STRUCTURAL ANALYSIS OF RESIDENTIAL HOME

18 MAY 2023

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1 DESIGN CRITERIA:

1.1 DESIGN CODES & REFERENCES

- 2019 Edition of the International Building Code
- 2019 Edition of the International Residential Code
- ASCE 7-16
- Steel design: AISC 360-10/05: ASD
- Cold-Formed steel: AISI S100-16w
- Seismic AISC 341-10/05
- Concrete: Reinforced Concrete Design Handbook (ACI)
- Ultimate Strength Design Handbook (ACI)

Calculation performed by SCIA Engineer 20.0

1.2 CODE CRITERIA

2019 IBC

Seismic Design Category:	**D**
Wind Speed:	**110** MPH
Wind Exposure:	**C**
Snow Load (Roof):	**0** psf

1.3 MATERIALS

Hot Rolled Steel Grade	A53 Gr.B
Cold Formed Steel Grade	A653 SS Gr33

1.4 STRUCTURAL LOADS

Ground Snow Load	**0.00 psf**
Roof Live Load	**20.00 psf**
Roof Dead Loads	
Framing	2.40 psf
Sheathing	1.55 psf
Insulation	1.05 psf
Solar panels	4 psf
Misc.	2 psf
Total roof Dead Load	**10 psf**
Wall Dead Loads	
Sheathing	2.5 psf
Insulation	1.05 psf
Drywall	2.5 psf
Misc.	4 psf
Total wall Dead Load	**10.05 psf**
Floor Dead Loads	
Finish	1.8 psf
Plywood	2.4 psf
LGS joist	2.9 psf
Misc.	4 psf
Total wall Dead Load	**11.5 psf**

LOAD SUMMARY

	Dead load (DL), psf	Live Load (LL), psf
Roof	10	20
Wall	10.05	
Floor	11.5	40

Load Combinations for strength design (per 2.3.1 ASCE 7-16)

1.4D

1.2D + 1.6L + 0.5(Lr or S or R)

1.2D + 1.6(Lr or S or R) + (L or 0.5W)

1.2D + 1.0W + L + 0.5(Lr or S or R)

0.9D + 1.0W

1.2D+1E+0.5LL

0.9D+1W

0.9D+1E

Per 2.2 ASCE 7-16

D =dead load:

L =live load

Lr =roof live load

R =rain load

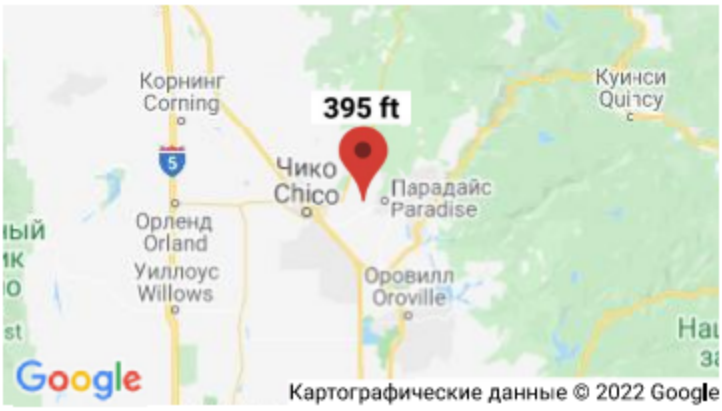
W =wind load

E =earthquake load

Wind loads

Search Information

Address:	4686 Cable Bridge Dr, Chico, CA 95928, USA
Coordinates:	39.7448017, -121.6836148
Elevation:	395 ft
Timestamp:	2022-02-18T08:06:22.228Z
Hazard Type:	Wind



ASCE 7-16

MRI 10-Year	66 mph
MRI 25-Year	71 mph
MRI 50-Year	76 mph
MRI 100-Year	81 mph
Risk Category I	88 mph
Risk Category II	94 mph
Risk Category III	101 mph
Risk Category IV	105 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category IIIHV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

WIND LOAD ON AN ARCHED ROOF

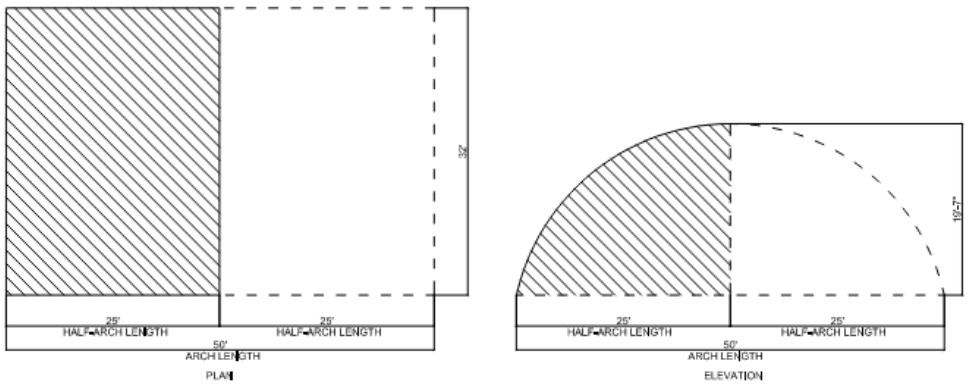
The roof of the building has the shape of a half-arch. To determine the wind load on the sections of the half-arch, we use the algorithm for determining the wind load on an arched roof with a long arch equal to 2 lengths of the half-arch.

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.03



Building data

Type of roof	Arched
Length of building	b = **32.00** ft
Width of building	d = **50.00** ft
Height to eaves	H = **1.00** ft
Radius of arch	r_{arc} = **26.1** ft
Diameter of arch	D_{arc} = **52.2** ft
Rise of arch	H_{arc} = **18.60** ft
Angle of roof at springer	α_{eaves} = **73.3** deg
Mean height	h = **10.30** ft

General wind load requirements

Basic wind speed	V = **110.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	K_e = exp(-0.0000362 ? z_g/1ft) = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Enclosed buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.18**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **-0.18**
Gust effect factor	G_f = **0.85**
Minimum design wind loading (cl.27.4.7)	p_{min_r} = **8** lb/ft^2

Topography

Topography factor not significant	K_{zt} = 1.0
Velocity pressure equation	q = 0.00256 ? K_z ? K_{zt} ? K_d ? V^2 ? 1psf/mph^2

Velocity pressures table

z (ft)	K_z (Table 26.10-1)	q_z (psf)
1.00	0.85	22.38
10.30	0.85	22.38
15.00	0.85	22.38
19.60	0.90	23.59

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.) q_i = **22.38** psf

Pressures and forces

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

Roof load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	10.30	0.32	22.38	2.12	651.46	1.38
B (-ve)	10.30	-1.07	22.38	-24.42	834.25	-20.37
C (-ve)	10.30	-0.50	22.38	-13.54	651.46	-8.82

Total vertical net force $F_{w,v}$ = **-25.55** kips

Total horizontal net force $F_{w,h}$ = **7.33** kips

Walls load case 1 - Wind 0, GC_{pi} 0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	1.00	0.80	22.38	11.19	32.00	0.36
B	10.30	-0.39	22.38	-11.40	32.00	-0.36
C	10.30	-0.70	22.38	-17.34	734.13	-12.73
D	10.30	-0.70	22.38	-17.34	734.13	-12.73

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H$ = **32.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0} = b \times H_{arc}$ = **595.29** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **5.27** kips

Leeward net force $F_l = F_{w,wB}$ = **-0.4** kips

Windward net force $F_w = F_{w,wA}$ = **0.4** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **8.1** kips

Roof load case 2 - Wind 0, GC_{pi} -0.18, $-0c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	10.30	0.32	22.38	10.18	651.46	6.63
B (-ve)	10.30	-1.07	22.38	-16.37	834.25	-13.65
C (-ve)	10.30	-0.50	22.38	-5.48	651.46	-3.57

Total vertical net force $F_{w,v}$ = **-11.53** kips

Total horizontal net force $F_{w,h}$ = **7.33** kips

Walls load case 2 - Wind 0, GC_{pi} -0.18, -0c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A	1.00	0.80	22.38	19.25	32.00	0.62
B	10.30	-0.39	22.38	-3.34	32.00	-0.11
C	10.30	-0.70	22.38	-9.29	734.13	-6.82
D	10.30	-0.70	22.38	-9.29	734.13	-6.82

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_0}$ = b x H = **32.00** ft^2

Projected vertical area of roof $A_{vert\_r\_0}$ = b x H_{arc} = **595.29** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **5.27** kips

Leeward net force $F_l = F_{w,wB}$ = **-0.1** kips

Windward net force $F_w = F_{w,wA}$ = **0.6** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **8.1** kips

Roof load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	10.30	-0.90	22.38	-21.15	344.00	-7.28
B (-ve)	10.30	-0.90	22.38	-21.15	344.00	-7.28
C (-ve)	10.30	-0.50	22.38	-13.54	687.99	-9.32
D (-ve)	10.30	-0.30	22.38	-9.74	761.19	-7.41

Total vertical net force $F_{w,v}$ = **-23.42** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Walls load case 3 - Wind 90, GC_{pi} 0.18, -c_{pe}

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	11.19	50.00	0.56
A_2	19.60	0.80	23.59	12.01	92.57	1.11
B	10.30	-0.50	22.38	-13.54	734.13	-9.94
C	10.30	-0.70	22.38	-17.34	32.00	-0.56
D	10.30	-0.70	22.38	-17.34	32.00	-0.56

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90}$ = **734.13** ft^2

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90}$ = **11.75** kips

Leeward net force $F_l = F_{w,wB}$ = **-9.9** kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2}$ = **1.7** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{11.7}$ kips

Roof load case 4 - Wind 90, GC_{pi} -0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A (-ve)	10.30	-0.90	22.38	-13.09	344.00	-4.50
B (-ve)	10.30	-0.90	22.38	-13.09	344.00	-4.50
C (-ve)	10.30	-0.50	22.38	-5.48	687.99	-3.77
D (-ve)	10.30	-0.30	22.38	-1.68	761.19	-1.28

Total vertical net force $F_{w,v} = \mathbf{-10.52}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

Walls load case 4 - Wind 90, GC_{pi} -0.18, $-c_{pe}$

Zone	Ref. height (ft)	Ext pressure coefficient c_{pe}	Peak velocity pressure q_p (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
A_1	15.00	0.80	22.38	19.25	50.00	0.96
A_2	19.60	0.80	23.59	20.07	92.57	1.86
B	10.30	-0.50	22.38	-5.48	734.13	-4.03
C	10.30	-0.70	22.38	-9.29	32.00	-0.30
D	10.30	-0.70	22.38	-9.29	32.00	-0.30

Overall loading

Projected vertical plan area of wall $A_{vert\_w\_90} = \mathbf{734.13}$ ft^2

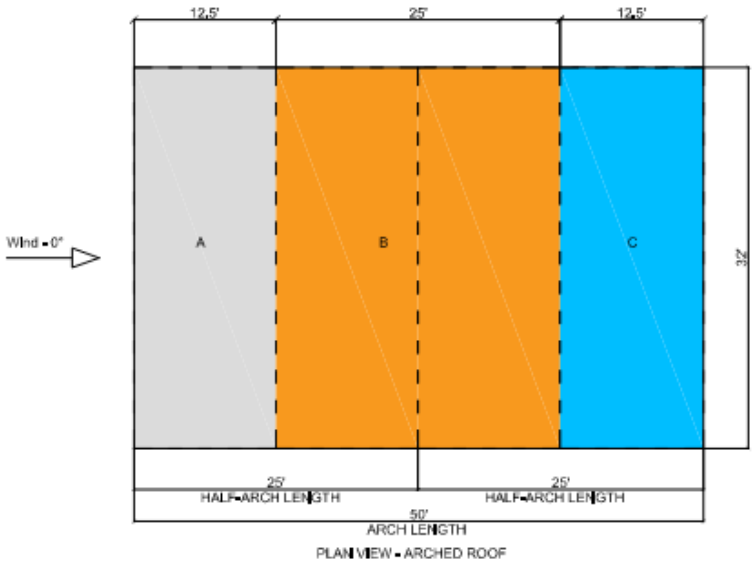
Projected vertical area of roof $A_{vert\_r\_90} = \mathbf{0.00}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{11.75}$ kips

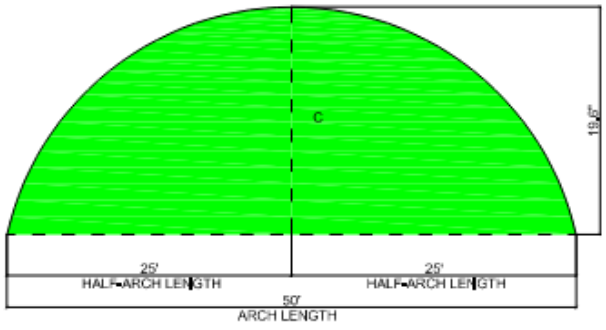
Leeward net force $F_l = F_{w,wB} = \mathbf{-4.0}$ kips

Windward net force $F_w = F_{w,wA\_1} + F_{w,wA\_2} = \mathbf{2.8}$ kips

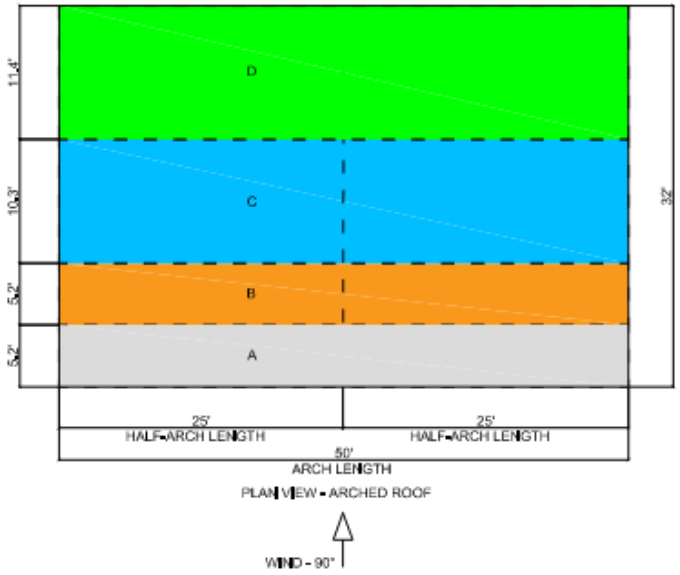
Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{11.7}$ kips



PLAN VIEW - ARCHED ROOF

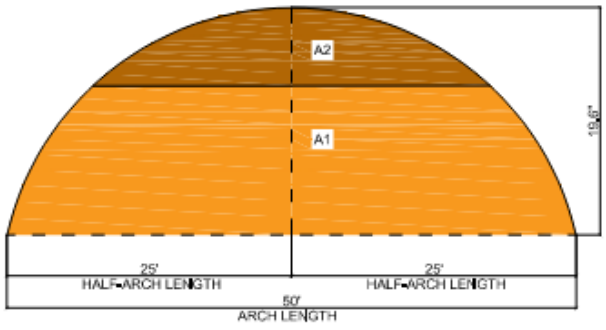


SIDE FACE

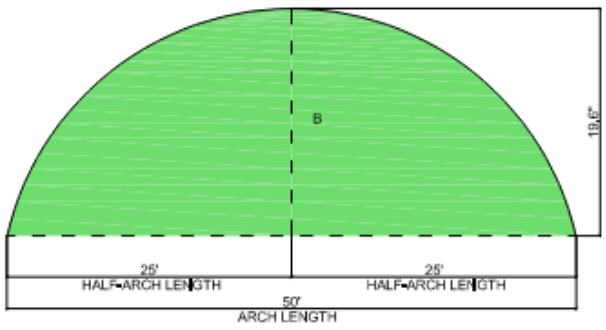


PLAN VIEW - ARCHED ROOF

WIND - 90°



WINDWARD FACE



LEEWARD FACE

WIND LOAD ON THE BALCONY

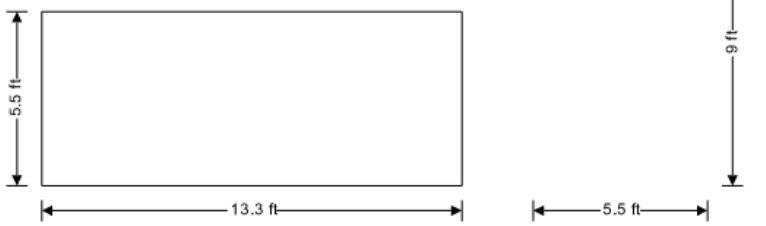
The calculation for a 13.3 x 5.5 ft structure with a 9 ft roof to determine the wind load on the cantilever part of the 2nd-floor joists 5.5 ft in length (balcony). These wind loads are taken into account when checking the floor joists for strength.

WIND LOADING

In accordance with ASCE7-16

Using the directional design method

Tedds calculation version 2.1.03



Building data

Type of roof	Monoslope free
Length of building	b = **13.25** ft
Width of building	d = **5.50** ft
Height to eaves	H = **9.00** ft
Pitch of roof	α_0 = **0.1** deg
Mean height	h = **9.00** ft
Wind flow	Clear

General wind load requirements

Basic wind speed	V = **110.0** mph
Risk category	II
Velocity pressure exponent coef (Table 26.6-1)	K_d = **0.85**
Ground elevation above sea level	z_{gl} = **0** ft
Ground elevation factor	K_e = exp(-0.0000362 x z_{gl}/1ft) = **1.00**
Exposure category (cl 26.7.3)	C
Enclosure classification (cl.26.12)	Open buildings
Internal pressure coef +ve (Table 26.13-1)	GC_{pi_p} = **0.00**
Internal pressure coef –ve (Table 26.13-1)	GC_{pi_n} = **0.00**
Gust effect factor	G_f = **0.85**

Topography

Topography factor not significant	K_{zt} = 1.0

Velocity pressure

Velocity pressure coefficient (Table 26.10-1)	K_z = **0.85**
Velocity pressure	q_h = 0.00256 x K_z x K_{zt} x K_d x K_e x V^2 x 1psf/mph^2 = **22.4** psf

Peak velocity pressure for internal pressure

Peak velocity pressure – internal (as roof press.)	q_i = **22.38** psf

Pressures and forces

Net pressure $p = q_h \times G \times C_N$

Net force $F_w = p \times A_{ref}$

Minimum design wind loading (cl.27.4.7) $p_{min\_r} = \mathbf{16}$ lb/ft^2

Roof load case 1 - Wind 0, Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	9.00	1.20	22.38	22.83	36.44	0.83
2 (+ve)	9.00	0.30	22.38	5.71	36.44	0.21

Total vertical net force $F_{w,v} = \mathbf{1.04}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_0} = b \times d \times \tan(\alpha_0) = \mathbf{0.13}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_0} = \mathbf{0.00}$ kips

Roof load case 2 - Wind 0, Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	9.00	-1.10	22.38	-20.93	36.44	-0.76
2 (+ve)	9.00	-0.10	22.38	-1.90	36.44	-0.07

Total vertical net force $F_{w,v} = \mathbf{-0.83}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_0} = b \times d \times \tan(\alpha_0) = \mathbf{0.13}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_0} = \mathbf{0.00}$ kips

Roof load case 3 - Wind 180, Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	9.00	1.20	22.38	22.83	36.44	0.83
2 (+ve)	9.00	0.30	22.38	5.71	36.44	0.21

Total vertical net force $F_{w,v} = \mathbf{1.04}$ kips

Total horizontal net force $F_{w,h} = \mathbf{0.00}$ kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_180} = b \times d \times \tan(\alpha_0) = \mathbf{0.13}$ ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_180} = \mathbf{0.00}$ kips

Roof load case 4 - Wind 180, Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	9.00	-1.10	22.38	-20.93	36.44	-0.76

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
2 (+ve)	9.00	-0.10	22.38	-1.90	36.44	-0.07

Total vertical net force $F_{w,v}$ = **-0.83** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_180} = b \times d \times \tan(\alpha_0)$ = **0.13** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_180}$ = **0.00** kips

Roof load case 5 - Wind 90, Loadcase A

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	9.00	-0.80	22.38	-15.22	49.53	-0.75
2 (+ve)	9.00	-0.60	22.38	-11.41	23.35	-0.27

Total vertical net force $F_{w,v}$ = **-1.02** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_90}$ = **0.00** kips

Roof load case 6 - Wind 90, Loadcase B

Zone	Ref. height (ft)	Ext pressure coefficient c_N	Peak velocity pressure q_h (psf)	Net pressure p (psf)	Area A_{ref} (ft^2)	Net force F_w (kips)
1 (+ve)	9.00	0.80	22.38	15.22	49.53	0.75
2 (+ve)	9.00	0.50	22.38	9.51	23.35	0.22

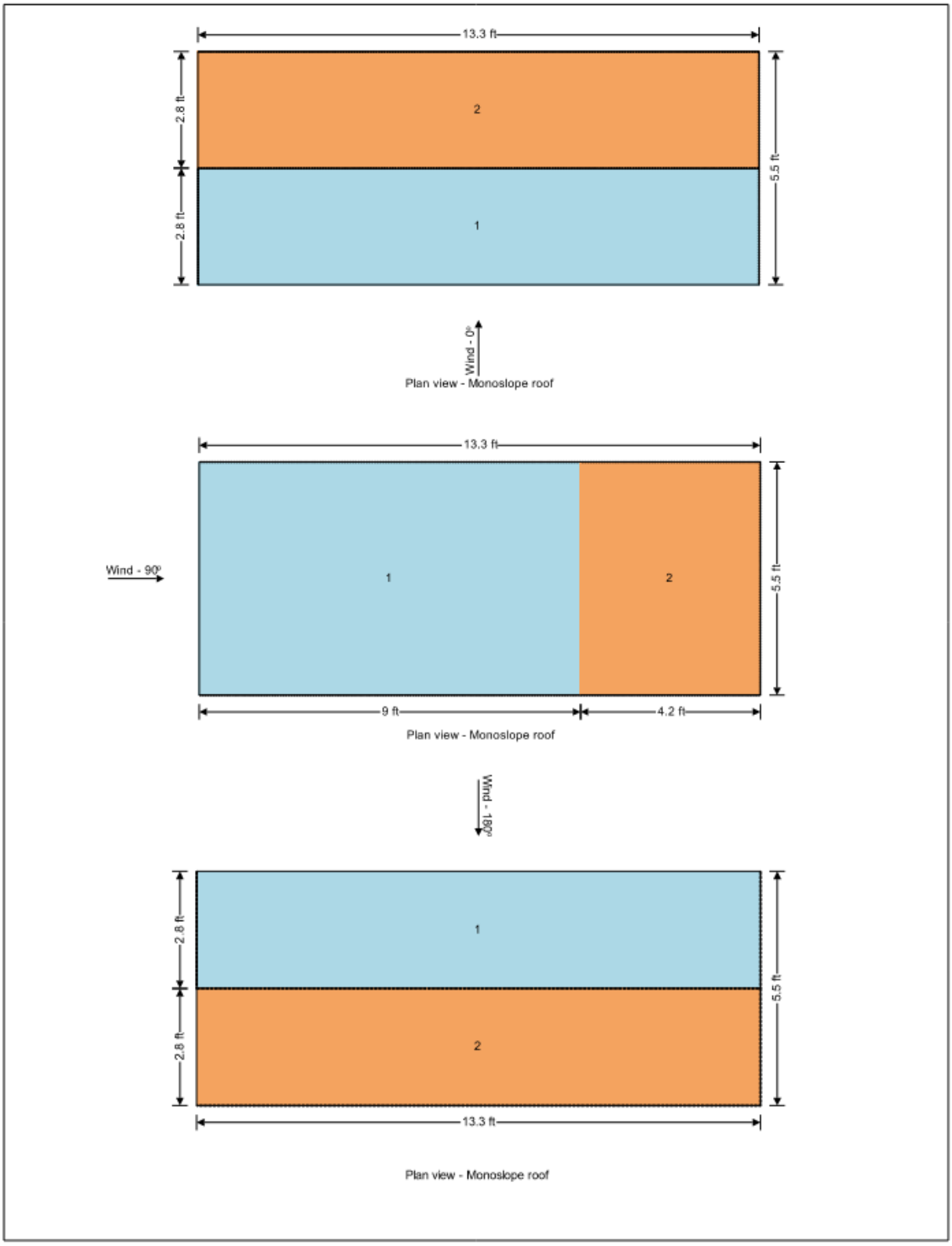
Total vertical net force $F_{w,v}$ = **0.98** kips

