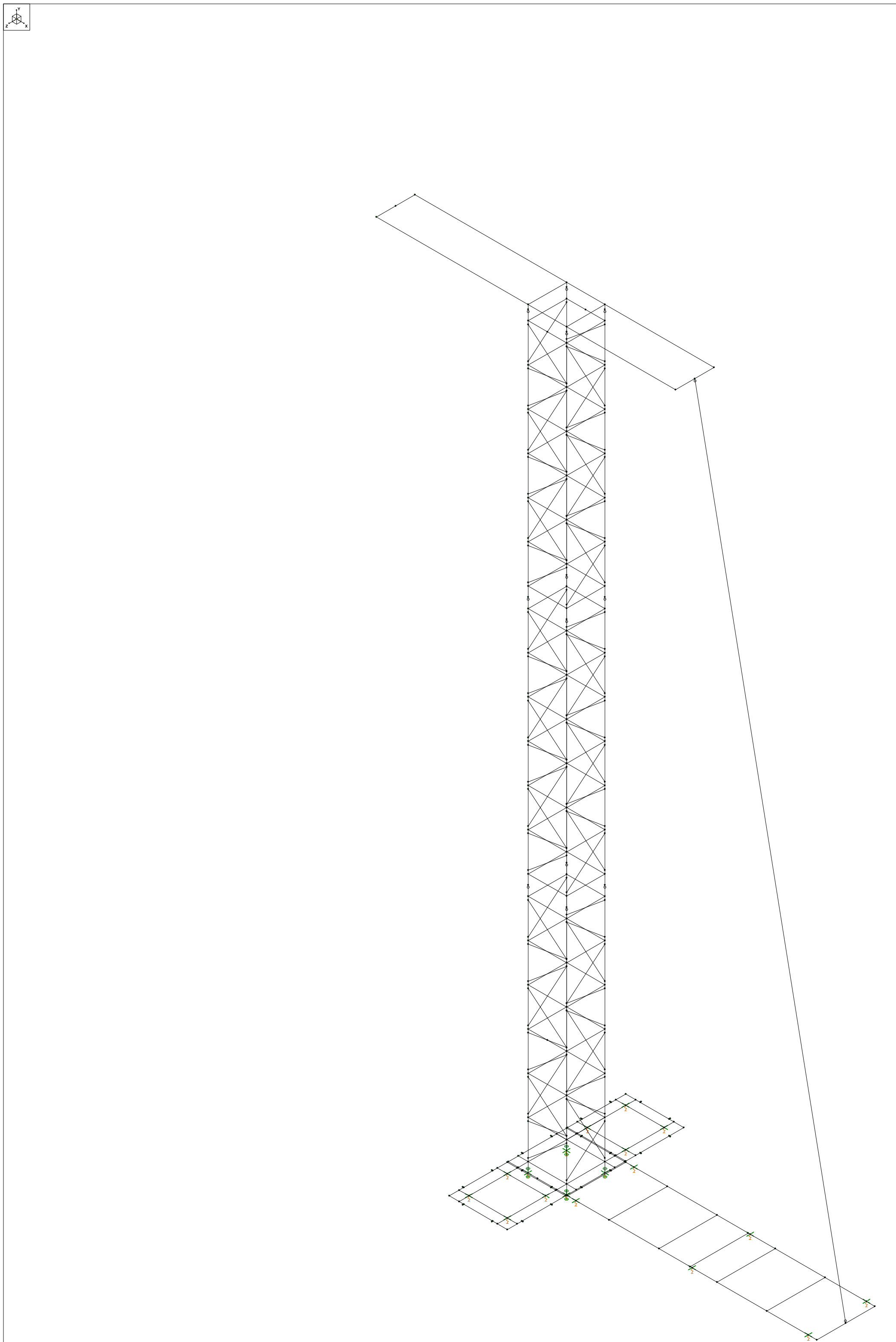
Limiting lateral unbraced length: $L_p := 1.76 \cdot rybub \cdot \sqrt{\frac{E_s}{F_y}} = 27.299 \cdot in$

Lateral torsional buckling does not control.