Total horizontal net force $F_{w,h}$ = **0.00** kips

Minimum loading

Projected vertical area of roof $A_{vert\_r\_90}$ = **0.00** ft^2

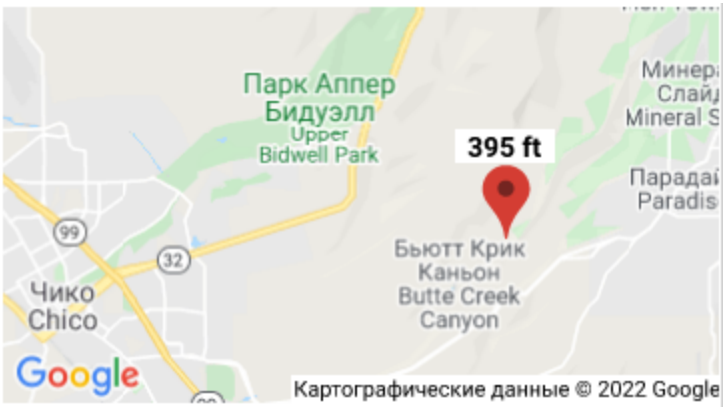
Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_r} \times A_{vert\_r\_90}$ = **0.00** kips


13.3 ft
2.8 ft
2
5.5 ft
2.8 ft
1
Wind - 0°
Plan view - Monoslope roof
13.3 ft
Wind - 90°
1
2
5.5 ft
9 ft
4.2 ft
Plan view - Monoslope roof
Wind - 180°
2.8 ft
1
5.5 ft
2.8 ft
2
13.3 ft
Plan view - Monoslope roof

SEISMIC LOADS

Search Information

Address:	4686 Cable Bridge Dr., Chico, CA 95928
Coordinates:	39.7448017, -121.6836148
Elevation:	395 ft
Timestamp:	2022-02-15T16:32:27.195Z
Hazard Type:	Seismic
Reference Document:	ASCE7-16
Risk Category:	II
Site Class:	D-default



Basic Parameters

Name	Value	Description
S_S	0.707	MCE_R ground motion (period=0.2s)
S_1	0.295	MCE_R ground motion (period=1.0s)
S_{MS}	0.873	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	0.582	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.234	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.908	Coefficient of risk (0.2s)
CR_1	0.904	Coefficient of risk (1.0s)
PGA	0.312	MCE_G peak ground acceleration
F_{PGA}	1.288	Site amplification factor at PGA
PGA_M	0.402	Site modified peak ground acceleration
T_L	16	Long-period transition period (s)
SsRT	0.707	Probabilistic risk-targeted ground motion (0.2s)
SsUH	0.779	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.295	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.327	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.5	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

2. STRUCTURAL ANALYSIS

2.1 BUILDING DESCRIPTION

The building is 32’-0" long and 23’-11" feet wide. It consists of LGS walls, and arch metal roof panels. Walls, floor joist consist of LGS structural framing. The stability of the building in the transverse direction is ensured by the support of the arch on the foundations. The stability of the building in the longitudinal direction is ensured by a vertical X-brace strap. Foundations is a ground screw.

The design of the structure is based on the requirements of the 2019 California Building Code and reference codes.

The analytical model is presented as a spatial model.

Structural rigidity of the house in longitudinal and transverse directions is achieved due to walls combination with slab-on-grade, wall sheathing, and arch metal roof panels.

Structural analysis is done in SCIA Engineer 20.0 software. This software allows automatic determination of the load combination that causes the highest forces in structural members for further analysis and cross-section selection. Governing load cases are shown in the sections "CHECKING STEEL ELEMENTS". Seismic force resisting system is A18: Light-frame (cold-formed steel) wall systems using flat strap bracing.

To determine the design forces from dynamic loads (earthquake), two types of calculations are used: Modal Mode and Response Spectrum Method

Modal Mode:
This approach allows the modal analysis of the structure, setting the first n values and eigenvectors of the structure.
The available analysis methods: subspace iteration, Lanczos method, and the basis reduction method.
Iterations will be completed if the following condition is met: where:

$$\frac{\left|\omega_i^k - \omega_i^{k-1}\right|}{\left|\omega_i^k\right|} < tolerance$$

i = 1,2,...,n vibration modes, k - number of iterations.
Upper limit is the period value (pulsation, frequency), which describes that in the range, (0, upper limit) the following values and eigenvectors will be set. Sturm check, which allows finding the skipped pulsations, is possible.

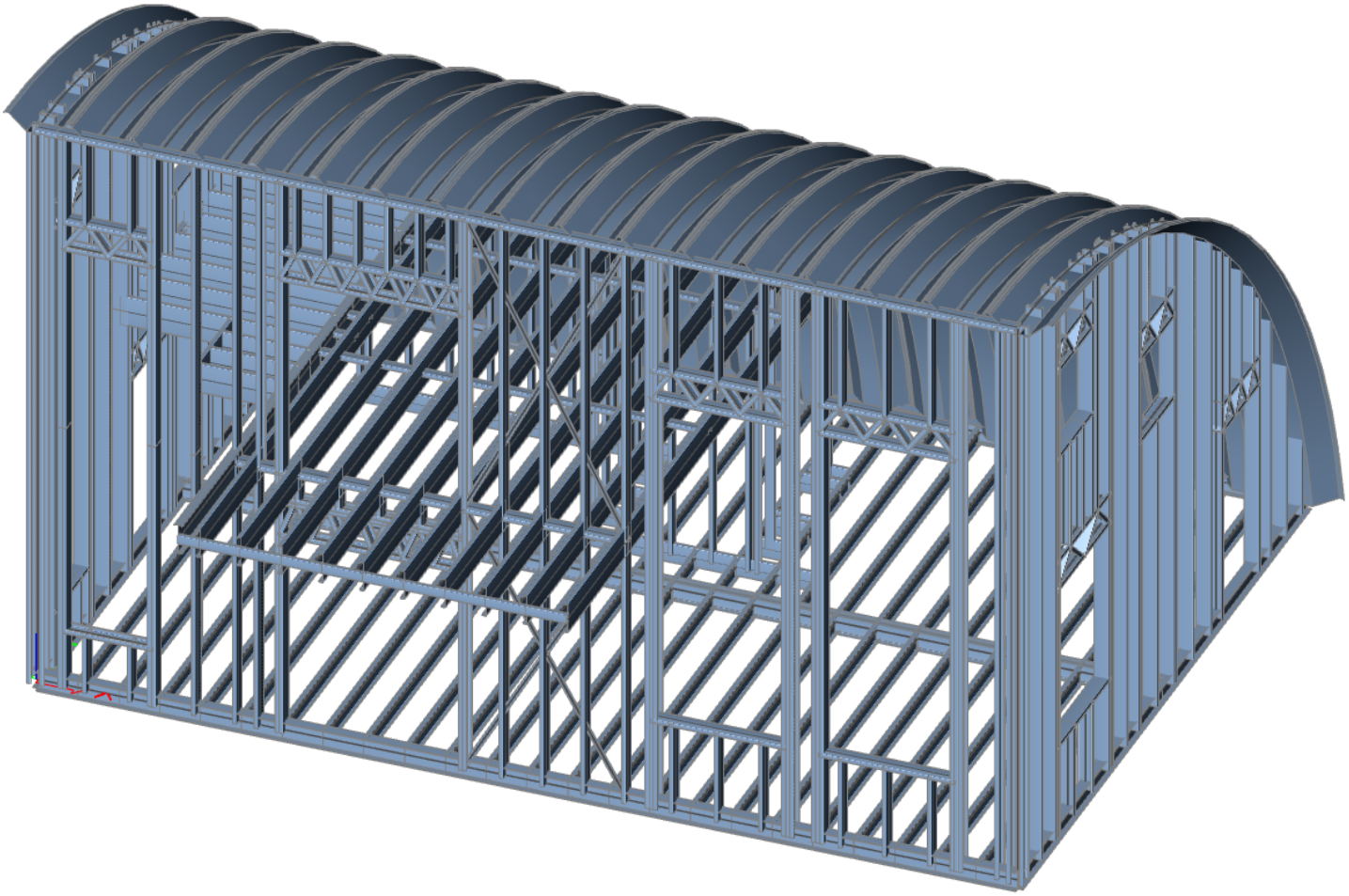
Response Spectrum Method:
Seismic analysis is based on the response spectrum method. All data is defined the same way as in modal analysis. Additionally, parameters required by a specific national code to establish the response spectrum shape must be specified. Calculations and results are the same as those for spectral analysis.
In addition to results obtained from modal analysis, for each eigenform the seismic analysis provides the following values:

- Seismic excitation multiplier (value of the accelerating excitation spectrum).
- Seismic participation factors calculated as those for the modal analysis. However, vector D describing excitation direction is user defined. Coefficients are specified for each dynamic degree of freedom according to the method selected in Job Preferences. (Maximum or Distinct).
- Seismic mode coefficients as a product of the seismic excitation factor and the respective seismic participation factor for each dynamic degree of freedom.
- Displacements, internal forces, and reactions for each form of vibration or quadratic combination calculated with the SRSS or CQC method.
- Pseudo static forces, which are the external loads generated according to the seismic analysis assumptions.

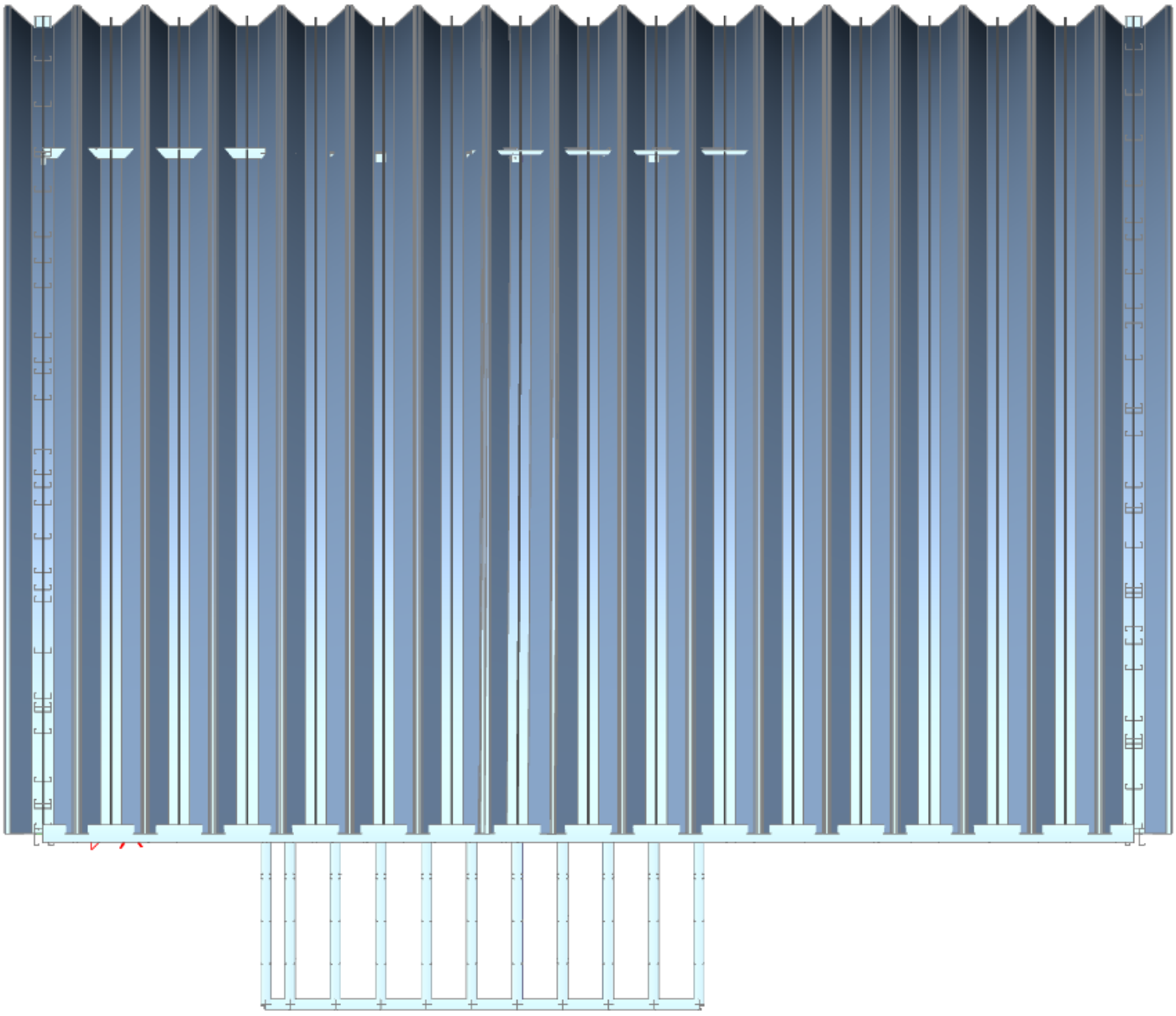
For seismic analysis, the same quadratic combination methods as those for spectral analysis are available.