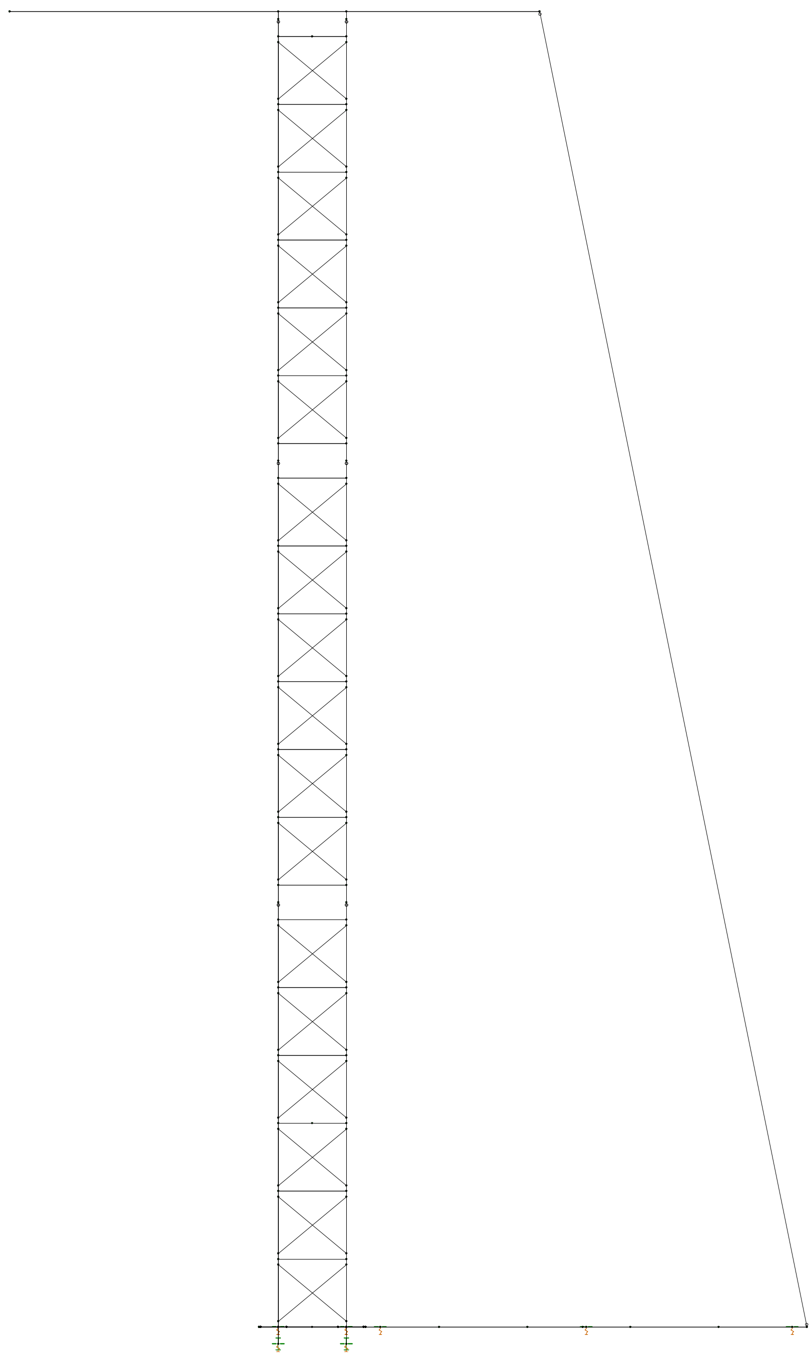
Allowable moment in channel: $M_{a\_bub} := \frac{M_n}{1.67} = 15.494 \cdot ft \cdot kip$

Weak Axis Bending

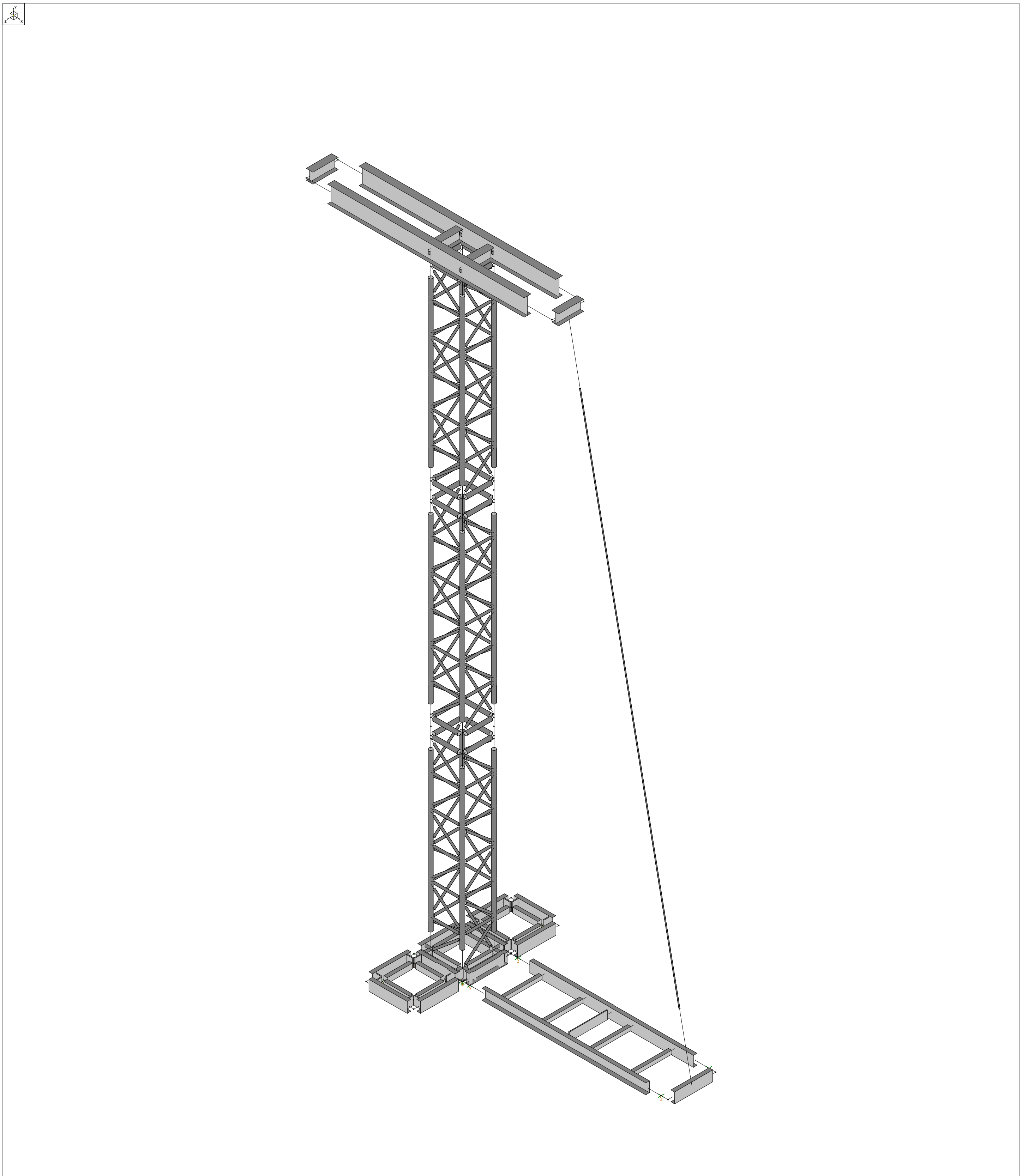
$$M_{ay\_bub} := \frac{\min\left(F_y \cdot Zybub, 1.6 \cdot F_y \cdot Sybub\right)}{1.67} = 2.726 \cdot ft \cdot kip$$