2.2 GENERAL SCHEME

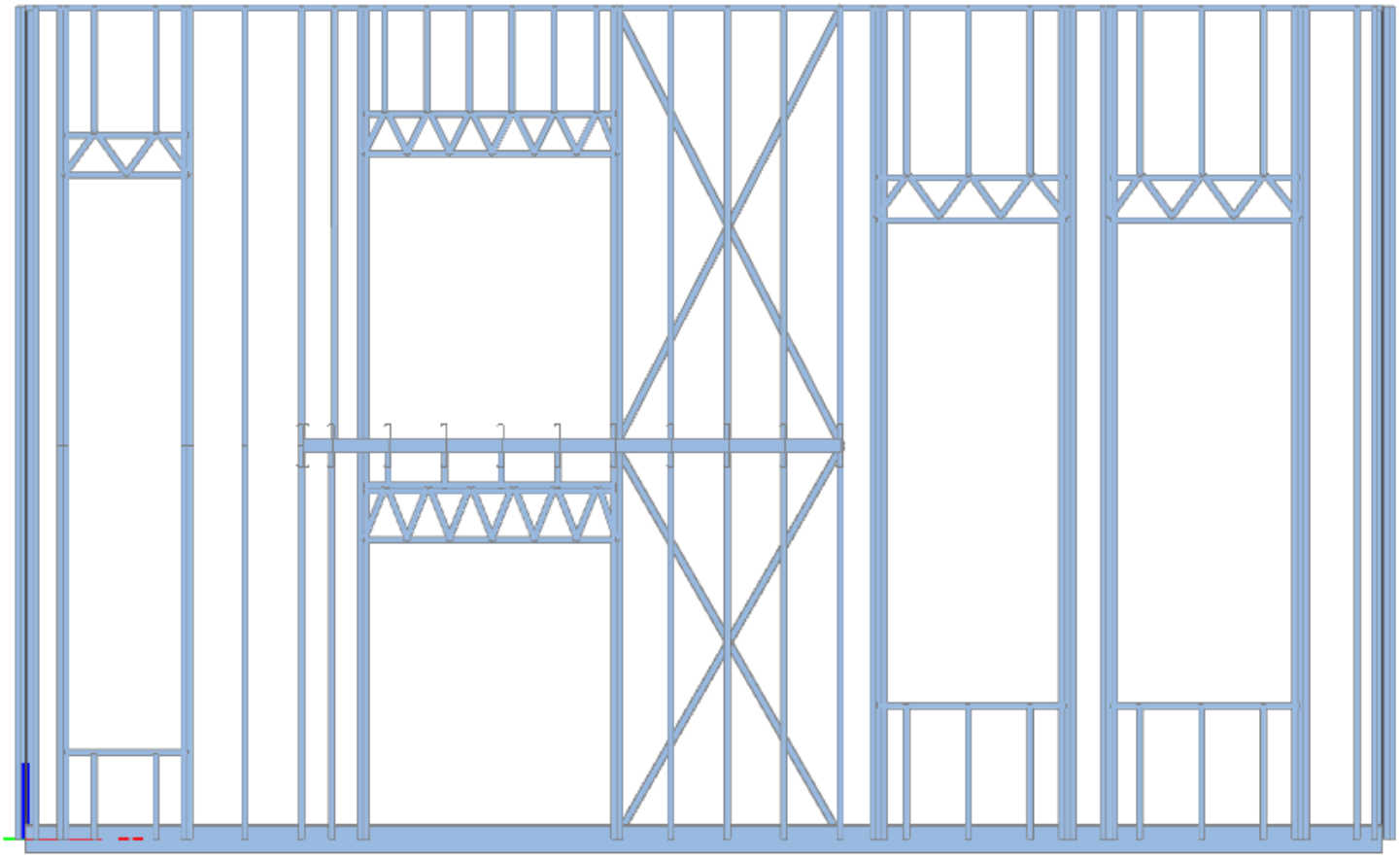
General view



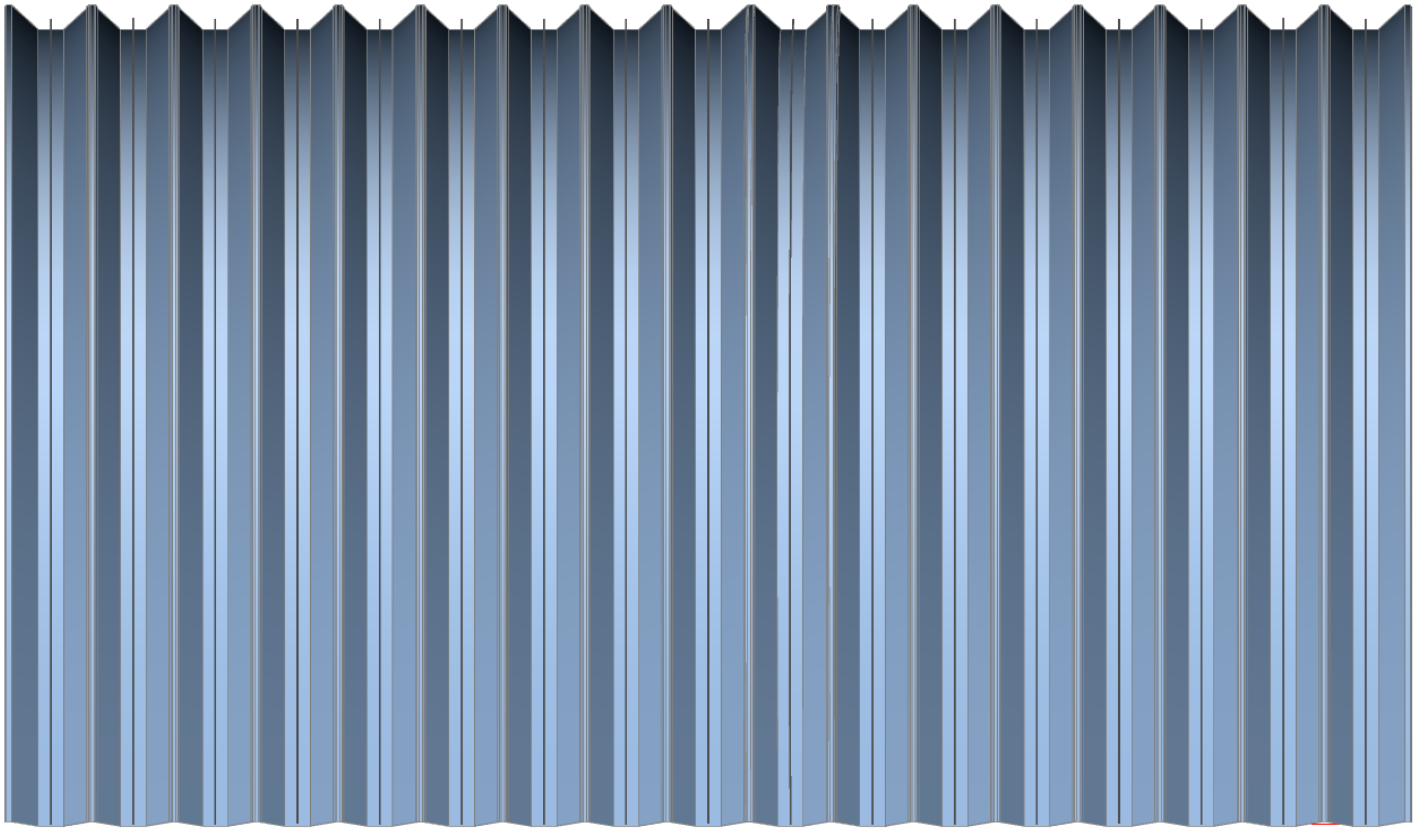
ROOF VIEW



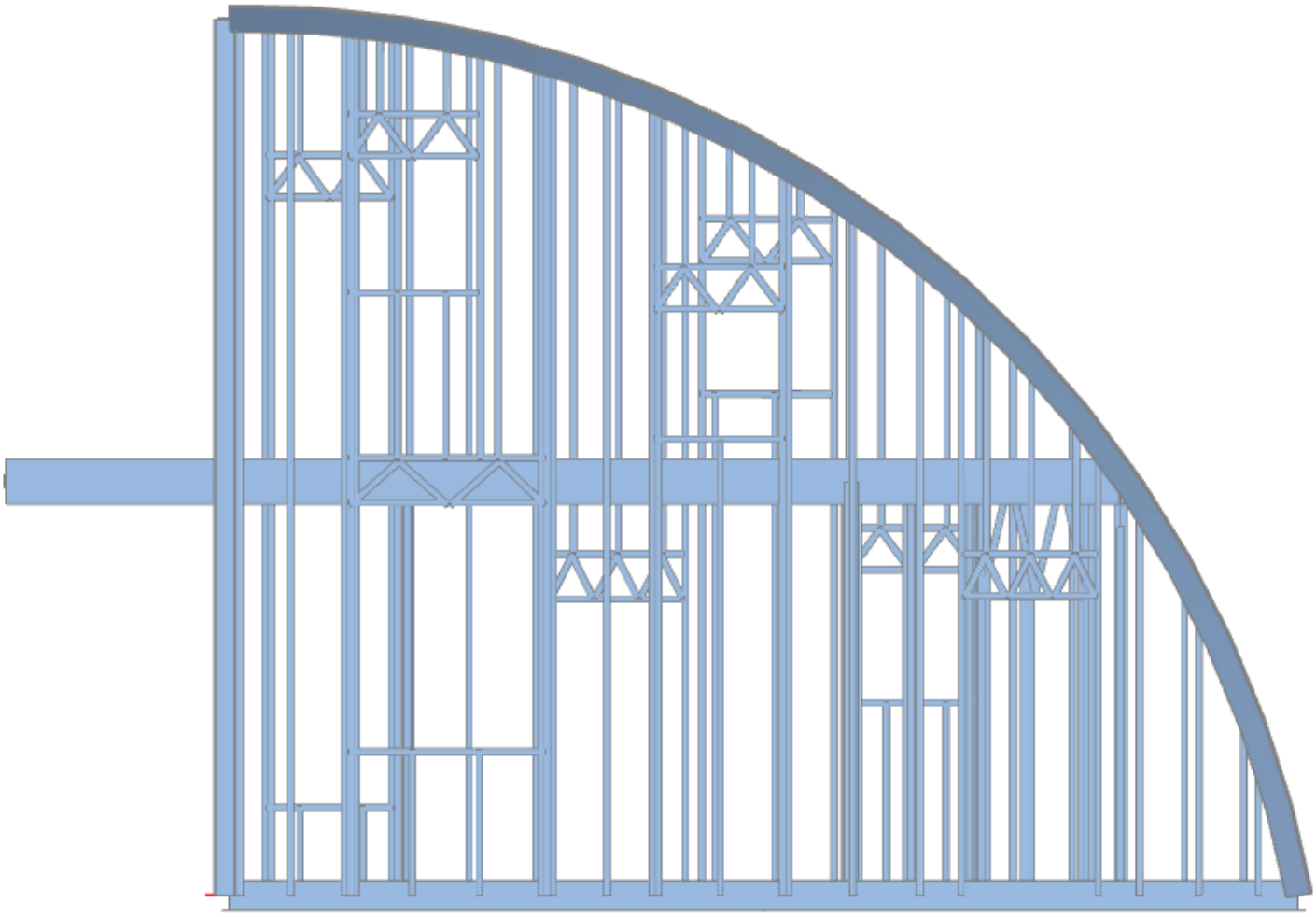
FRONT VIEW



BACK VIEW



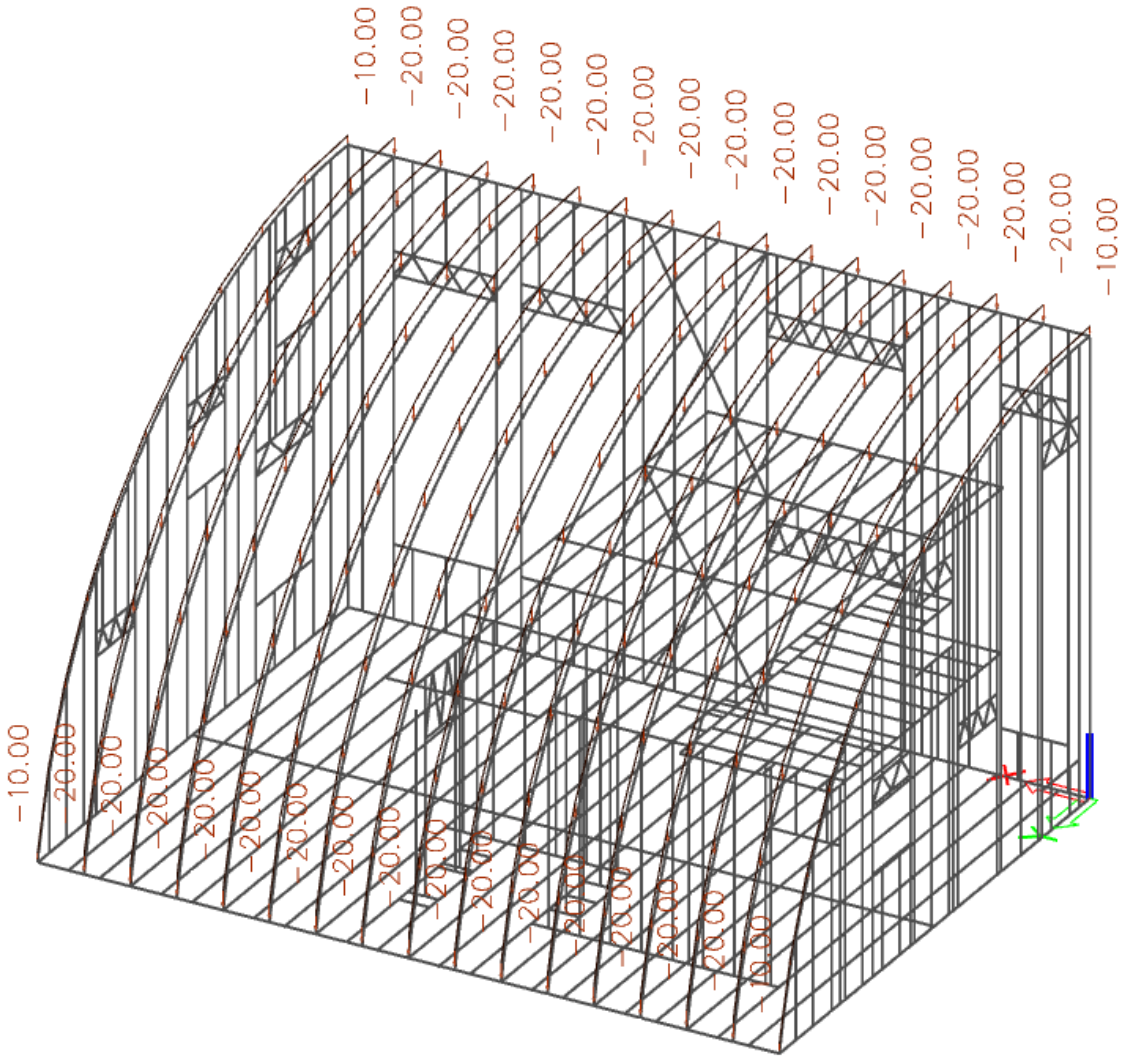
SIDE VIEW



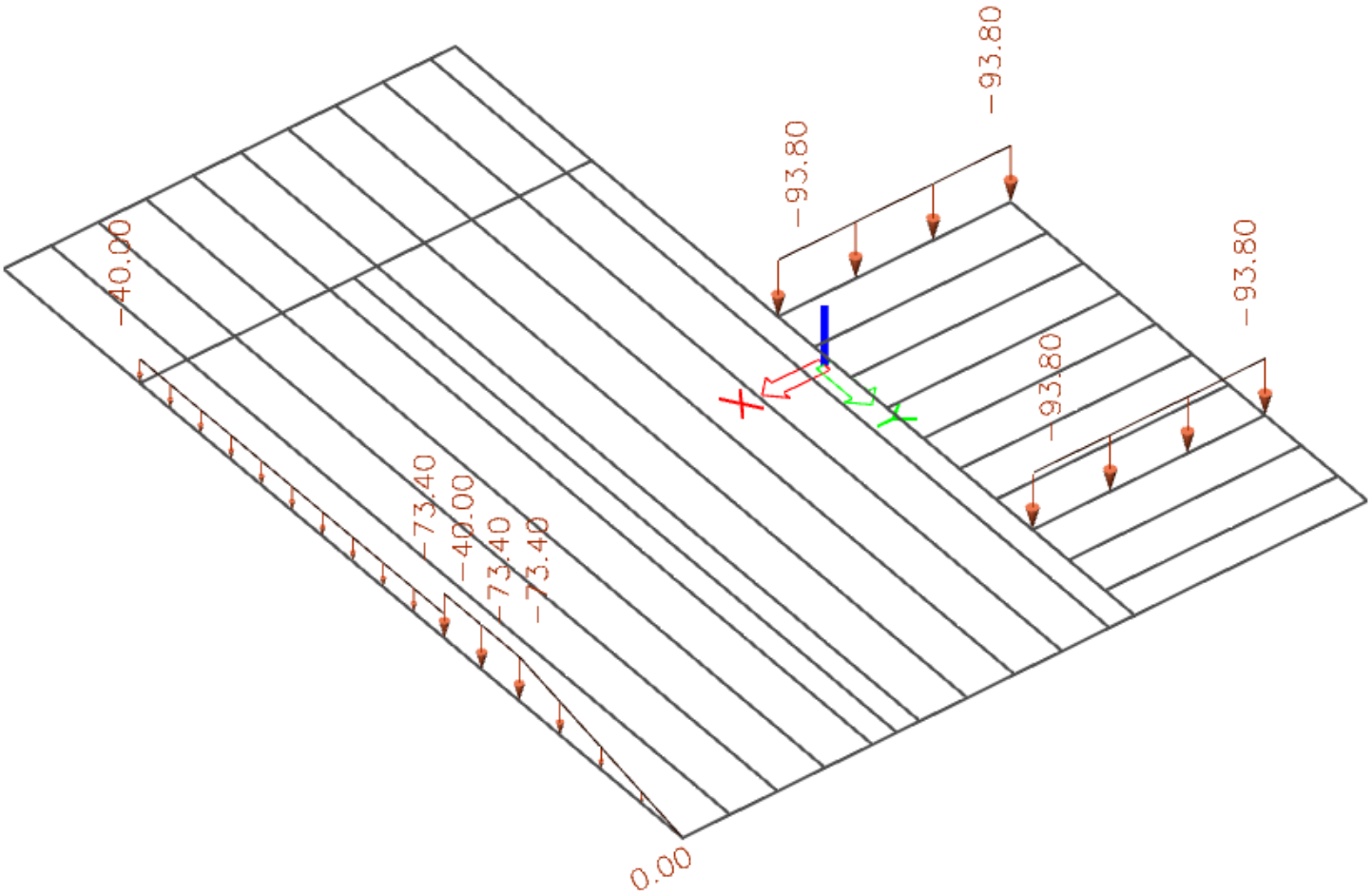
2.3 APPLIED LOADS

1. **DL1 - Dead weight of steel structures (automatically)**

2. **DL2 - Dead Load Roof**
   Dead loads distribution
   DL = 10 psf

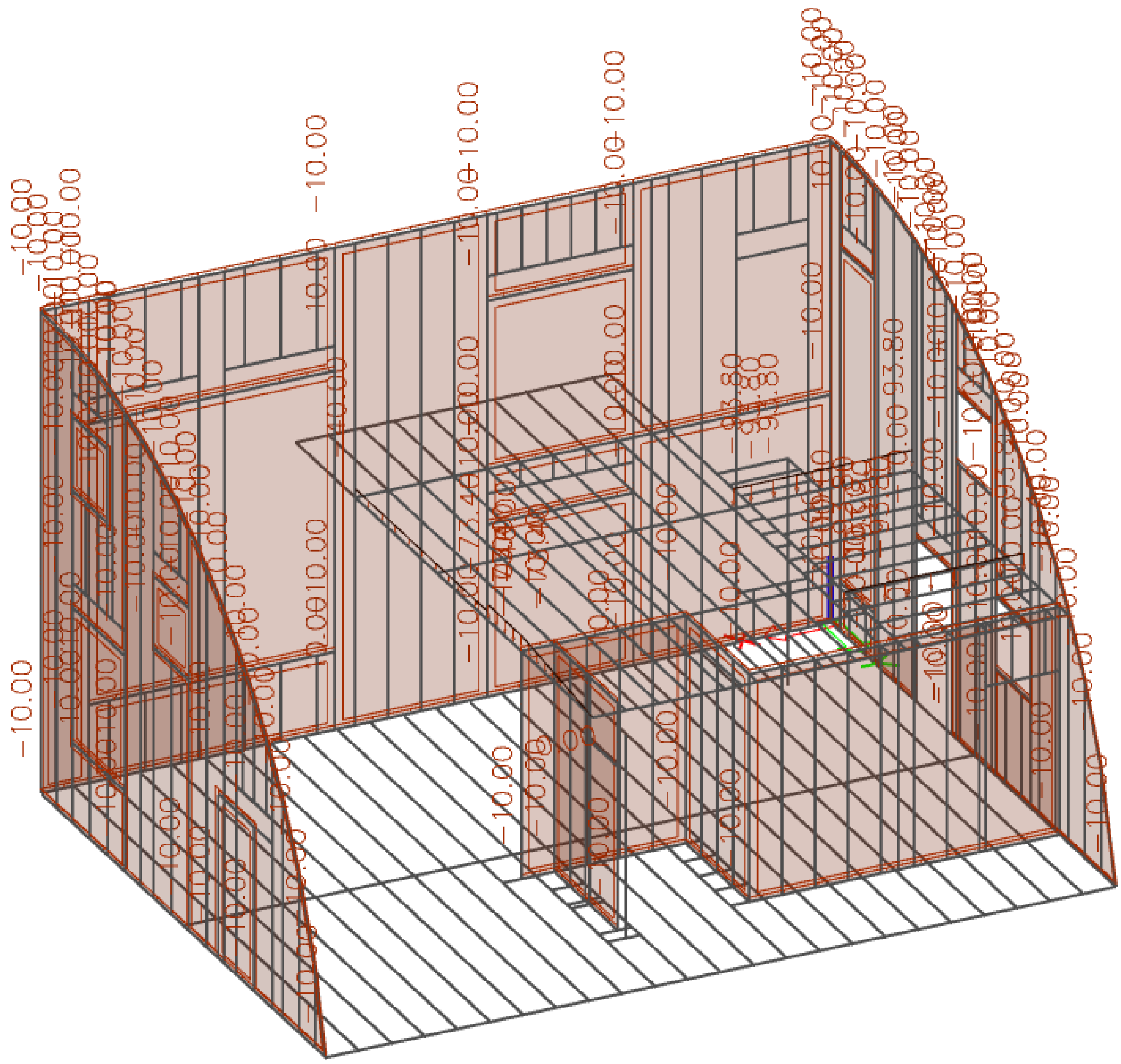


3. **DL3 - Dead Load Wall**
   Dead loads distribution interior walls
   DL = 10.0 psf

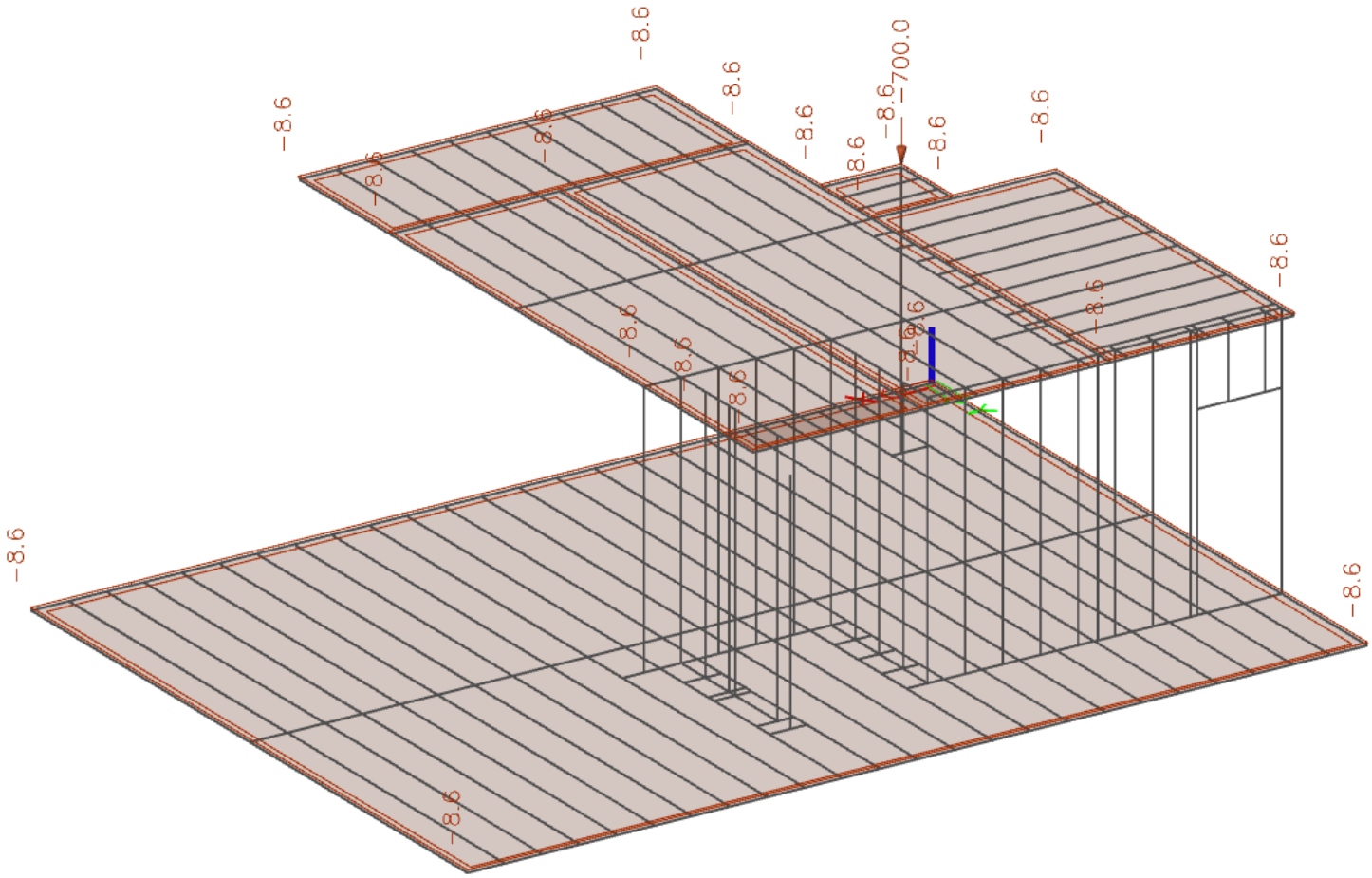


Dead loads distribution interior walls

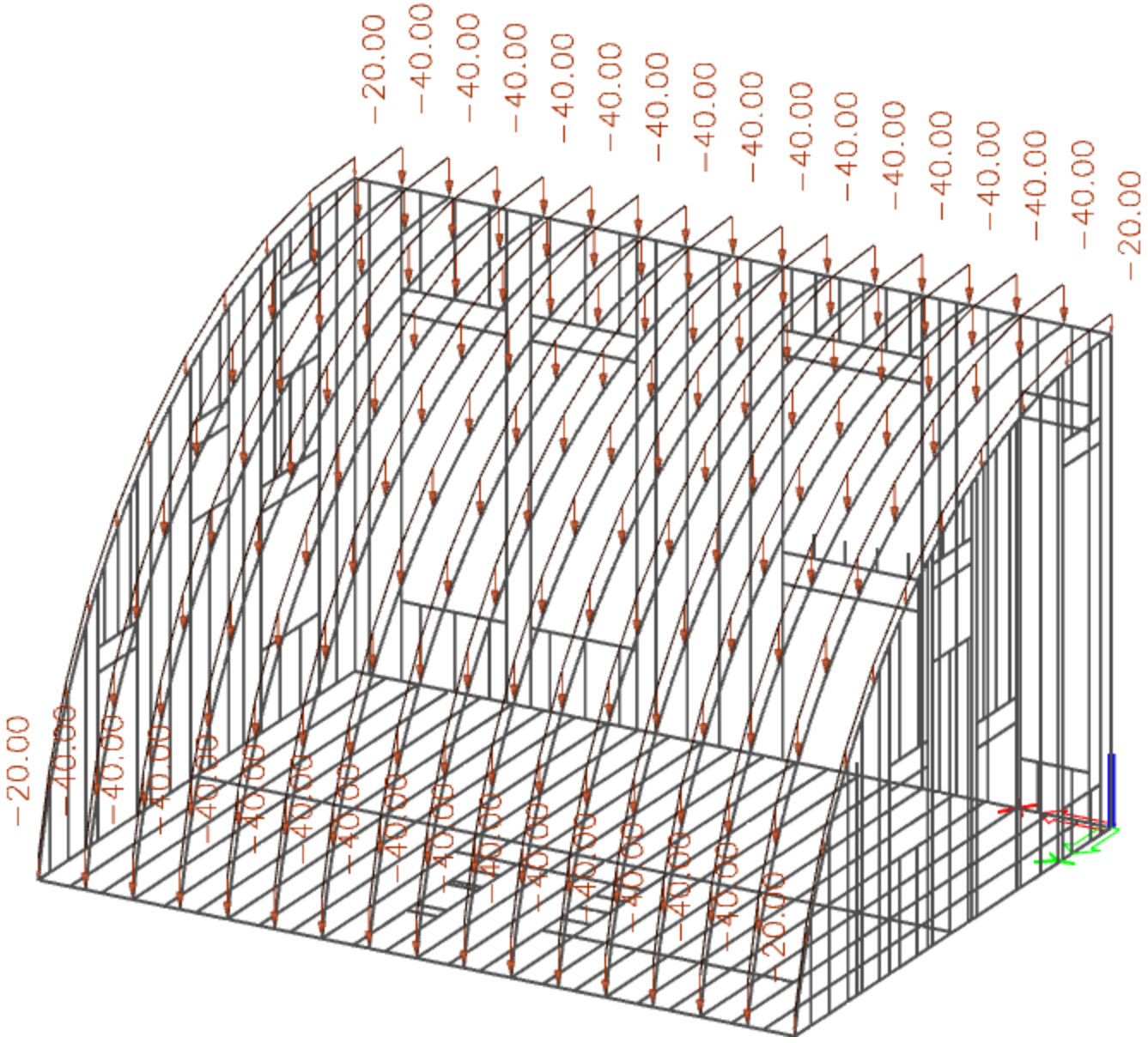
DL = 10.0 psf



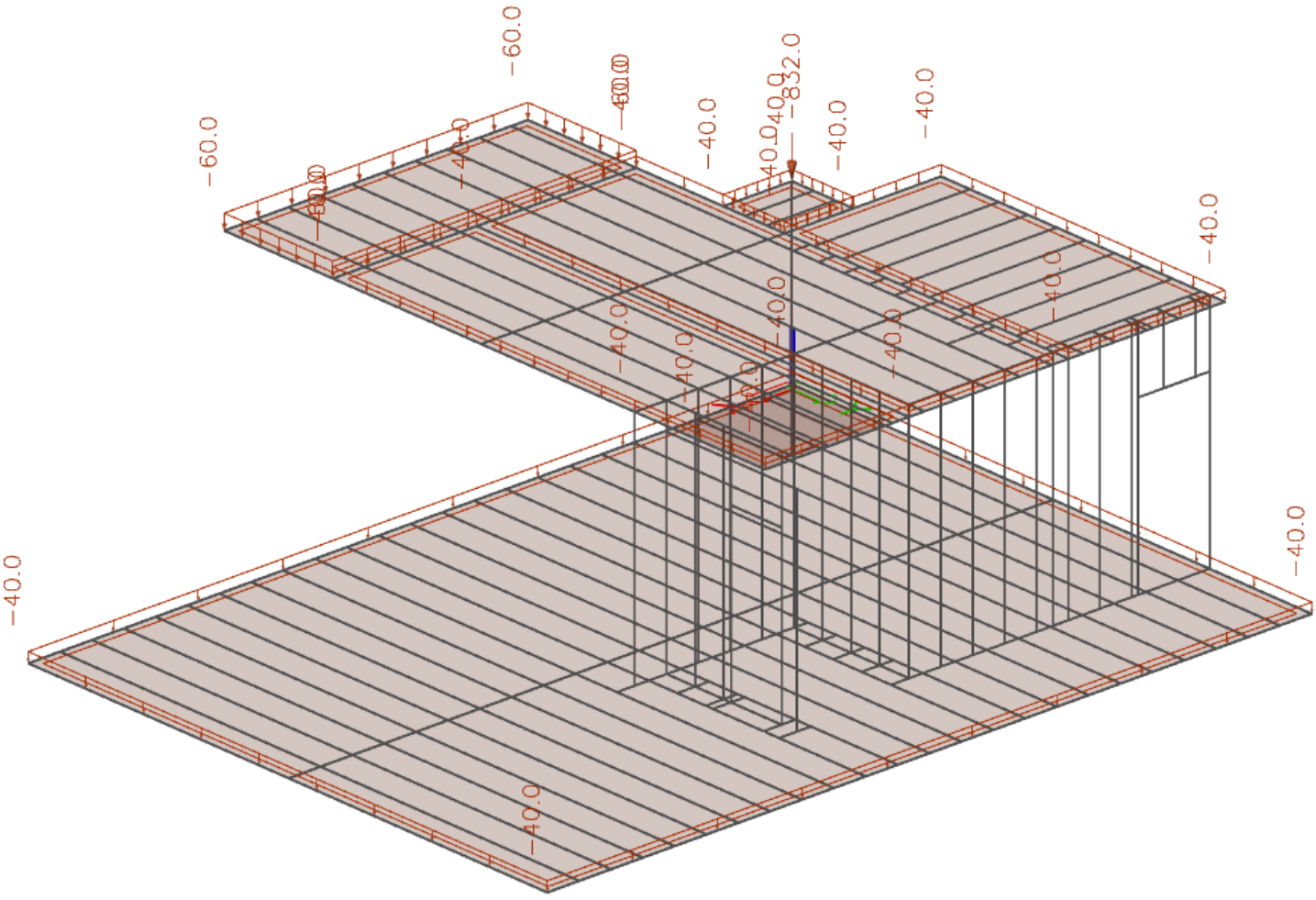
4. **DL4 - Dead Load Floor**
   Dead loads distribution
   DL = 8.6 psf
   DL = 700 lbs - spiral stair dead load



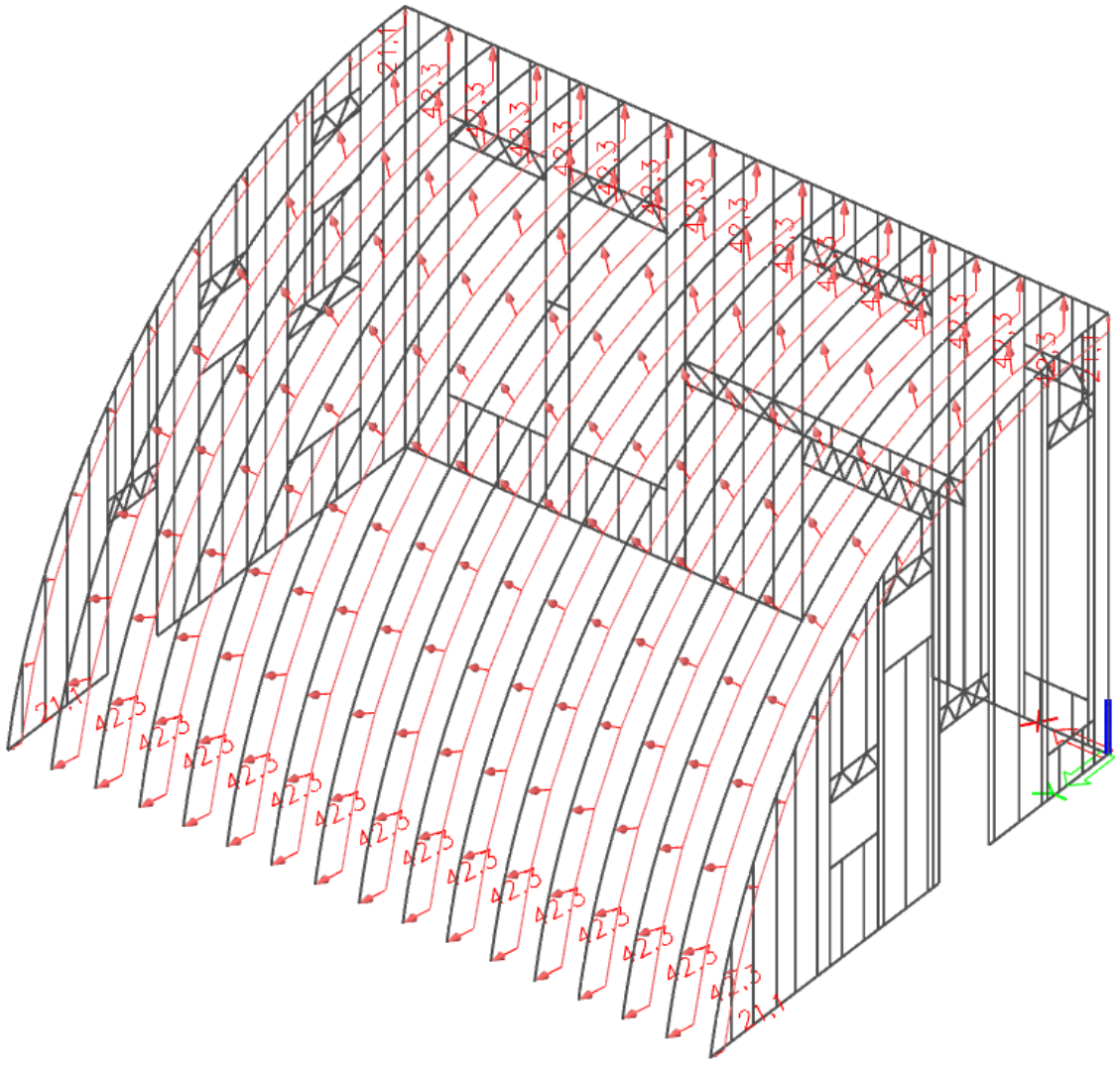
5. **Lr - Live Load Roof**
   Dead loads distribution
   Lr = 20.0 psf



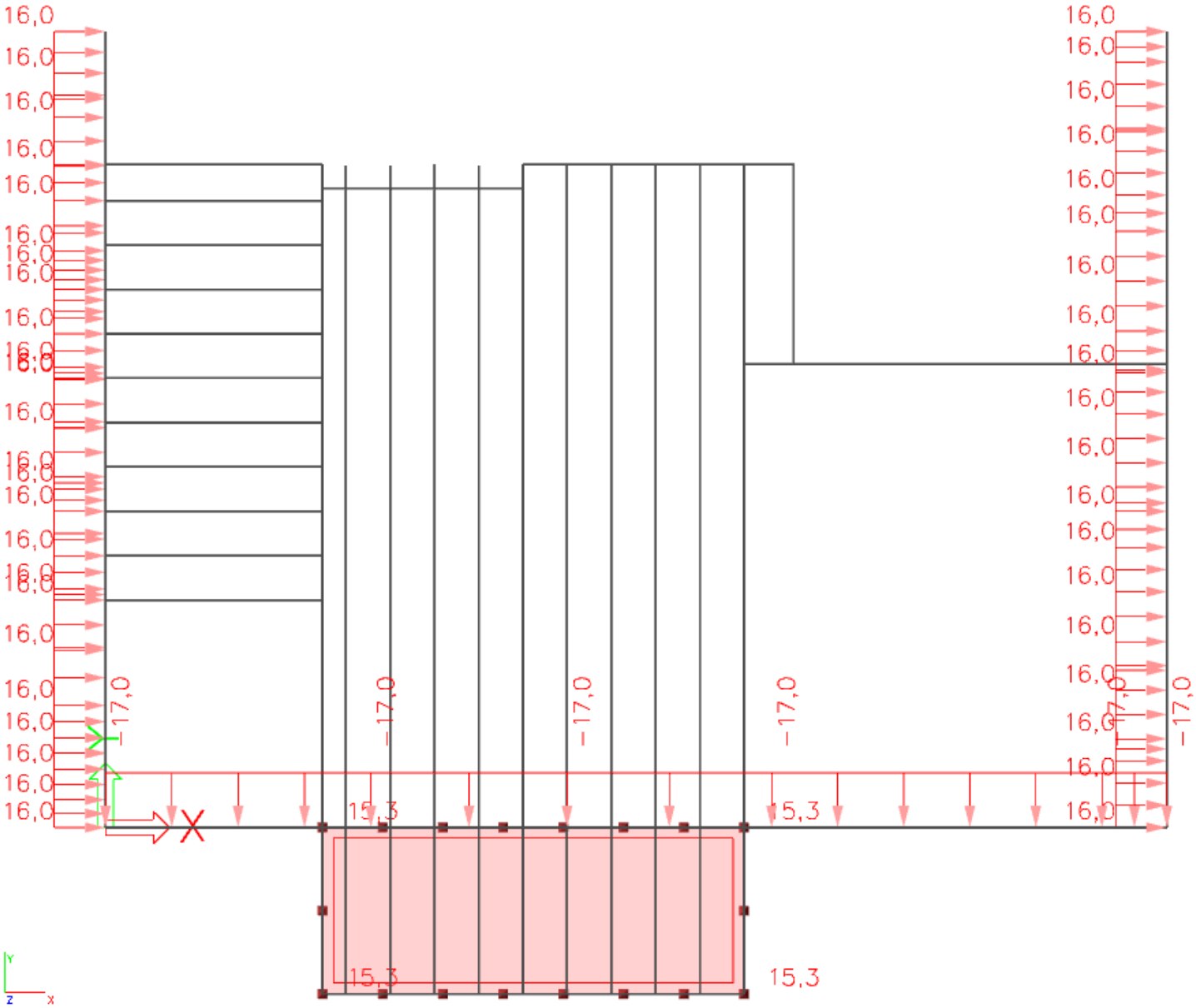
6. **L - Floor Live Load**
   Dead loads distribution
   Floor L = 40.0 psf.
   Balcony L = 60.0 psf.
   Spiral stair = 832 lbs - see page 81.



7. **Wx+ (0.18) - Wind Load**
   Dead loads distribution
   On the balcony - 15.30 psf;
   On the arches - 21.15 psf;
   On the windward wall - 16.0 psf;
   On the leeward wall - 16.0 psf;
   On the side wall - 17.0 psf.

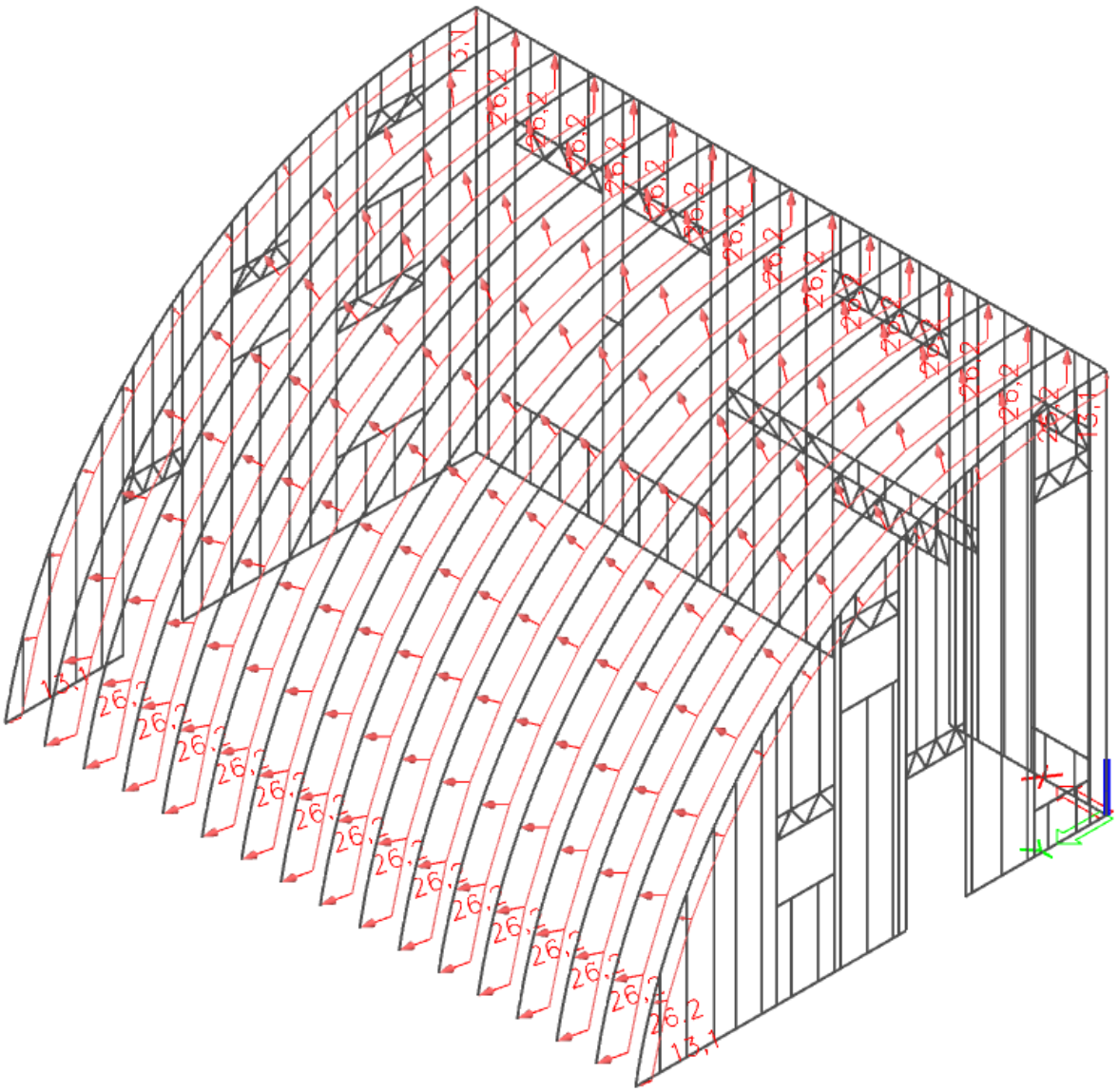


Wind on Arches

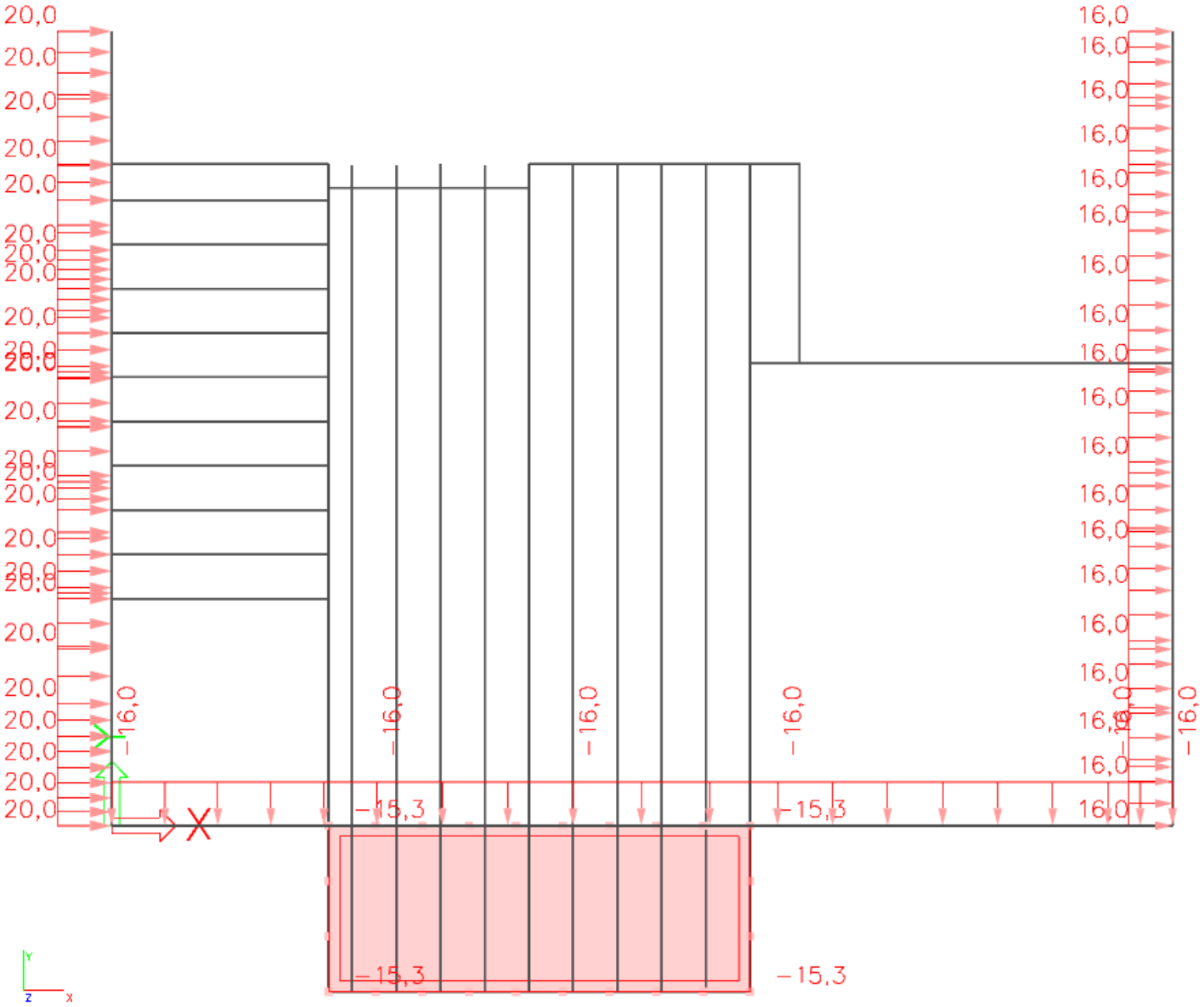


Wind on Walls and balcony

8. **Wx+ (-0.18) - Wind Load**
   Dead loads distribution
   On the balcony - 15.30 psf;
   On the arches - 13.10 psf;
   On the windward wall - 20.0 psf;
   On the leeward wall - 16.0 psf;
   On the side wall - 16.0 psf.

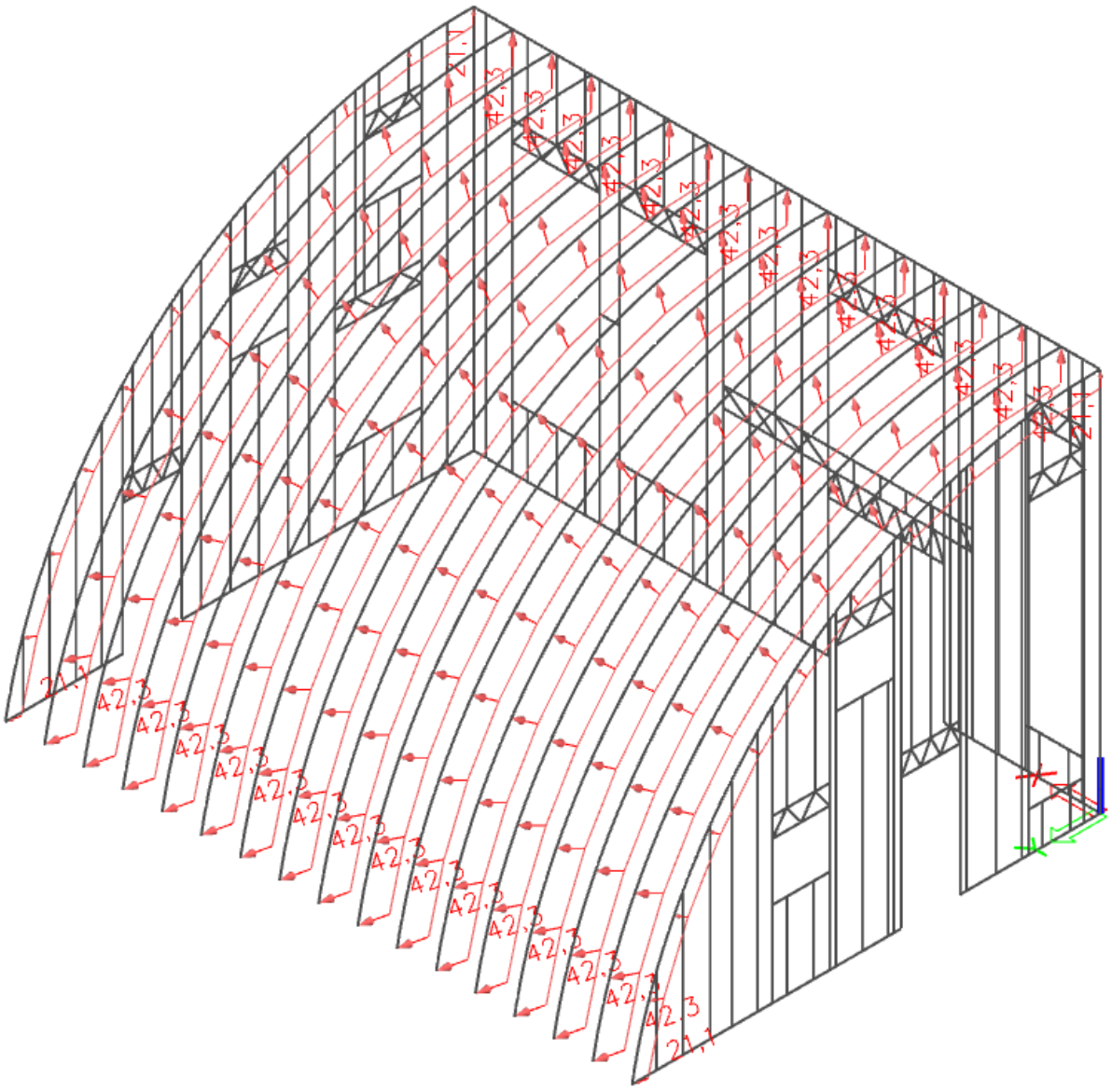


Wind on Arches

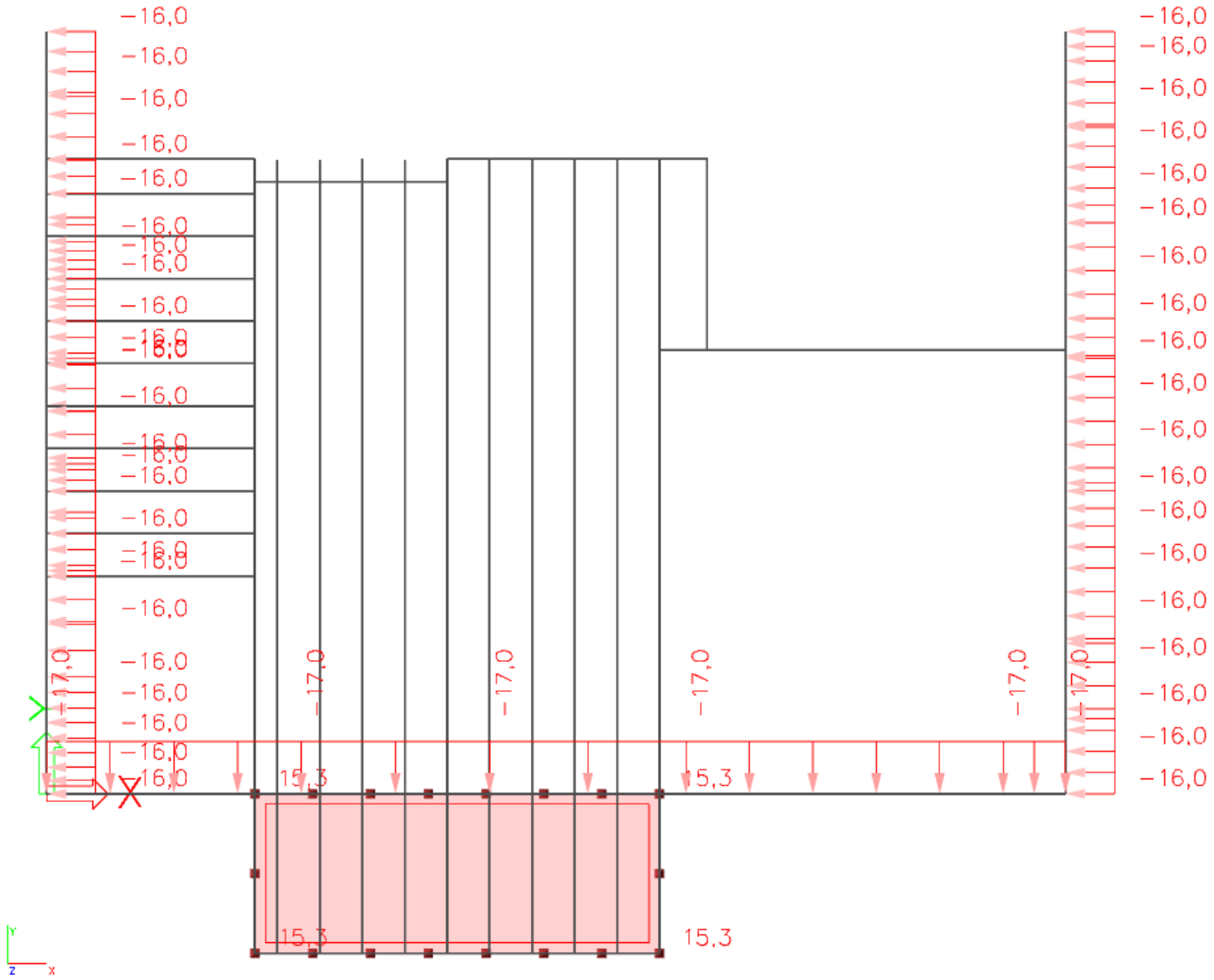


Wind on Walls and balcony

9. **Wx- (0.18) - Wind Load**
   Dead loads distribution
   On the balcony - 15.30 psf;
   On the arches - 21.15 psf;
   On the windward wall - 16.0 psf;
   On the leeward wall - 16.0 psf;
   On the side wall - 17.0 psf.