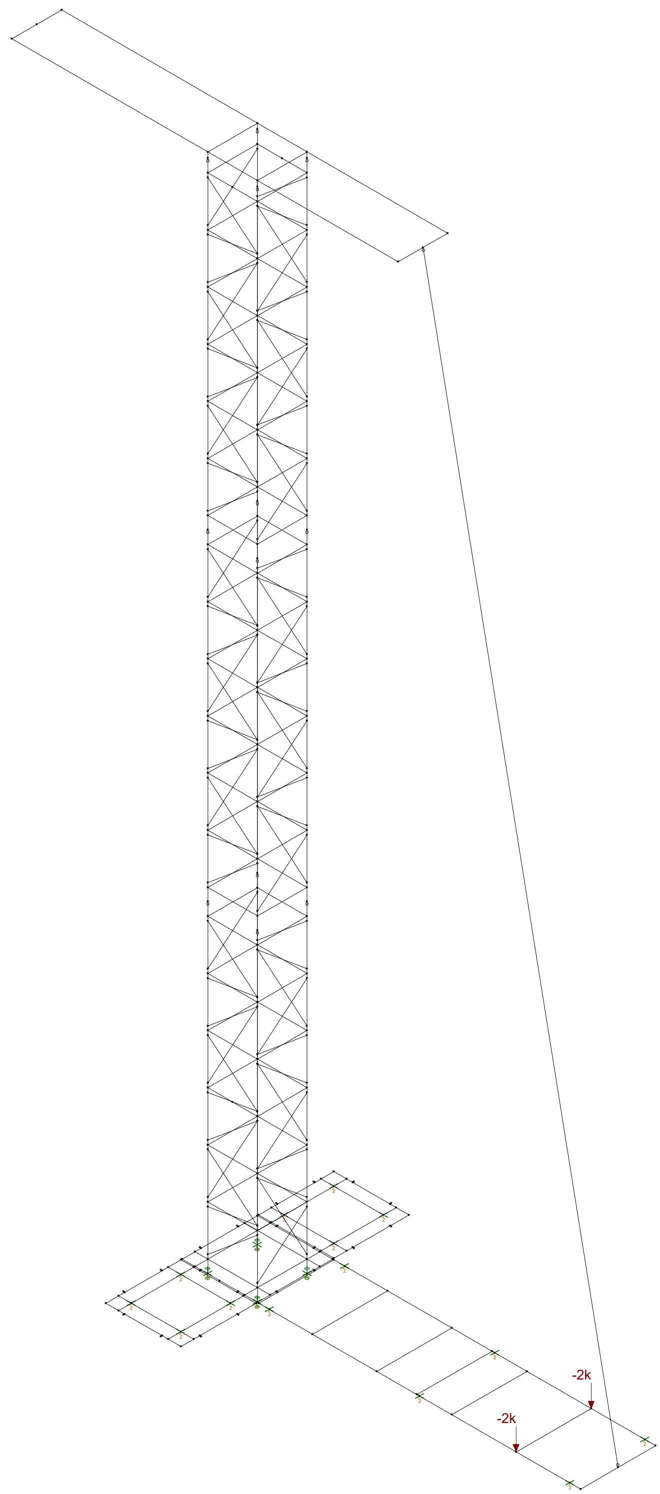
Clark Reder Engineering	Indoor Tower	SK - 1
JMR		Dec 17, 2019 at 6:21 PM
		30 ft Tower Indoors.r3d



Clark Reder Engineering	Indoor Tower	SK - 2
JMR		Dec 17, 2019 at 6:23 PM
		30 ft Tower Indoors.r3d