Wind on Arches



Wind on Walls and balcony

10. Wx- (-0.18) - Wind Load

Dead loads distribution

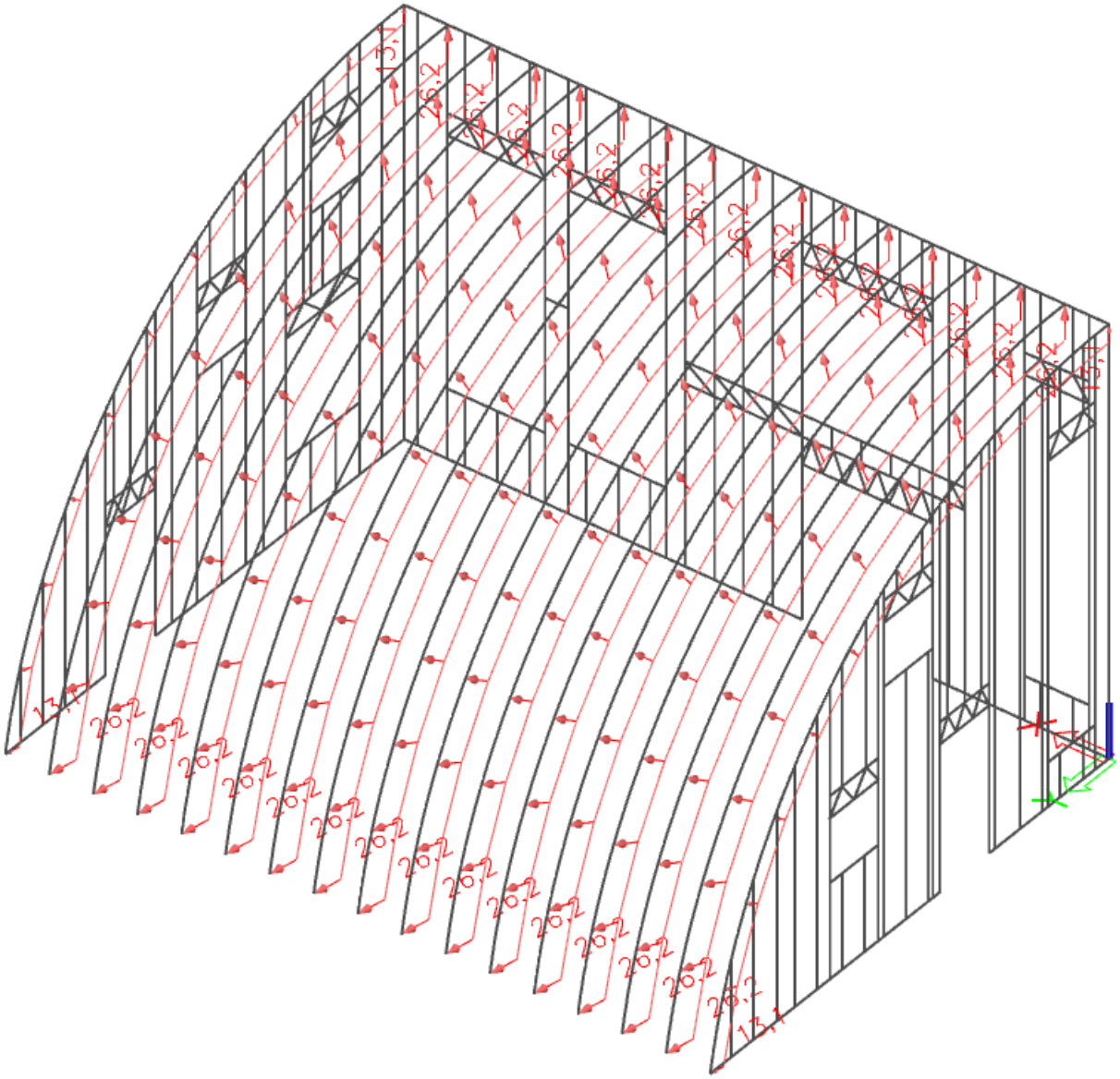
On the balcony - 15.30 psf;

On the arches - 13.10 psf;

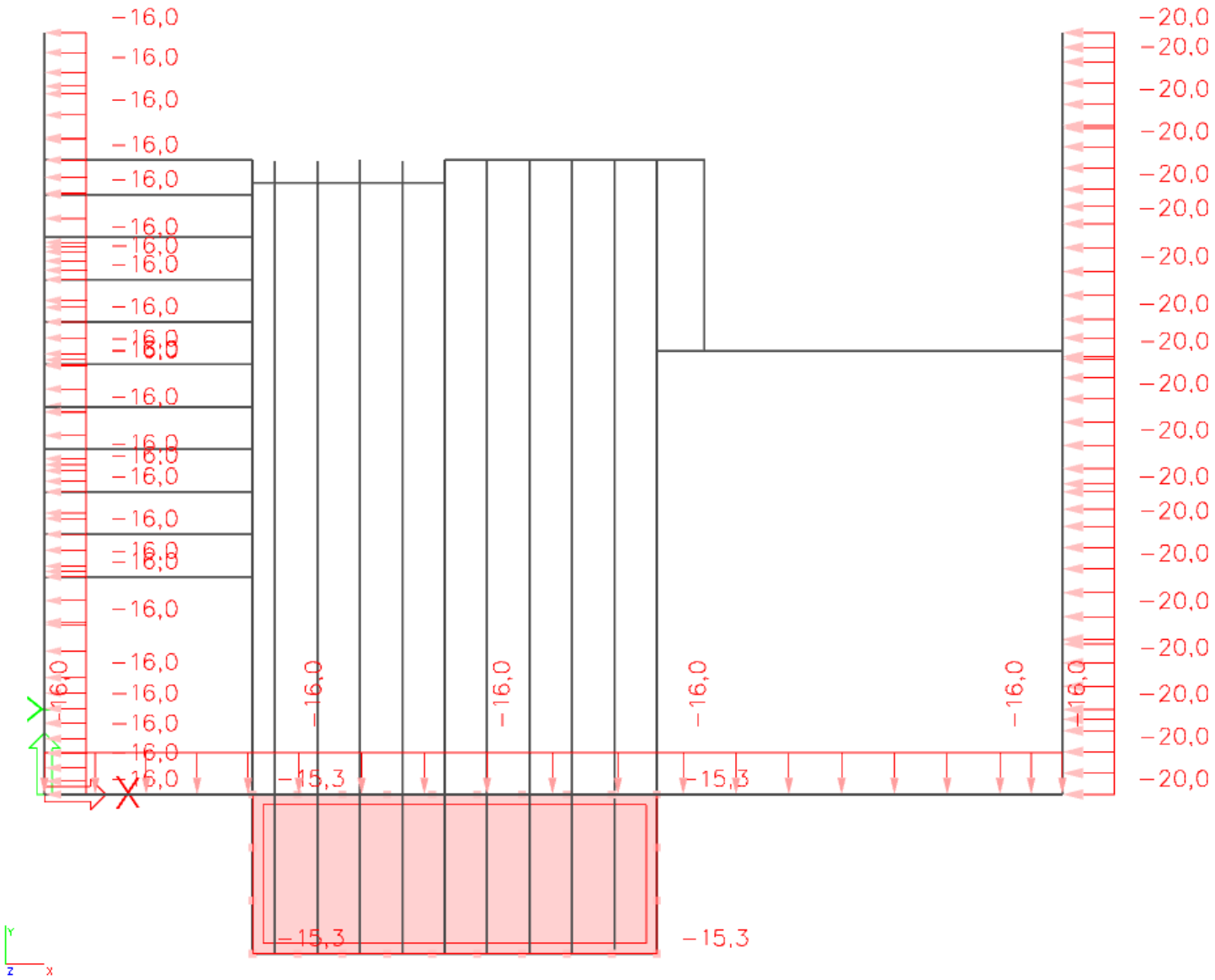
On the windward wall - 20.0 psf;

On the leeward wall - 16.0 psf;

On the side wall - 16.0 psf.



Wind on Arches



Wind on Walls and balcony

11. Wy+ (0.18) - Wind Load

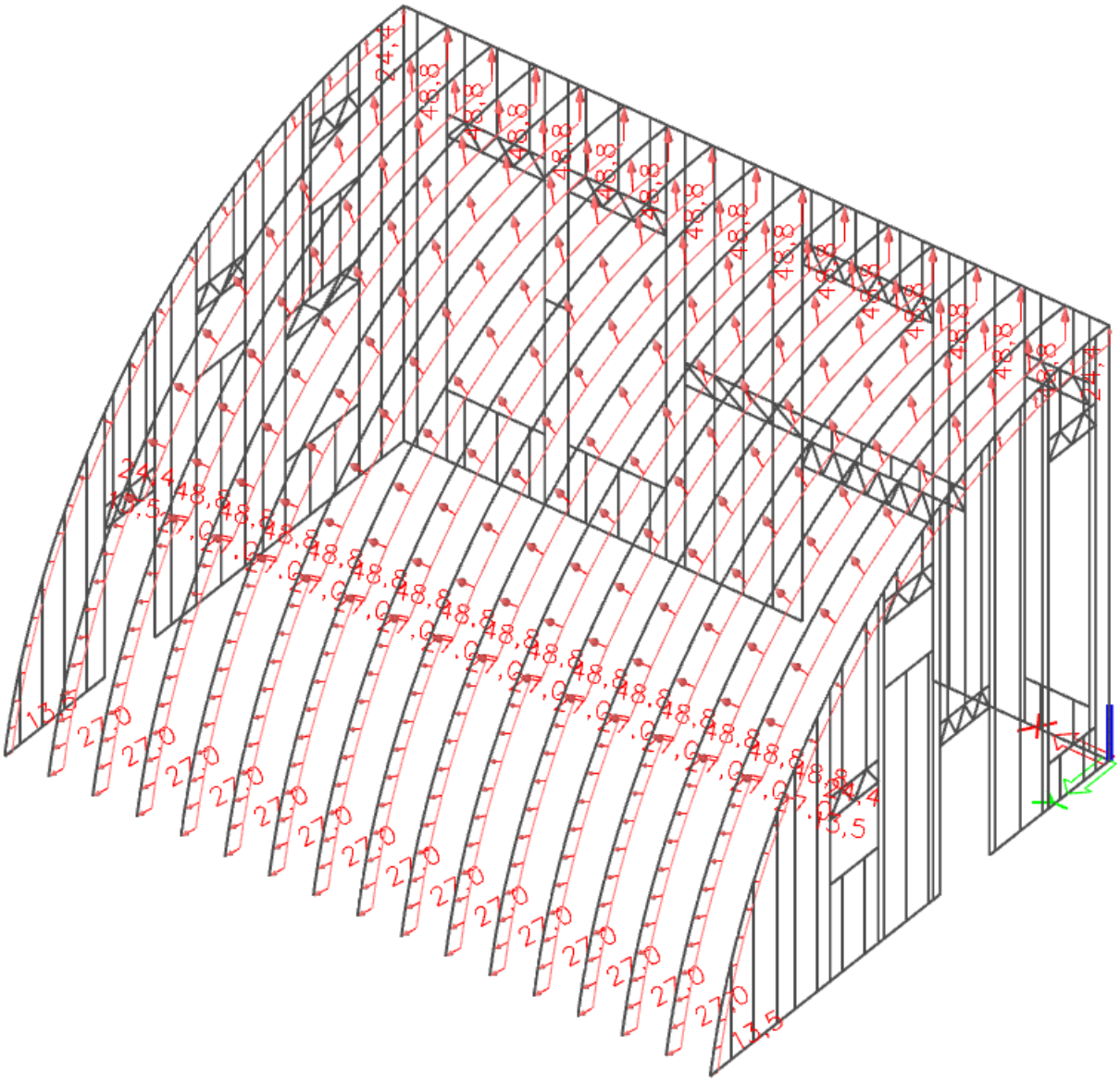
Dead loads distribution

On the balcony - 22.80 psf;

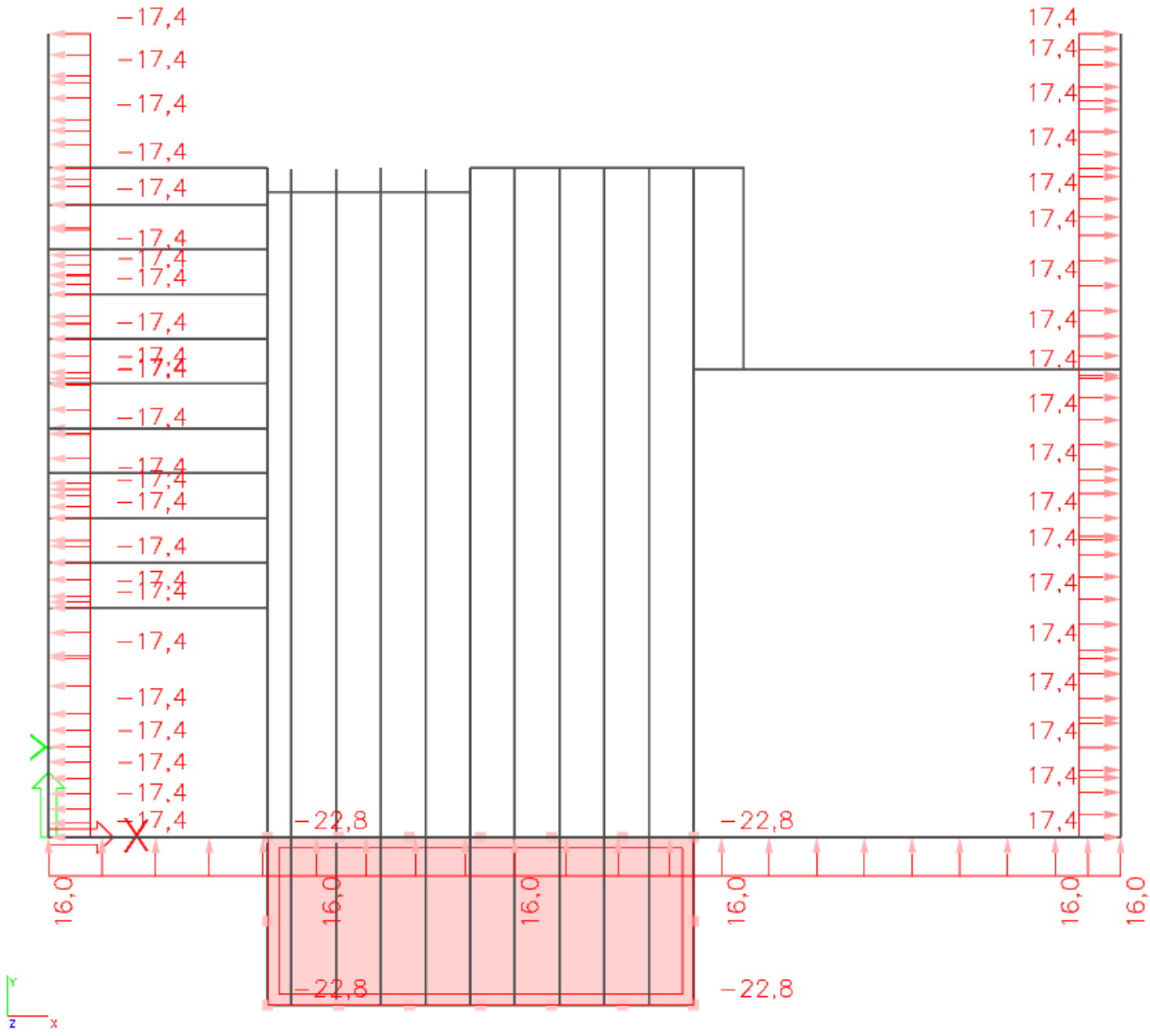
On the arches - 13.50, 48.80 psf;

On the windward wall - 16.0 psf;

On the side walls - 17.35 psf.



Wind on Arches



Wind on Walls and balcony

12. Wy+ (-0.18) - Wind Load

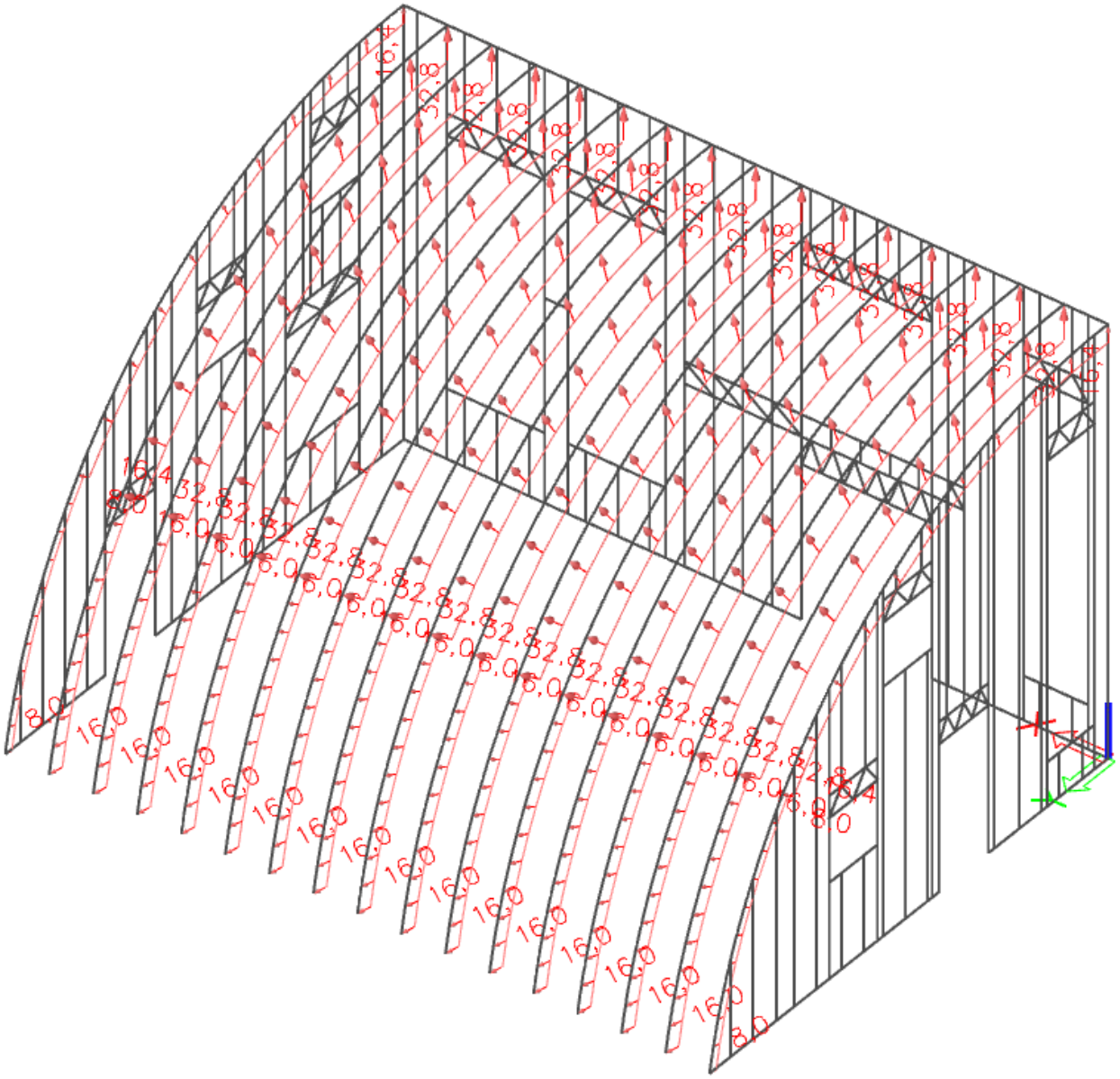
Dead loads distribution

On the balcony - 21.0 psf;

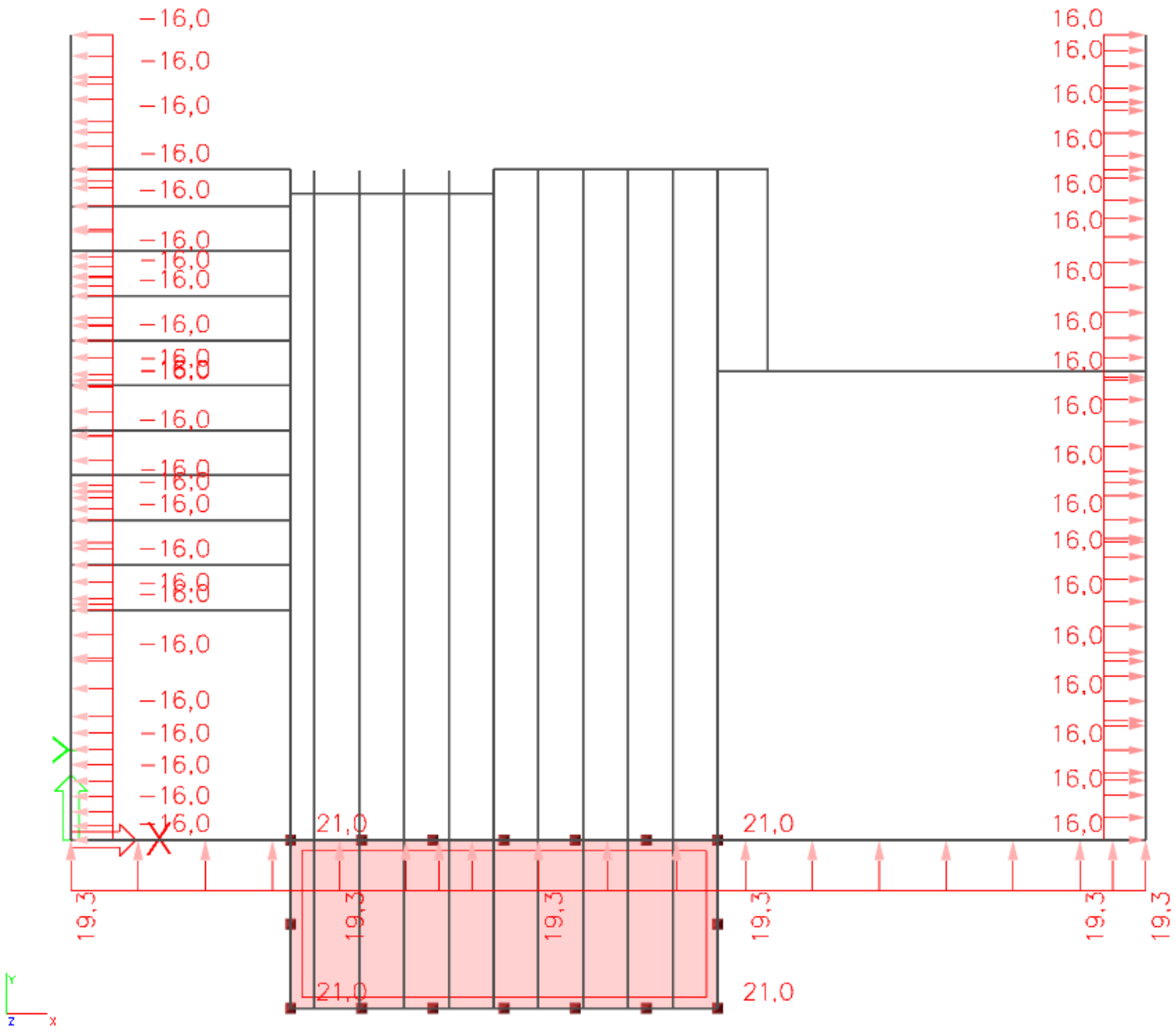
On the arches - 8.0, 16.40 psf;

On the windward wall - 19.25 psf;

On the side walls - 16.0 psf.



Wind on Arches



Wind on Walls and balcony

13. Wy- (0.18) - Wind Load

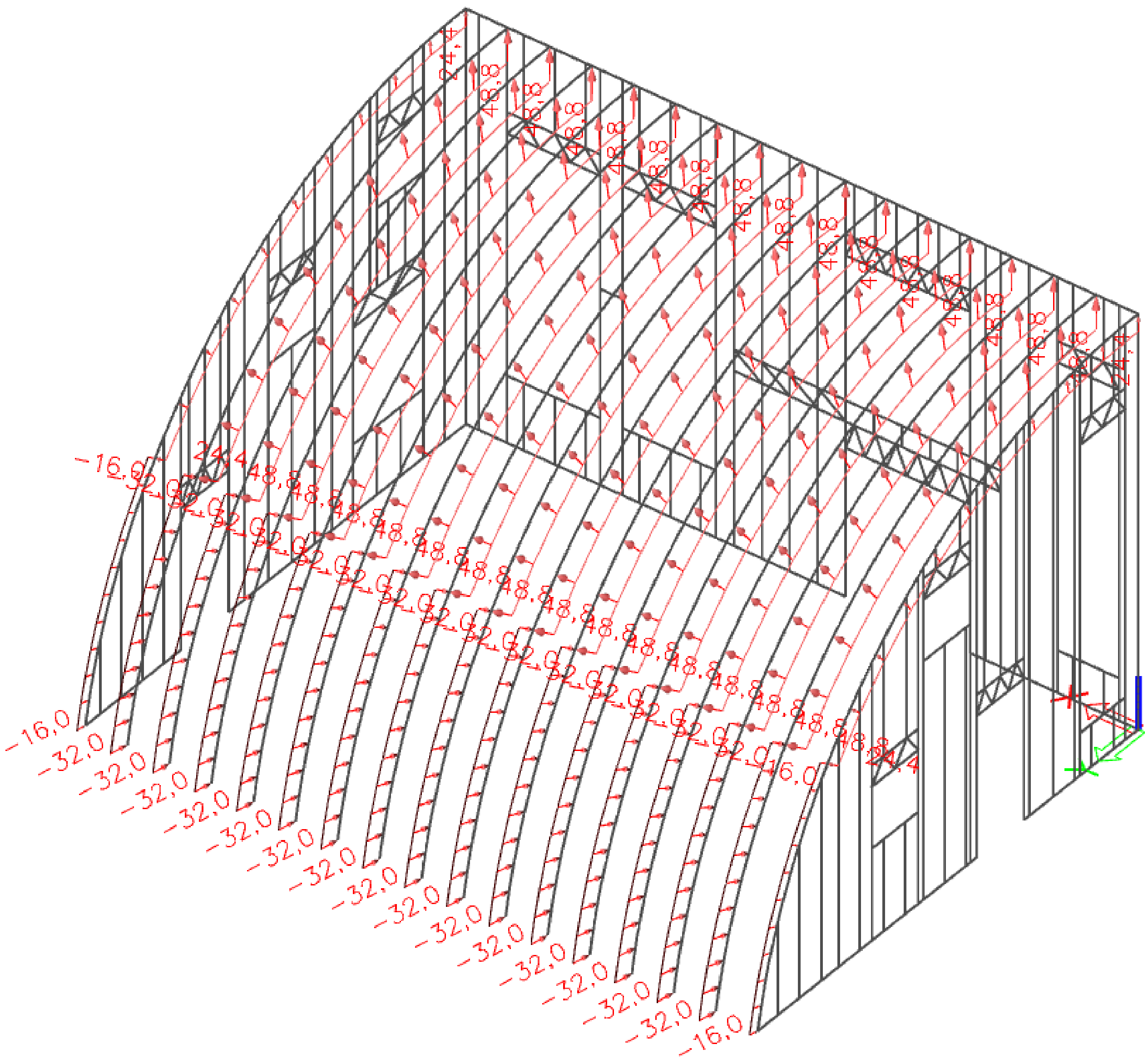
Dead loads distribution

On the balcony - 22.80 psf;

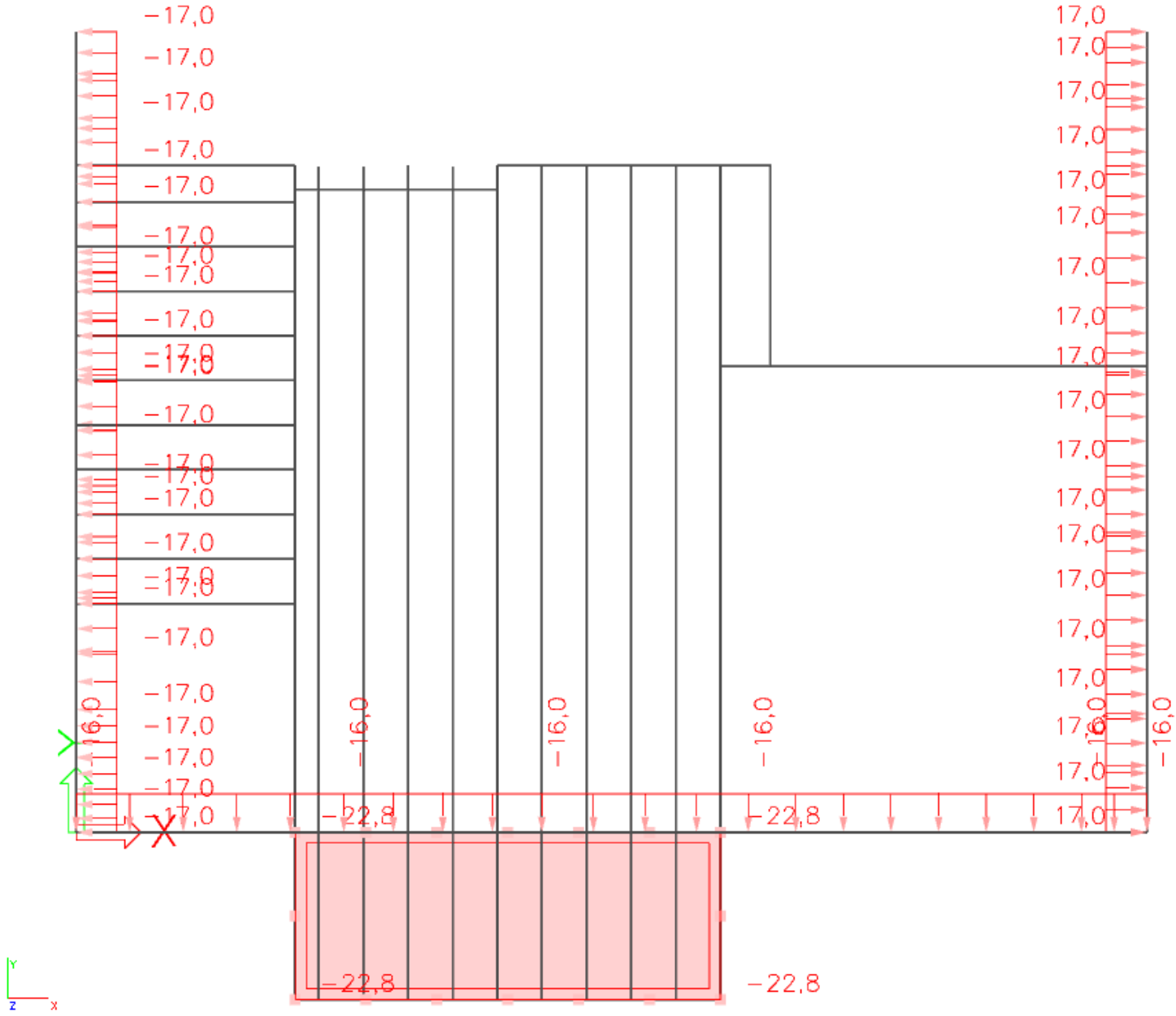
On the arches - 16.0, 24.40 psf;

On the leeward wall - 16.0 psf;

On the side wall - 17.0 psf.



Wind on Arches



Wind on Walls and balcony

14. Wy- (-0.18) - Wind Load

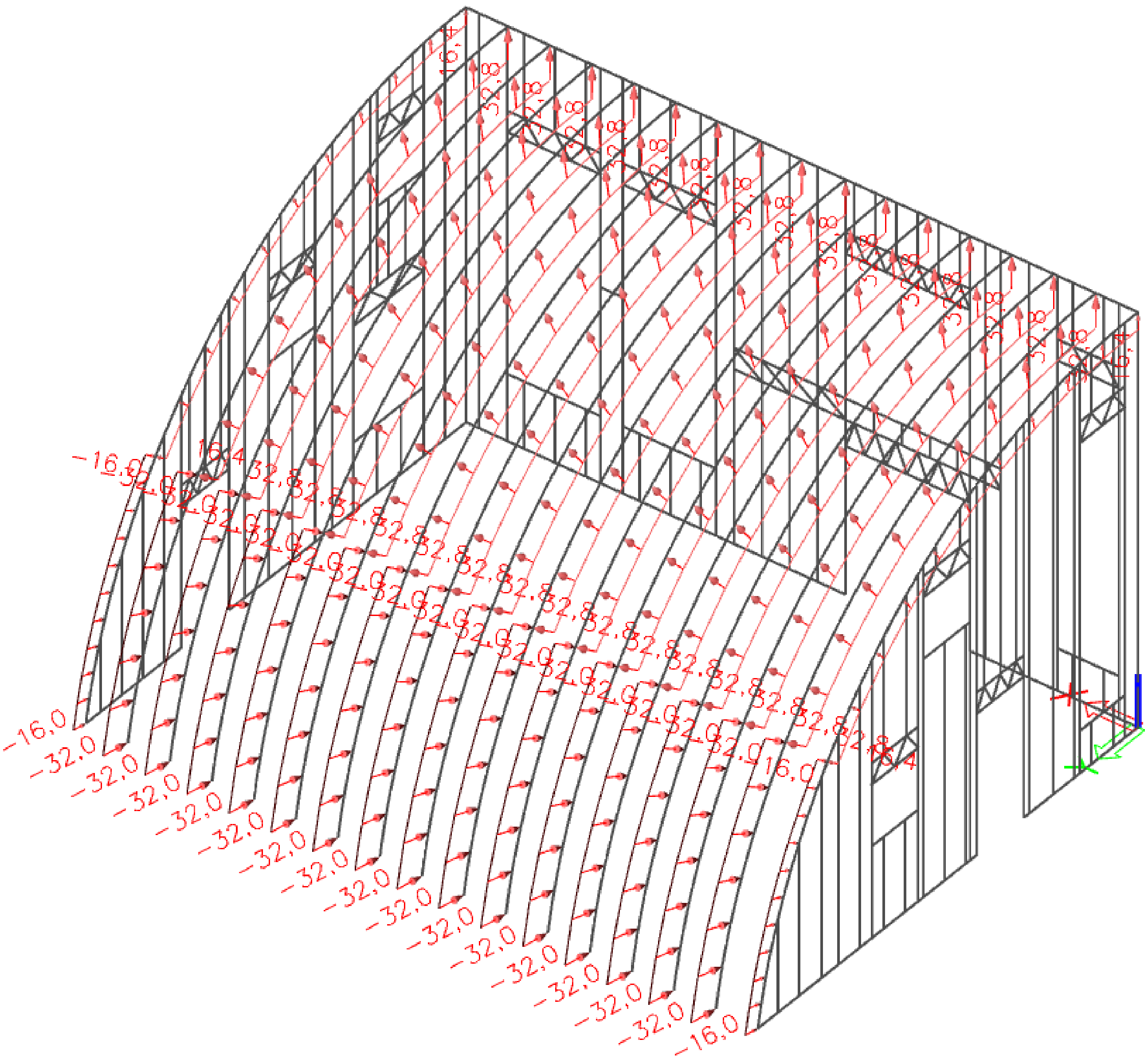
Dead loads distribution

On the balcony - 21.00 psf;

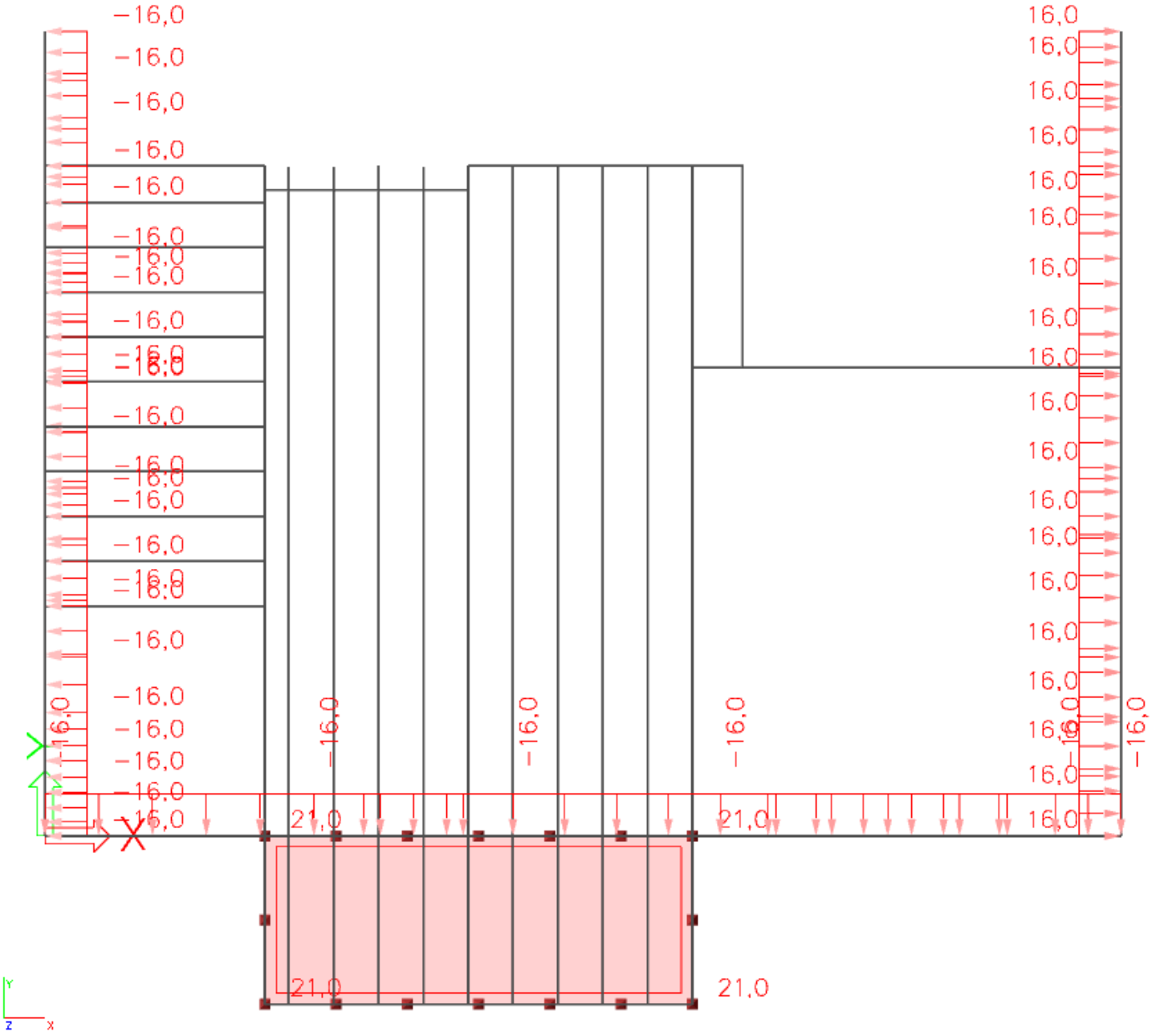
On the arches - -16.0, 16.8 psf;

On the leeward wall - 16.0 psf;

On the side wall - 16.0 psf.



Wind on Arches



Wind on Walls and balcony

Load cases

Case # Case name

Name	Description Spec	Action type Load type	Load group	Direction	Duration	Master load case
DL1	Self weight	Permanent	DL	-Z		
		Self weight				
DL2	Dead Load Roof	Permanent	DL			
		Standard				
DL3	Dead Load Wall	Permanent	DL			
		Standard				
DL4	Dead Load Floor	Variable	DL			
		Static				
Lr	Live Load Roof Standard	Variable	Lr		Short	None
		Static				
L	Live Load Floor Standard	Variable	L		Short	None
		Static				
Wx+ (0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Wx- (-0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Wx+ (0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Wx- (-0.18)	Wind Load Standard	Variable	W		Short	None
		Static				

Wy+ (0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Wy- (-0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Wy+ (0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Wy- (-0.18)	Wind Load Standard	Variable	W		Short	None
		Static				
Se-X	Seismic Load Seismicity	Variable	E			None
		Static equivalent				
Se-Y	Seismic Load Seismicity	Variable	E			None
		Static equivalent				

Load combinations

#	Comb. Name	Load cases
1	LRFD-Ult (auto) 1	1.4*DL1 + 1.4*DL2 + 1.4*DL3 + 1.4*DL4
2	LRFD-Ult (auto) 2	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4
3	LRFD-Ult (auto) 3	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr
4	LRFD-Ult (auto) 4	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L
5	LRFD-Ult (auto) 5	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1.6*L
6	LRFD-Ult (auto) 6	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L
7	LRFD-Ult (auto) 7	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr
8	LRFD-Ult (auto) 8	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*L
9	LRFD-Ult (auto) 9	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wx+ (0.18)
10	LRFD-Ult (auto) 10	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wx+ (-0.18)
11	LRFD-Ult (auto) 11	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wx- (0.18)
12	LRFD-Ult (auto) 12	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wx- (-0.18)
13	LRFD-Ult (auto) 13	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wy+ (0.18)
14	LRFD-Ult (auto) 14	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wy+ (-0.18)
15	LRFD-Ult (auto) 15	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wy- (0.18)
16	LRFD-Ult (auto) 16	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Wy- (-0.18)
17	LRFD-Ult (auto) 17	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wx+ (0.18)
18	LRFD-Ult (auto) 18	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wx+ (-0.18)
19	LRFD-Ult (auto) 19	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wx- (0.18)
20	LRFD-Ult (auto) 20	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wx- (-0.18)
21	LRFD-Ult (auto) 21	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wy+ (0.18)

#	Comb. Name	Load cases
22	LRFD-Ult (auto) 22	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wy+ (-0.18)
23	LRFD-Ult (auto) 23	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wy- (0.18)
24	LRFD-Ult (auto) 24	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wy- (-0.18)
25	LRFD-Ult (auto) 25	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wx+ (0.18)
26	LRFD-Ult (auto) 26	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wx+ (-0.18)
27	LRFD-Ult (auto) 27	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wx- (0.18)
28	LRFD-Ult (auto) 28	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wx- (-0.18)
29	LRFD-Ult (auto) 29	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L
30	LRFD-Ult (auto) 30	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wy+ (0.18)
31	LRFD-Ult (auto) 31	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wy+ (-0.18)
32	LRFD-Ult (auto) 32	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wy- (0.18)
33	LRFD-Ult (auto) 33	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Wy- (-0.18)
34	LRFD-Ult (auto) 34	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wx+ (0.18)
35	LRFD-Ult (auto) 35	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wx+ (0.18)
36	LRFD-Ult (auto) 36	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wx+ (-0.18)
37	LRFD-Ult (auto) 37	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wx+ (-0.18)
38	LRFD-Ult (auto) 38	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wx- (0.18)
39	LRFD-Ult (auto) 39	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wx- (0.18)
40	LRFD-Ult (auto) 40	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wx- (-0.18)
41	LRFD-Ult (auto) 41	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wx- (-0.18)
42	LRFD-Ult (auto) 42	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wy+ (0.18)