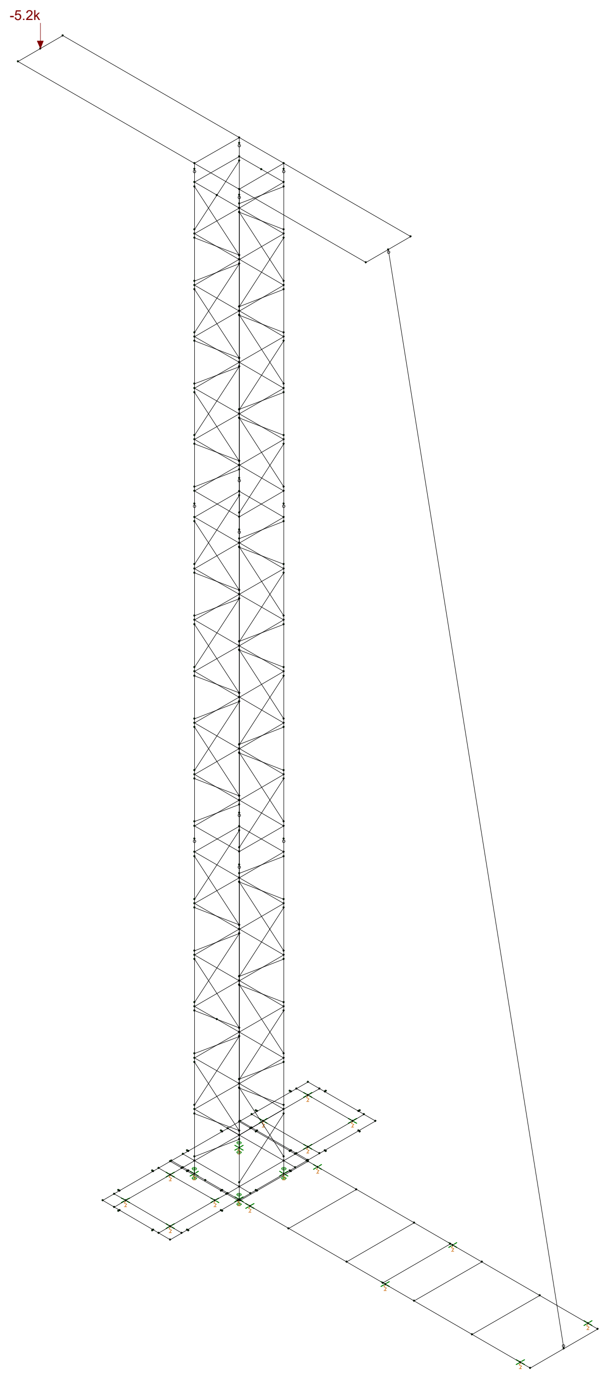


Clark Reder Engineering	Indoor Tower	SK - 3
JMR		Dec 17, 2019 at 6:36 PM
		30 ft Tower Indoors.r3d



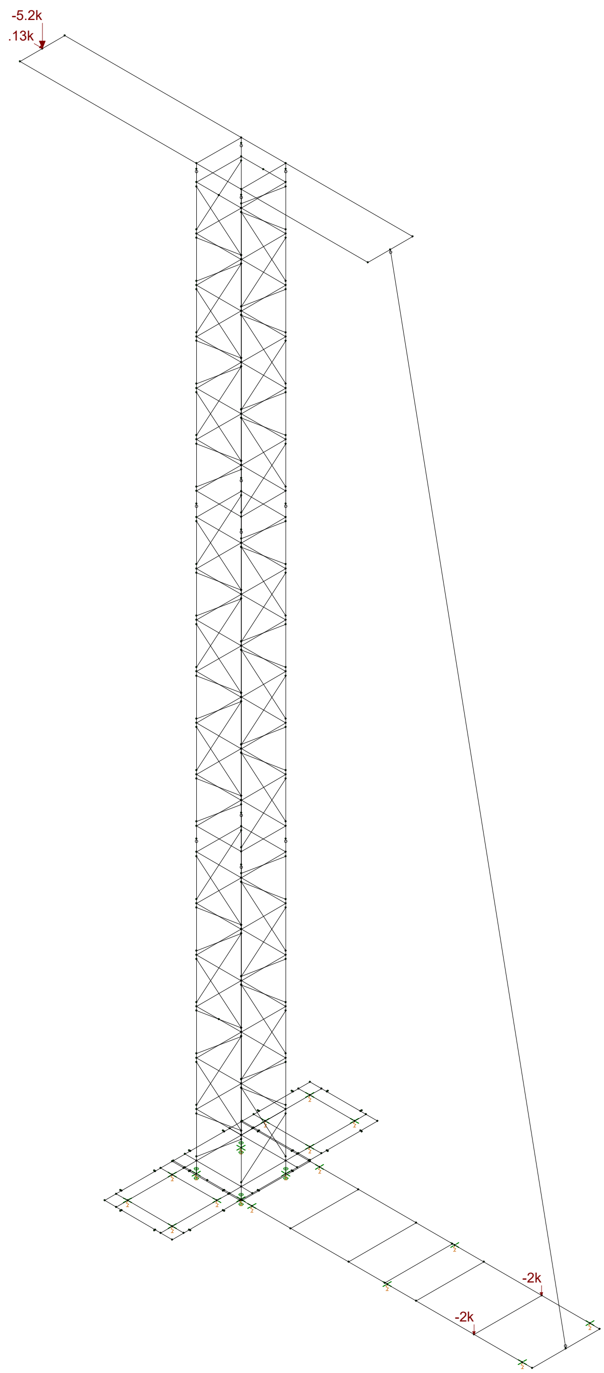
Loads: BLC 1, Dead Load

Clark Reder Engineering	Indoor Tower	SK - 4
JMR		Dec 17, 2019 at 6:37 PM
		30 ft Tower Indoors.r3d