#	Comb. Name	Load cases
43	LRFD-Ult (auto) 43	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wy+ (0.18)
44	LRFD-Ult (auto) 44	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wy+ (-0.18)
45	LRFD-Ult (auto) 45	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wy+ (-0.18)
46	LRFD-Ult (auto) 46	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wy- (0.18)
47	LRFD-Ult (auto) 47	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wy- (0.18)
48	LRFD-Ult (auto) 48	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1*Wy- (-0.18)
49	LRFD-Ult (auto) 49	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Wy- (-0.18)
50	LRFD-Ult (auto) 50	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wx+ (0.18)
51	LRFD-Ult (auto) 51	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wx+ (-0.18)
52	LRFD-Ult (auto) 52	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wx- (0.18)
53	LRFD-Ult (auto) 53	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wx- (-0.18)
54	LRFD-Ult (auto) 54	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wy+ (0.18)
55	LRFD-Ult (auto) 55	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wy+ (-0.18)
56	LRFD-Ult (auto) 56	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wy- (0.18)
57	LRFD-Ult (auto) 57	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 0.5*L + 1*Wy- (-0.18)
58	LRFD-Ult (auto) 58	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Ex
59	LRFD-Ult (auto) 59	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + (-1)*Ex
60	LRFD-Ult (auto) 60	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1*Ey
61	LRFD-Ult (auto) 61	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + (-1)*Ey
62	LRFD-Ult (auto) 62	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Ex
63	LRFD-Ult (auto) 63	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + (-1)*Ex

#	Comb. Name	Load cases
64	LRFD-Ult (auto) 64	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1*Ey
65	LRFD-Ult (auto) 65	1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + (-1)*Ey
66	LRFD-Ult (auto) 66	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4
67	LRFD-Ult (auto) 67	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wx+ (0.18)
68	LRFD-Ult (auto) 68	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wx+ (-0.18)
69	LRFD-Ult (auto) 69	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wx- (0.18)
70	LRFD-Ult (auto) 70	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wx- (-0.18)
71	LRFD-Ult (auto) 71	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wy+ (0.18)
72	LRFD-Ult (auto) 72	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wy+ (-0.18)
73	LRFD-Ult (auto) 73	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wy- (0.18)
74	LRFD-Ult (auto) 74	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Wy- (-0.18)
75	LRFD-Ult (auto) 75	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Ex
76	LRFD-Ult (auto) 76	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + (-1)*Ex
77	LRFD-Ult (auto) 77	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 1*Ey
78	LRFD-Ult (auto) 78	0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + (-1)*Ey
79	LRFD-Ult (auto) 79	1*DL1 + 1*DL2 + 1*DL3 + 1*DL4
80	LRFD-Ult (auto) 80	1*DL1 + 1*DL2 + 1*DL3 + 1*DL4 + 1*Lr + 1*L

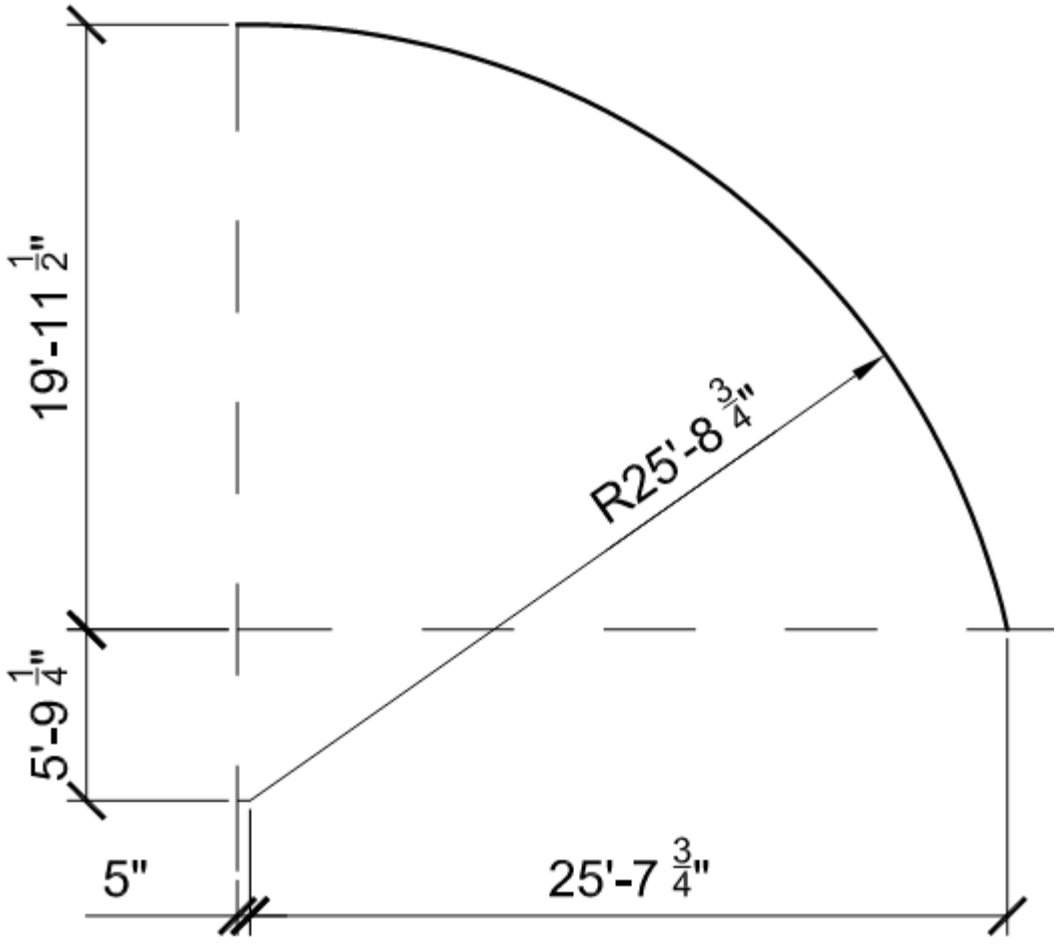
2.4 STEEL STRUCTURE CHECK
2.4.1 ARCH ROOF CHECK

Arches spacing - 2’-0”;

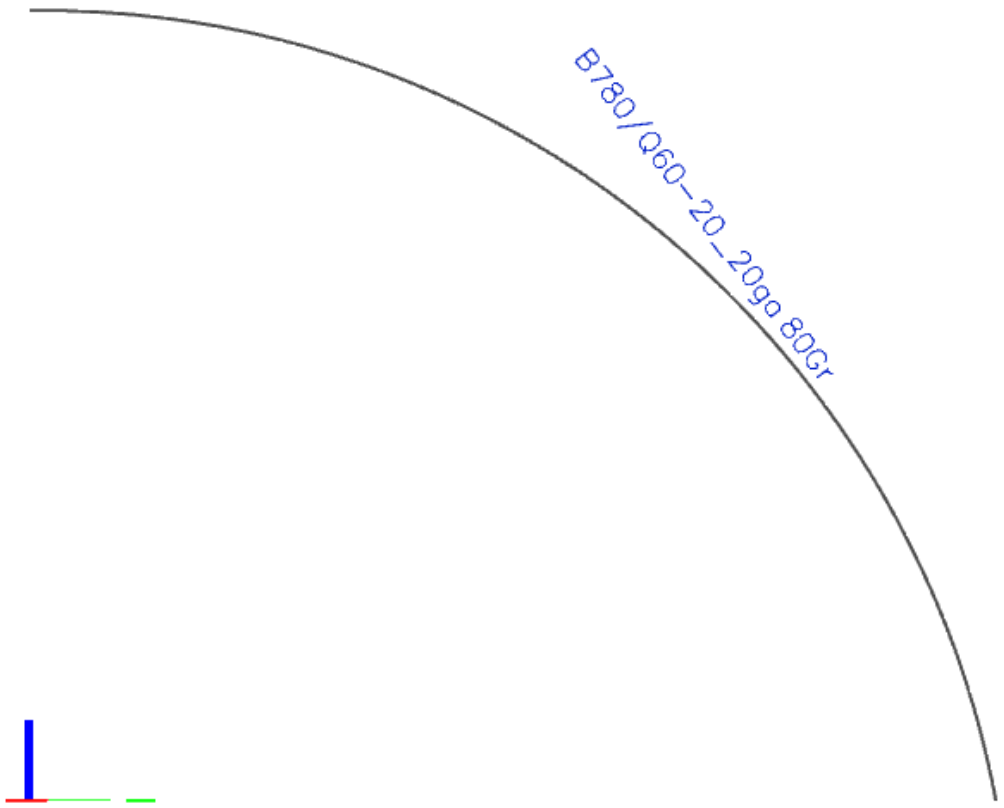
Arches height - 12’-8 1/2”.

For example, consider the bearing capacity of one arch.п

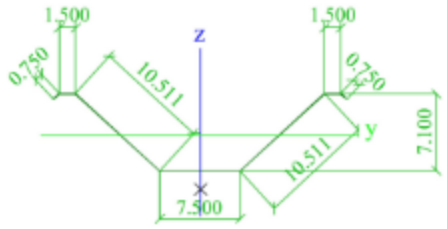
Analytical model



Numbers of members & Cross-sections of member



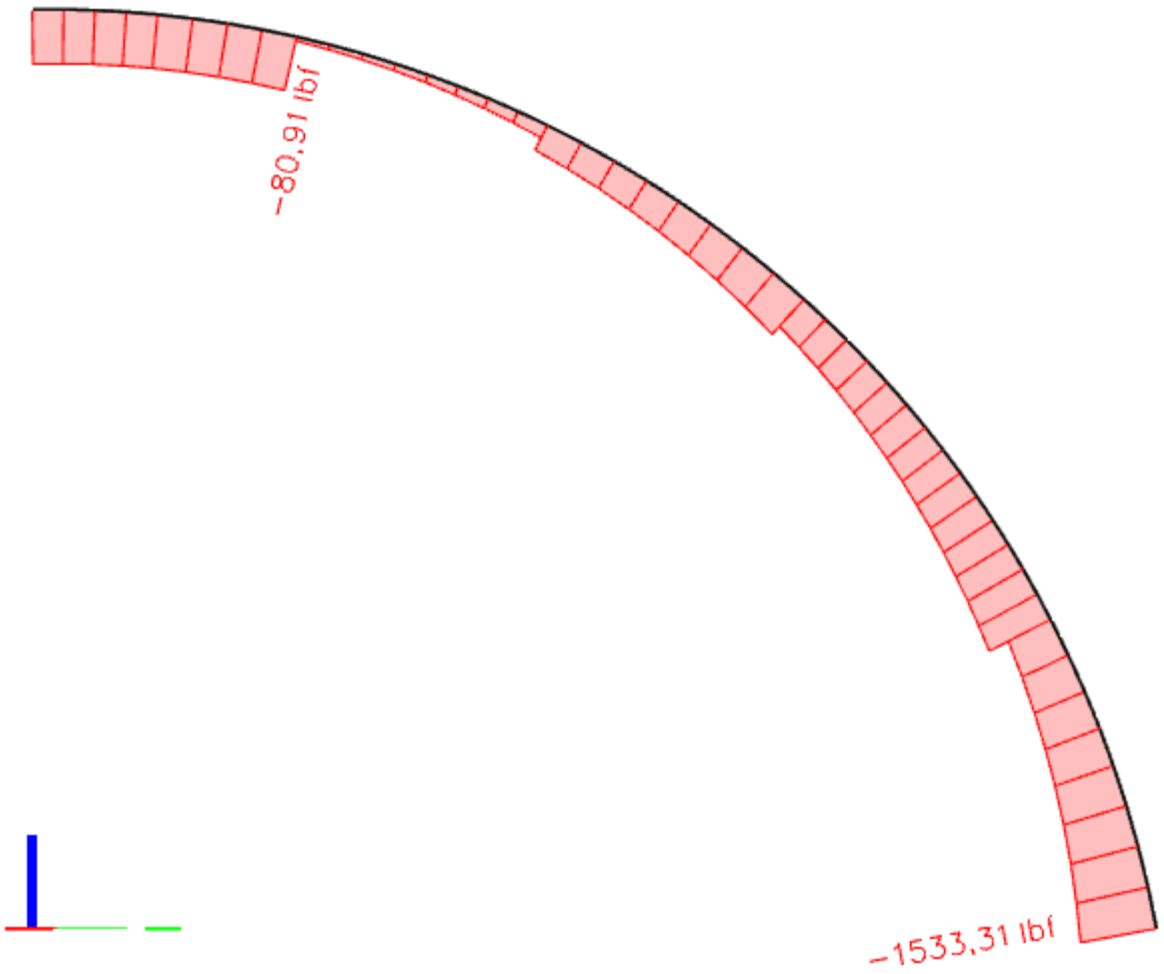
Wall thickness of cross-section 20ga or 0.0375 in.

2		
Type	Q60-20_20ga 80Gr	
Shape type	Thin-walled	
Item material	SG80	
Fabrication	cold formed	
Colour		
A [inch²]	1.238	
A_y [inch²], A_z [inch²]	0.965	0.804
A_L [inch²/inch], A_D [inch²/inch]	6.61e+01	6.61e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	-0.098	-0.383
α [deg]	0.00	
I_y [inch⁴], I_z [inch⁴]	8.748	77.915
i_y [inch], i_z [inch]	2.658	7.932
$W_{el,y}$ [inch³], $W_{el,z}$ [inch³]	2.242	5.744
$W_{pl,y}$ [inch³], $W_{pl,z}$ [inch³]	2.954	8.663
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	2.36e+02	2.36e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	6.93e+02	6.93e+02
d_y [inch], d_z [inch]	0.000	-4.894
I_t [inch⁴], I_w [inch⁶]	0.001	25.927
β_y [inch], β_z [inch]	29.497	0.000
Picture		

FORCE DIAGRAM

Axial force diagram N,

LRFD-Ult (auto) 24 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wy-(-0.18)), lbf.



Axial force diagram N,
LRFD-Ult (auto) 69 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 0.5*Wy-(0.18)), lbf.

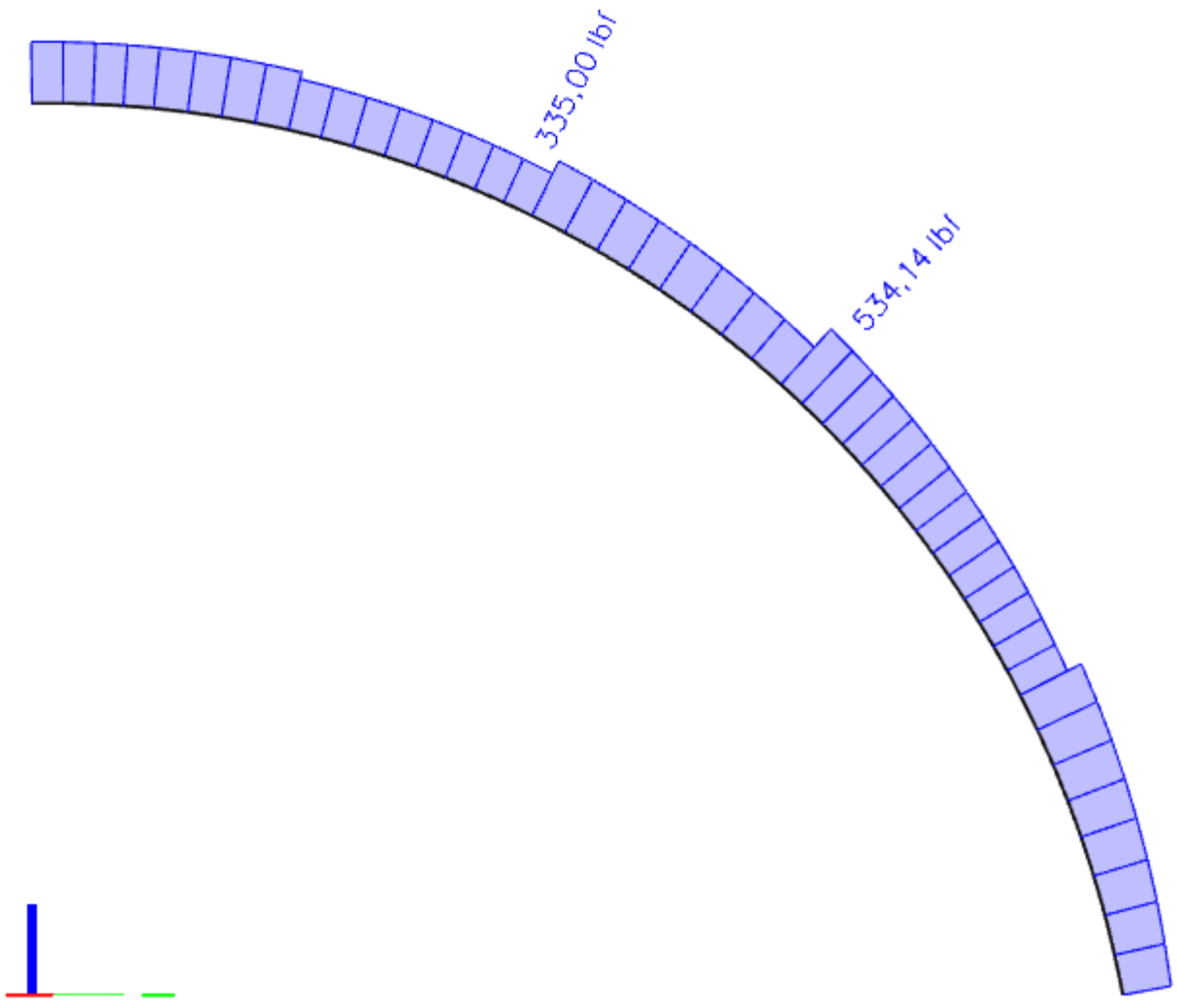


Diagram of bending moments My,
LRFD-Ult (auto) 24 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*Wy-(-0.18)), lbfft.

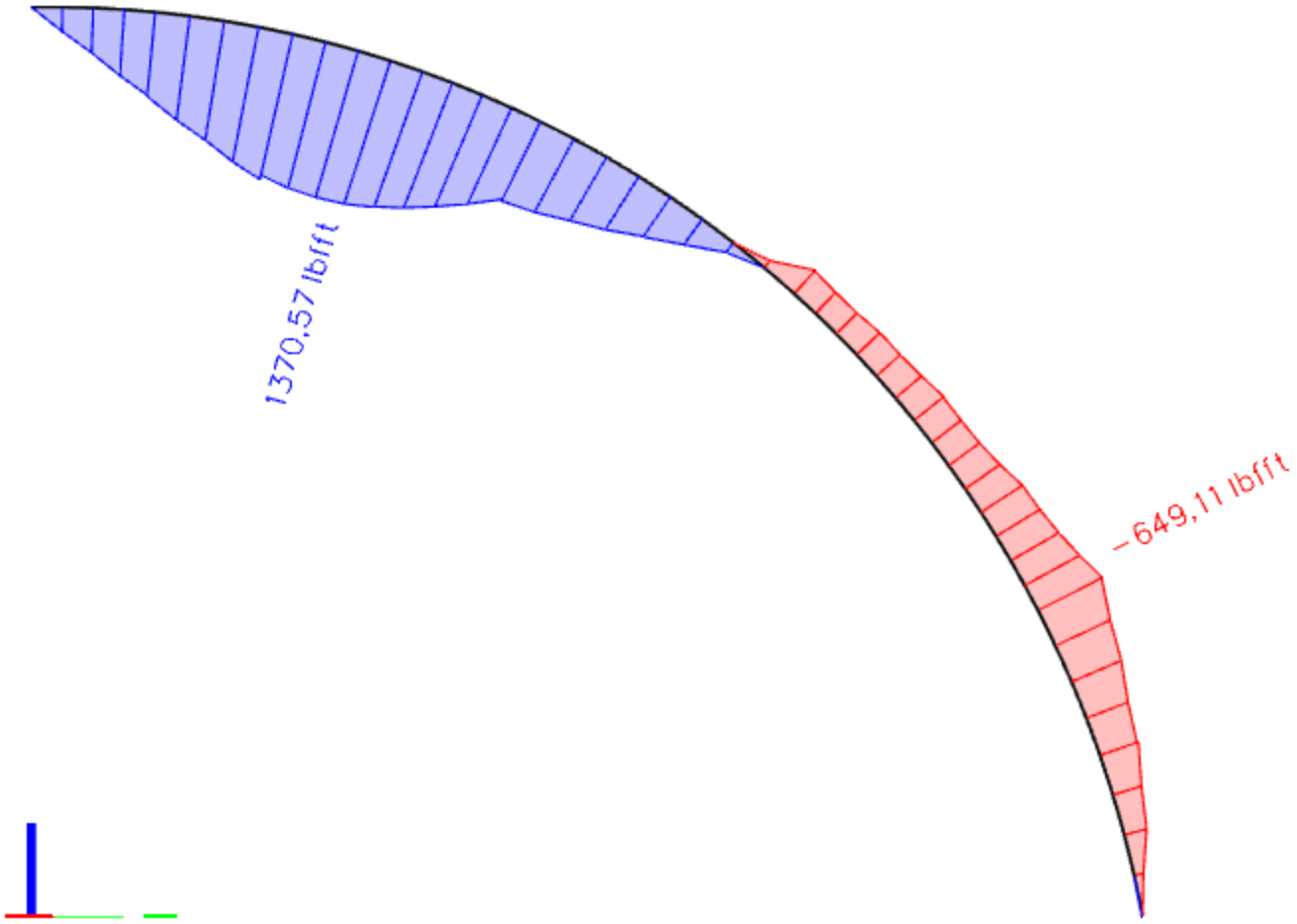
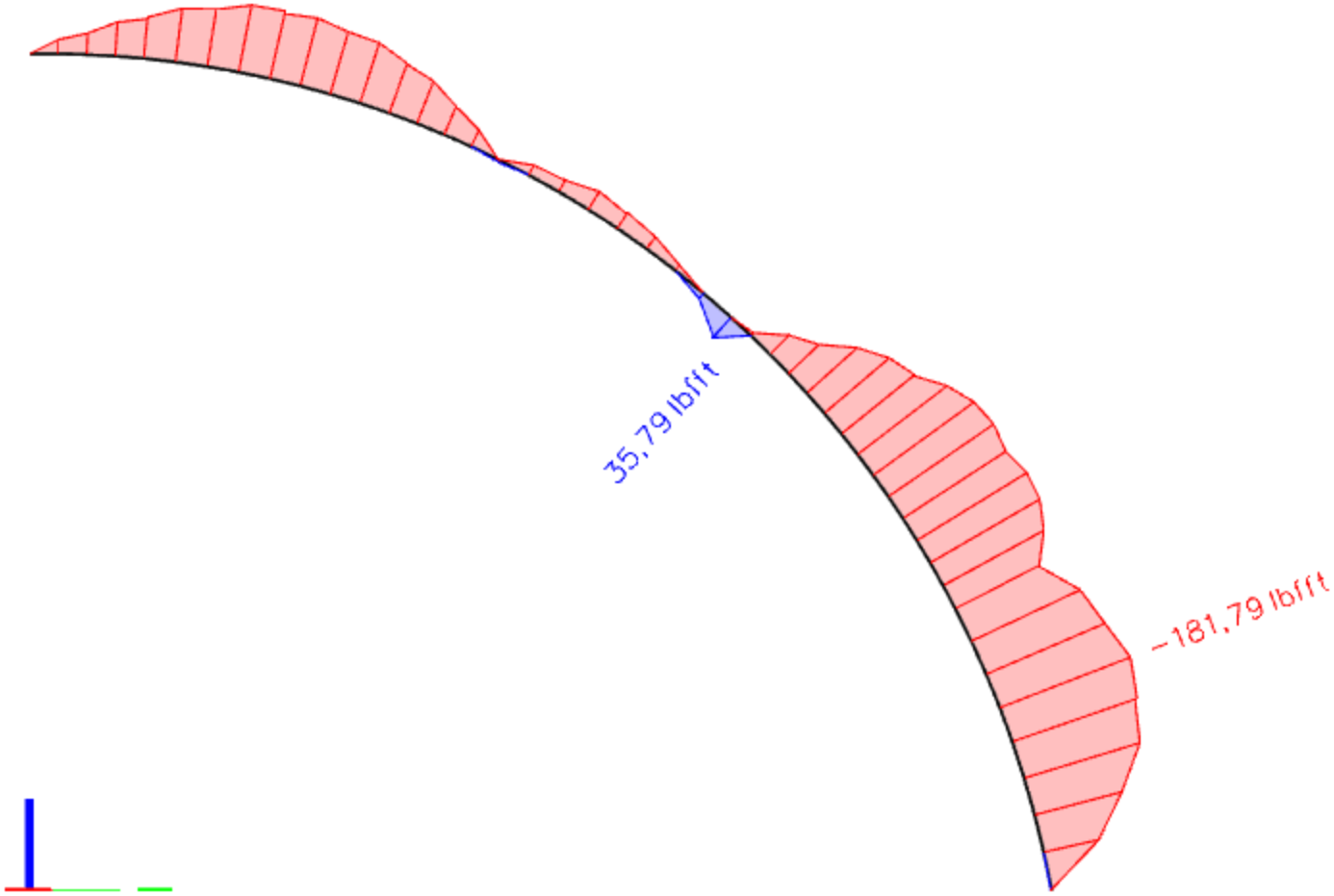
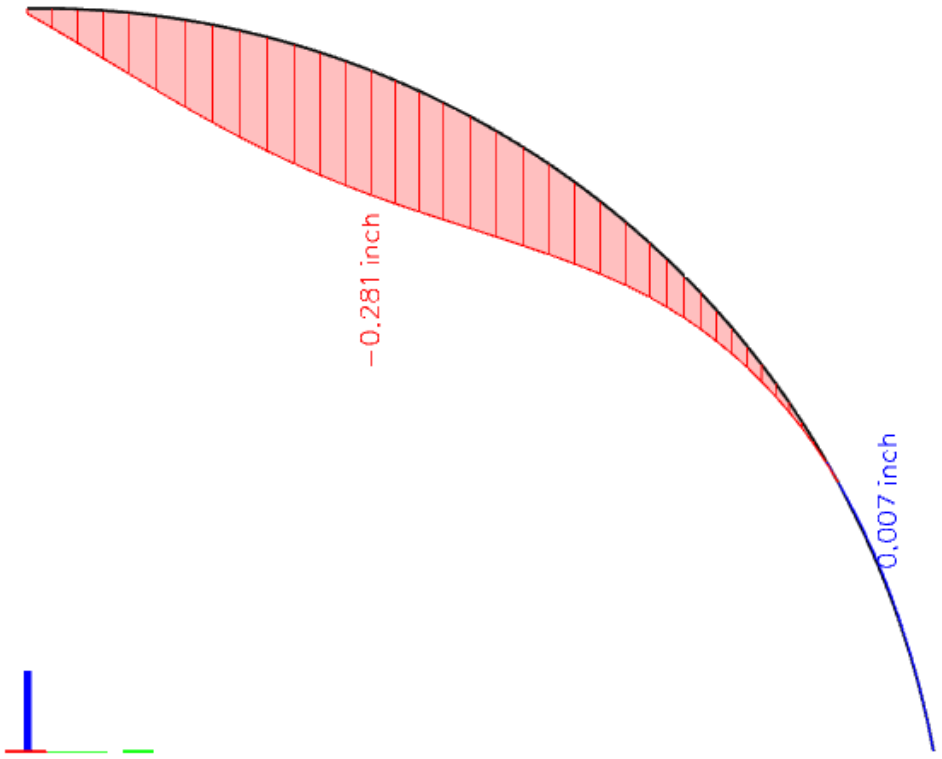