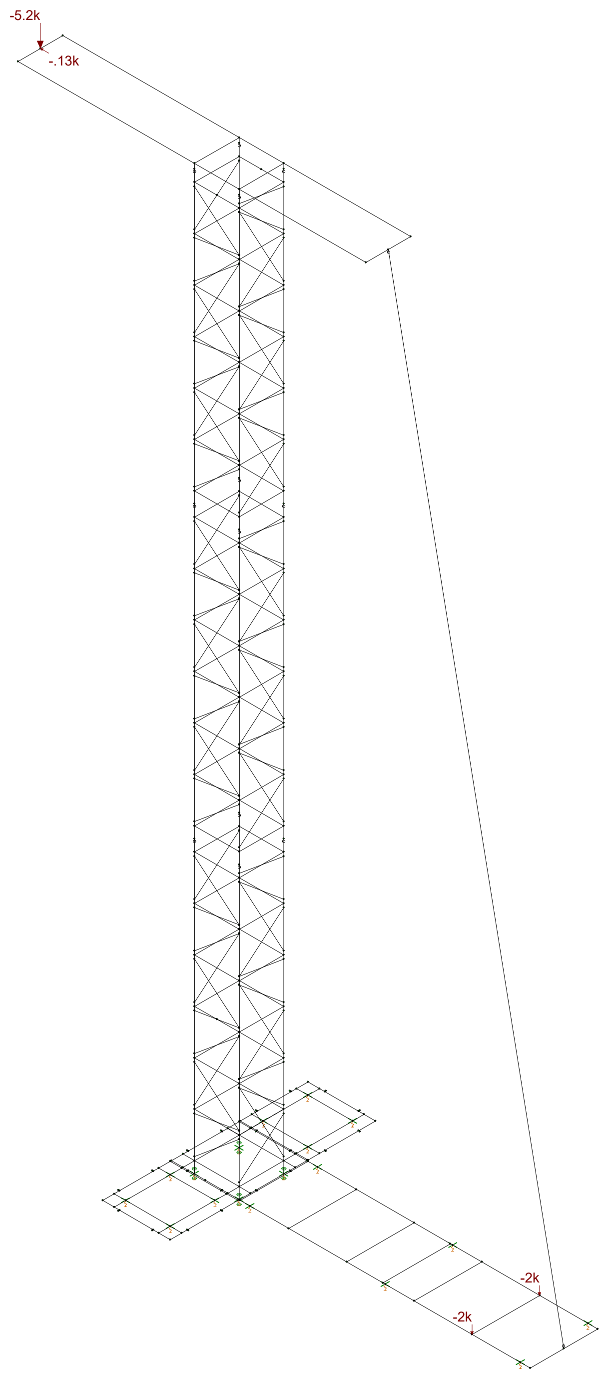
Loads: BLC 3, 5200#

Clark Reder Engineering	Indoor Tower	SK - 5
JMR		Dec 17, 2019 at 6:40 PM
		30 ft Tower Indoors.r3d



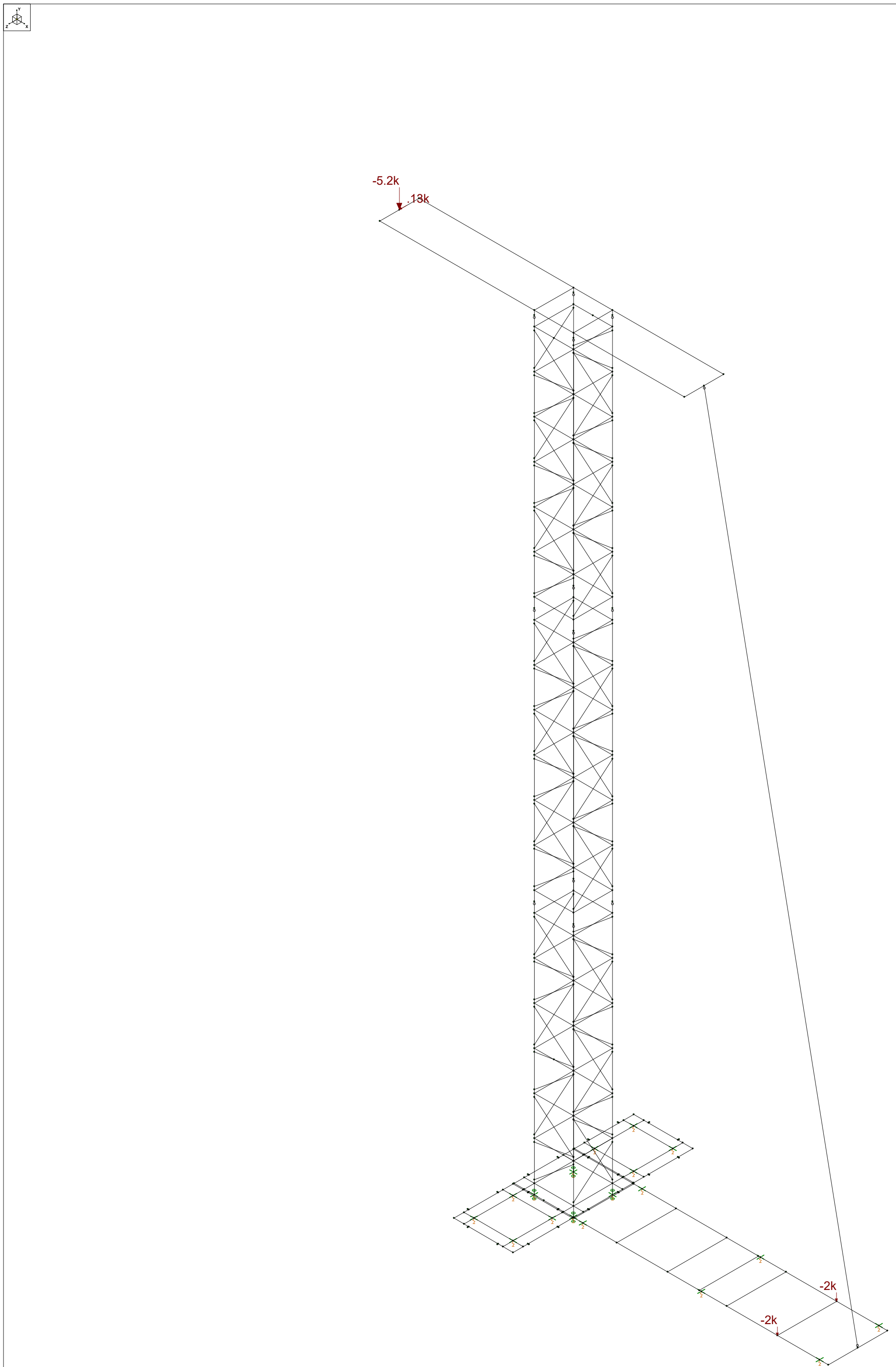
Loads: LC 3, DL+5200#+Lateral X

Clark Reder Engineering	Indoor Tower	SK - 6
JMR		Dec 17, 2019 at 6:41 PM
		30 ft Tower Indoors.r3d



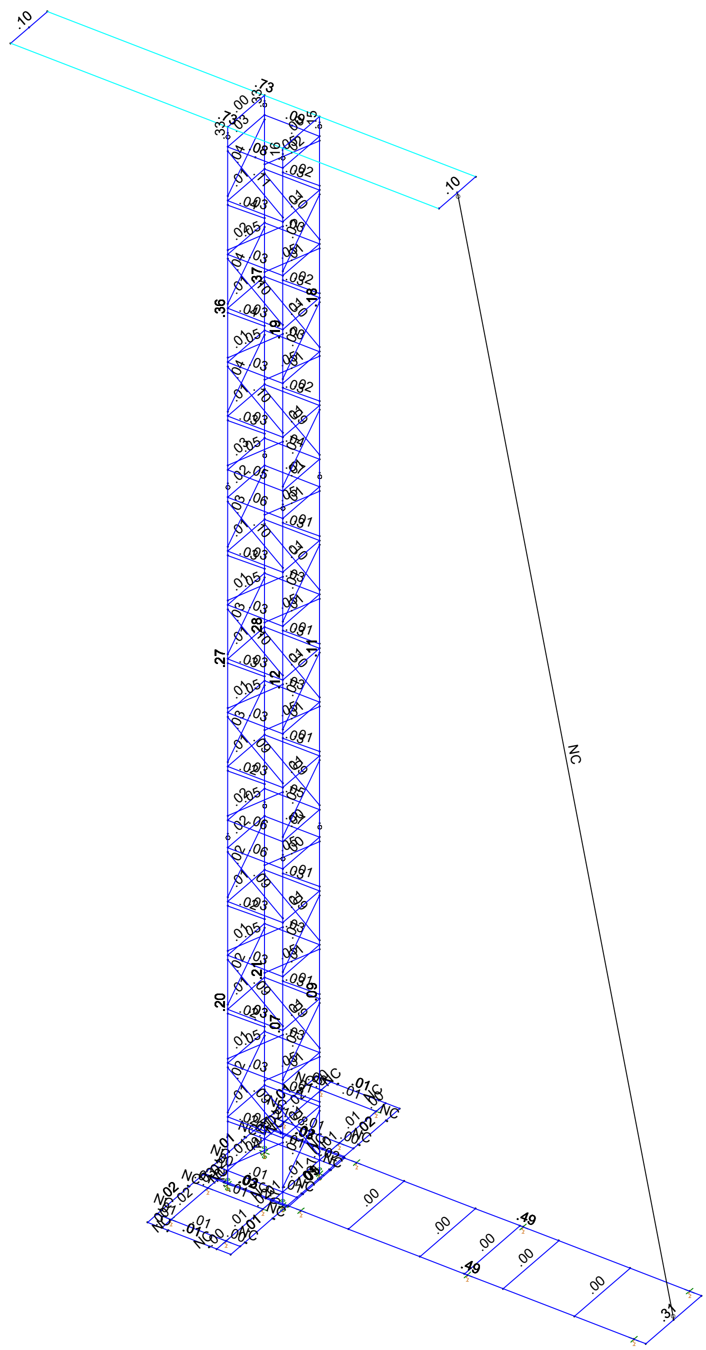
Loads: LC 4, DL+5200#-Lateral X

Clark Reder Engineering	Indoor Tower	SK - 7
JMR		Dec 17, 2019 at 6:41 PM
		30 ft Tower Indoors.r3d



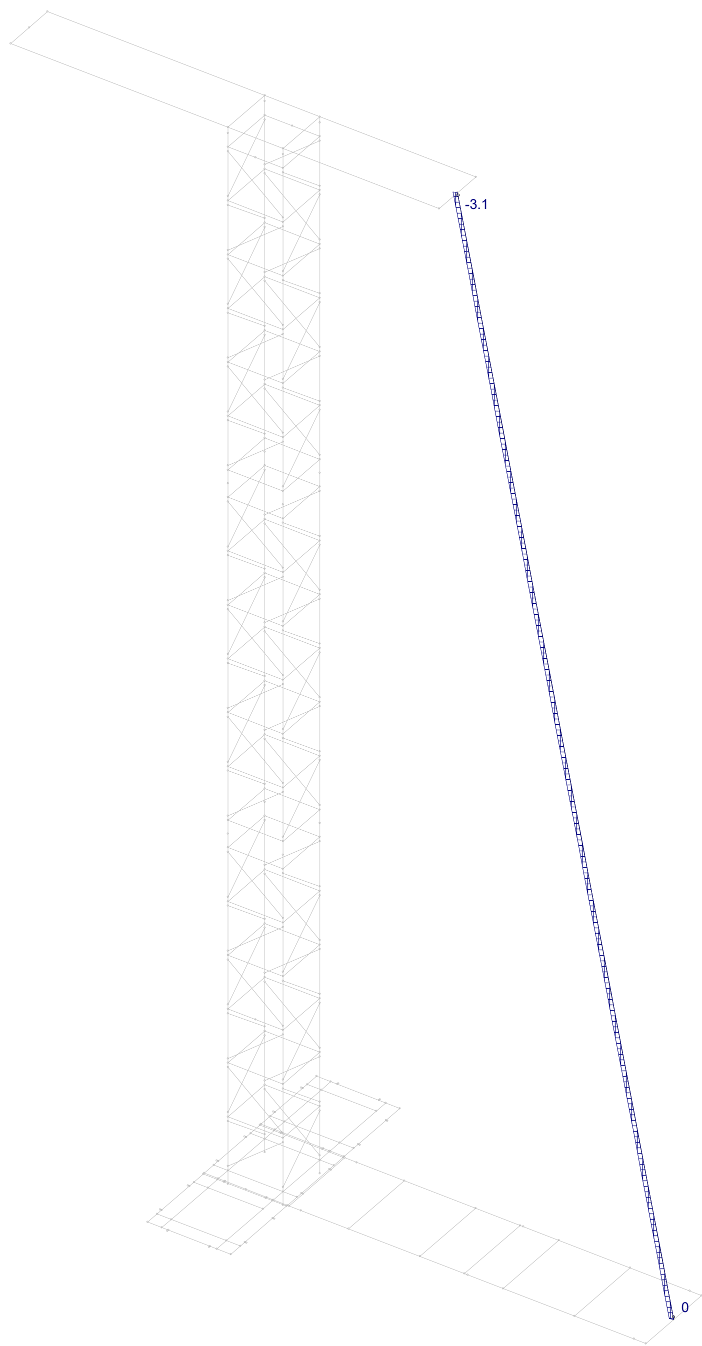
Loads: LC 5, DL+5200#+Lateral Z

Clark Reder Engineering	Indoor Tower	SK - 8
JMR		Dec 17, 2019 at 6:42 PM
		30 ft Tower Indoors.r3d



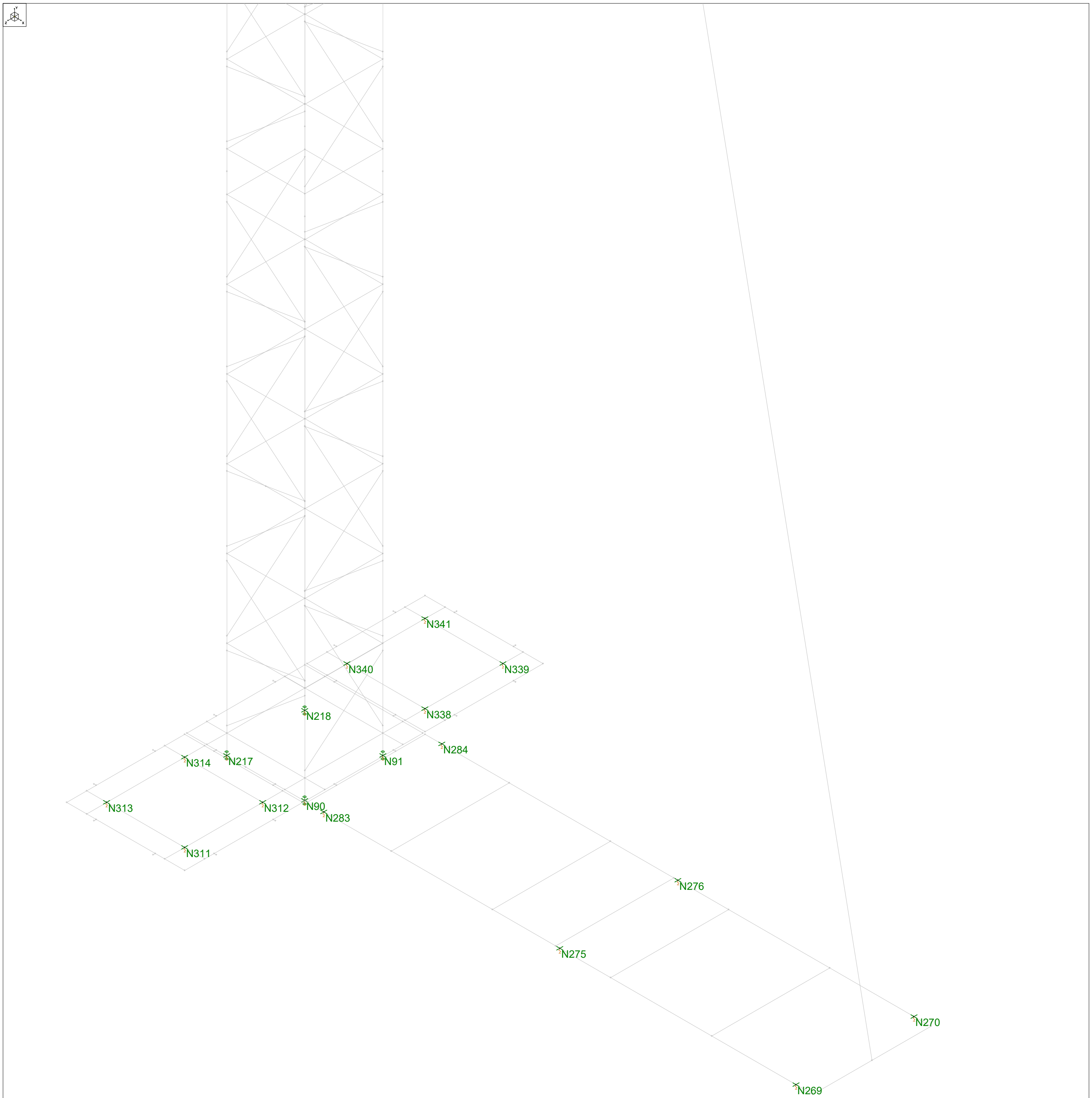
Member Code Checks Displayed (Enveloped)
Results for LC 1, DL

Clark Reder Engineering	Indoor Tower	SK - 9
JMR		Dec 17, 2019 at 6:43 PM
		30 ft Tower Indoors.r3d



Member Axial Forces (k) (Enveloped)

Clark Reder Engineering	Indoor Tower	SK - 10
JMR		Dec 17, 2019 at 6:43 PM
		30 ft Tower Indoors.r3d



Member Axial Forces (k) (Enveloped)

Clark Reder Engineering	Indoor Tower	SK - 12
JMR		Dec 17, 2019 at 6:45 PM
		30 ft Tower Indoors.r3d

Basic Load Cases

	BLC Description	Category	X Gravity	Y Gravity	Z Gravity	Joint	Point	Distribut..	Area(M...	Surface(...
1	Dead Load	DL		-1		2				
3	5200#	None				1				
6	Lateral 5200# X	None				1				
7	Lateral 5200# Z	None				1				

Load Combinations

	Description	So..	P...	S...	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac..	BLC	Fac...
1	DL	Yes			1	1																		
2	DL+5200#	Yes			1	1	3	1																
3	DL+5200#+Lateral X				1	1	3	1	6	.025														
4	DL+5200#-Lateral X				1	1	3	1	6	-.025														
5	DL+5200#+Lateral Z				1	1	3	1	7	.025														
6	DL+5200#-Lateral Z				1	1	3	1	7	-.025														

Joint Reactions

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
1	1	N90	-.009	.166	-.002	0	0	0
2	1	N217	-.002	.32	-.002	0	0	0
3	1	N91	-.01	.163	.002	0	0	0
4	1	N218	-.003	.32	.002	0	0	0
5	1	N269	.001	1.184	-.002	0	0	0
6	1	N270	.001	1.184	.002	0	0	0
7	1	N275	0	1.094	0	0	0	0
8	1	N276	0	1.094	0	0	0	0
9	1	N283	.004	0	.003	0	0	0
10	1	N284	.003	0	-.003	0	0	0
11	1	N311	0	.047	0	0	0	0
12	1	N312	.001	.04	0	0	0	0
13	1	N313	0	.039	-.002	0	0	0
14	1	N314	.006	.098	0	0	0	0
15	1	N338	.004	.047	0	0	0	0
16	1	N339	0	.042	-.002	0	0	0
17	1	N340	.005	.096	0	0	0	0
18	1	N341	0	.04	.002	0	0	0
19	1	Totals:	0	5.977	0			
20	1	COG (in):	X: 291.334	Y: 45.735	Z: 18.01			
21	2	N90	-.105	1.211	-.017	0	0	0
22	2	N217	-.069	2.727	-.03	0	0	0
23	2	N91	-.101	1.206	.014	0	0	0
24	2	N218	-.069	2.735	.032	0	0	0
25	2	N269	.307	0	-.457	0	0	0
26	2	N270	.307	0	.457	0	0	0
27	2	N275	0	.331	0	0	0	0
28	2	N276	0	.331	0	0	0	0
29	2	N283	-.04	.715	-.033	0	0	0
30	2	N284	-.048	.723	.045	0	0	0
31	2	N311	0	0	-.007	0	0	0
32	2	N312	-.02	.149	-.008	0	0	0
33	2	N313	.01	0	.027	0	0	0
34	2	N314	-.078	.455	-.015	0	0	0
35	2	N338	-.051	.144	-.013	0	0	0
36	2	N339	.002	0	.02	0	0	0
37	2	N340	-.056	.45	.004	0	0	0

Joint Reactions (Continued)

	LC	Joint Label	X [k]	Y [k]	Z [k]	MX [k-ft]	MY [k-ft]	MZ [k-ft]
38	2	N341	.011	0	-.018	0	0	0
39	2	Totals:	0	11.177	0			
40	2	COG (in):	X: 216.623	Y: 192.883	Z: 18.006			