Diagram of bending moments My,
LRFD-Ult (auto) 69 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + 0.5*Wy-(0.18)), lbfft.

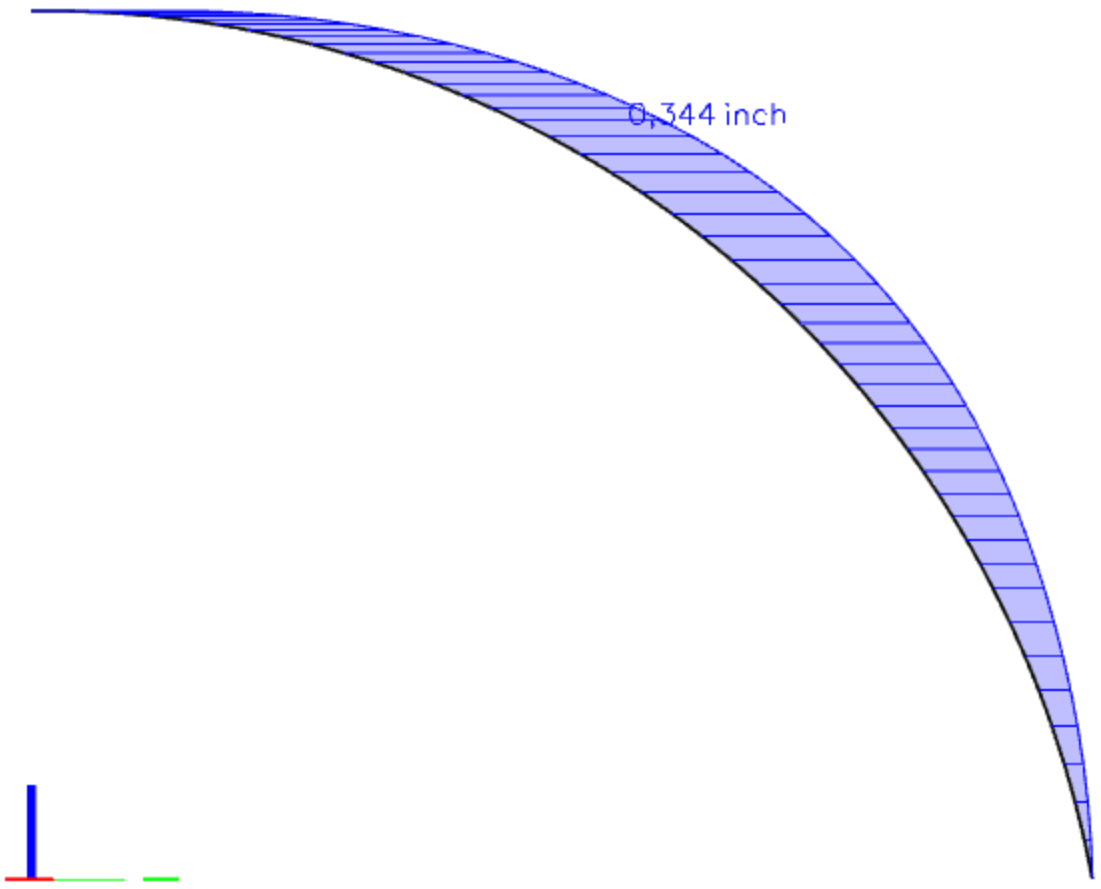


Displacement
LRFD-Ult (auto) 80 (1*DL1+1*DL2+1*DL3+1*DL4+1*Lr+1*L), inch:



The maximum deflection of 0.281”. According to TABLE R301.7 the code IRC 2018, maximum arch member deflection should not exceed - L/240.
L=13’-7”=163”, 140”/240=0.679”. 0.107"< 0.679”. Deflection is OK!

Arch deflection by Load Wy+ (0.18), inch.



The maximum horizontal drift of 0.344” according to table 12.12-1 the code ASCE 7-16. Structure type / Risk category - “All other structures / II - the maximum story drift should not exceed 0.020hsx. hsx is the story height below Level x.

0.020*(20’*12”)=4.80”. 0.344"< 4.80”. Drift is OK!

ARCH MEMBER CHECK
MEMBER B780 CHECK

AISI S100-16 LRFD Check

Member B780	Q60-20_20ga 80Gr	SG80	LRFD-Ult (auto)	0.93

Material data		
Yield stress Fy	80.00	ksi
Tensile stress Fu	82.00	ksi
fabrication	cold formed	

The critical check is on position 6.98 ft

Axis definition :
- local x- axis in this code check is referring to the local z axis in Scia Engineer
- local y- axis in this code check is referring to the local y axis in Scia Engineer

Internal forces		
Pu	-1429.56	lbf
Vux	115.81	lbf
Vuy	0.06	lbf
Mut	-0.00	lbfft
Mux	5.01	lbfft
Muy	-1053.81	lbfft

...::Flexural Strength about X-axis::...
Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	4' 10.748"	ft
Sigma,ey	586.047	ksi
Kt	1.00	
Lt	4' 10.748"	ft
Sigma,t	18.536	ksi
Cb	1.98	
Sfx	2.242	$inch^3$
Fcre	1106.309	ksi

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.750	-22.536 -30.415	-	-	-	-	-	- -	- -	-	- -	-
2	1.500	-30.415 -30.415	-	-	-	-	-	- -	- -	-	- -	-
3	10.511	80.000 -30.415	0.38	12.019	4.011	4.466	0.213	- 2.238	0.662 1.119	-	- -	-
4	7.500	80.000 80.000	1.00	4.000	2.622	5.524	0.174	1.304 -	- -	-	- -	-
5	10.511	80.000 -30.415	0.38	12.019	4.011	4.466	0.213	- 2.238	0.662 1.119	-	- -	-
6	1.500	-30.415 -30.415	-	-	-	-	-	- -	- -	-	- -	-
7	0.750	-22.536 -30.415	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.218	inch^3
Mnyo	1455.96	lbfft
Resistance factor	0.90	
Unity check	0.80	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	20878.074	ksi
Kt	1.00	
Lt	4' 10.748"	ft
Sigma,t	18.536	ksi
Cb	1.00	
Sfy	5.744	inch^3
Fcre	1299.924	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...

Shear Strength

According to article G2.1 and formula (G2.1.1)

Shear force Vx

Element ID	Aw [inch^2]	Vn [lbf]
1	0.015	733.98
2	0.056	2700.00
3	0.214	381.98
4	0.281	984.58
5	0.214	381.98
6	0.056	2700.00
7	0.015	733.98

Table of values		
Vn,x	8616.49	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnyo	1455.96	lbfft
Vnx	8616.49	lbf
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.65+0.00) = 0.80

...::Axial Compression Strength::...
Nominal Axial Strength
According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.750	80.000 80.000	1.00	0.430	28.184	1.685	0.516	0.387 -	- -	-	- -	-
2	1.500	80.000 80.000	1.00	4.000	65.543	1.105	0.725	1.087 -	- -	-	- -	-
3	10.511	80.000 80.000	1.00	4.000	1.335	7.741	0.126	1.319 -	- -	-	- -	-
4	7.500	80.000 80.000	1.00	4.000	2.622	5.524	0.174	1.304 -	- -	-	- -	-
5	10.511	80.000 80.000	1.00	4.000	1.335	7.741	0.126	1.319 -	- -	-	- -	-
6	1.500	80.000 80.000	1.00	4.000	65.543	1.105	0.725	1.087 -	- -	-	- -	-
7	0.750	80.000 80.000	1.00	0.430	28.184	1.685	0.516	0.387 -	- -	-	- -	-

Table of values		
Fn	80.000	ksi
Ae	0.258	inch^2
Pno	20672.06	lbf
Resistance factor	0.85	
Unity check	0.08	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	5	33 5/8	ft
Effective Length factor K	0.50	0.70	
Effective Length	2 1/2	23 5/8	ft
Slenderness	3.70	106.18	
Flexural Buckling stress Fcre	20878.074	25.395	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	20878.074	ksi
Sigma,ey	25.395	ksi
Kt	1.00	
Lt	5	ft
Sigma,t	18.536	ksi
Sigma,TF	18.532	ksi
Torsional (-Flexural) buckling stress Fcre	18.532	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.750	16.252 16.252	1.00	0.430	28.184	0.759	0.935	0.702 -	- -	-	- -	-
2	1.500	16.252 16.252	1.00	4.000	65.543	0.498	1.000	1.500 -	- -	-	- -	-
3	10.511	16.252 16.252	1.00	4.000	1.335	3.489	0.269	2.822 -	- -	-	- -	-
4	7.500	16.252 16.252	1.00	4.000	2.622	2.490	0.366	2.746 -	- -	-	- -	-
5	10.511	16.252 16.252	1.00	4.000	1.335	3.489	0.269	2.822 -	- -	-	- -	-
6	1.500	16.252 16.252	1.00	4.000	65.543	0.498	1.000	1.500 -	- -	-	- -	-
7	0.750	16.252 16.252	1.00	0.430	28.184	0.759	0.935	0.702 -	- -	-	- -	-

Table of values		
Fe	18.532	ksi
lambda, c	2.08	
Fn	16.252	ksi
Ae	0.480	inch^2
Pn	7797.41	lbf
Resistance factor	0.85	
Unity check	0.22	-

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (H1.2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.750	1.154 1.154	1.00	0.430	28.184	0.202	1.000	0.750 -	- -	-	- -	-
2	1.500	1.154 1.154	1.00	4.000	65.543	0.133	1.000	1.500 -	- -	-	- -	-
3	10.511	1.154 1.154	1.00	4.000	1.335	0.930	0.821	8.628 -	- -	-	- -	-
4	7.500	1.154 1.154	1.00	4.000	2.622	0.664	1.000	7.500 -	- -	-	- -	-

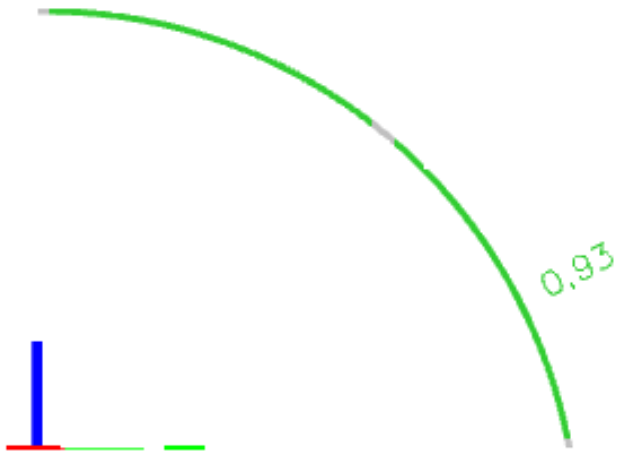
Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (H1.2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
5	10.511	1.154 1.154	1.00	4.000	1.335	0.930	0.821	8.628 -	- -	-	- -	-
6	1.500	1.154 1.154	1.00	4.000	65.543	0.133	1.000	1.500 -	- -	-	- -	-
7	0.750	1.154 1.154	1.00	0.430	28.184	0.202	1.000	0.750 -	- -	-	- -	-

Table of values		
Centerline shift ex	0.000	inch
Centerline shift ey	-0.043	inch
Additional moment Mx	5.12	lbfft
Additional moment My	0.00	lbfft
Mnx	33600.02	lbfft
Mny	1455.96	lbfft
PEx	25853201.99	lbf
PEy	31445.87	lbf
Alfa x	1.00	
Alfa y	0.95	
Cmx	0.85	
Cmy	0.85	
Pn	7797.41	lbf
Pno	20672.06	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.22+0.00+0.72 = 0.93 - (H1.2-1)
Unity check = 0.08+0.00+0.80 = 0.89 -

Unity check

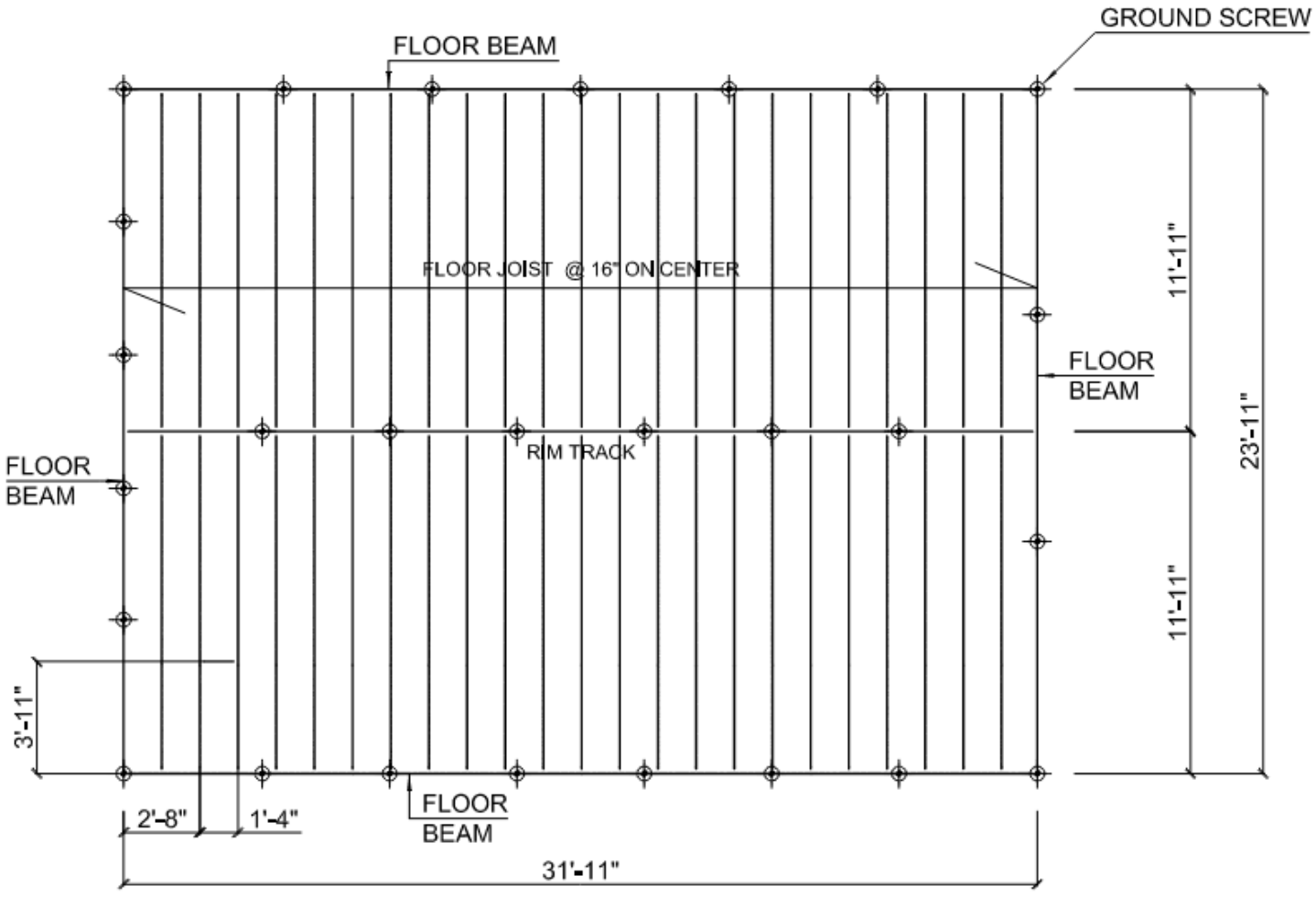


2.4.2 FLOOR FRAMING CHECK

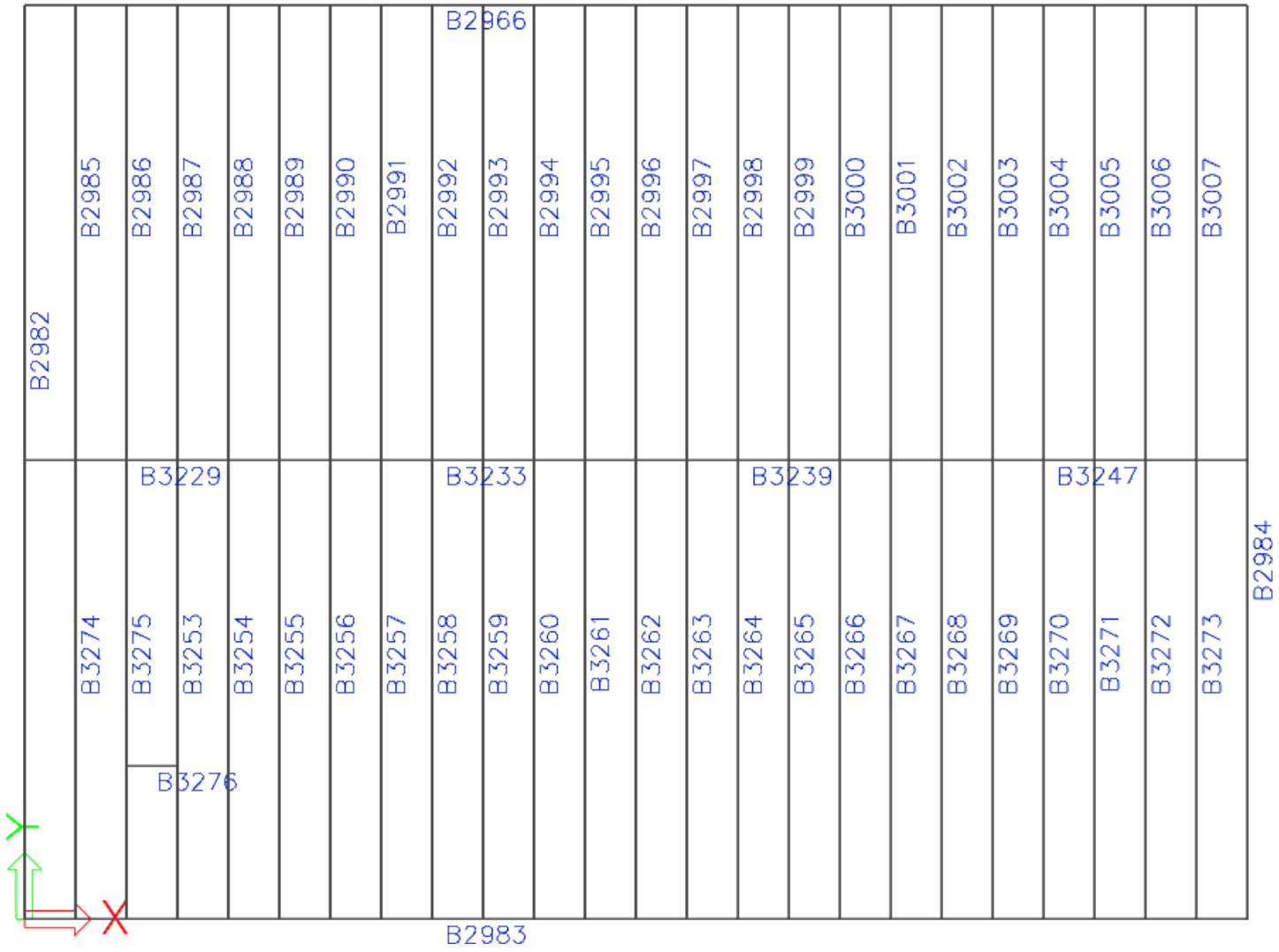
2.4.2.1 FIRST-STOREY FLOOR FRAMING CHECK

Floor joist spacing - 16";

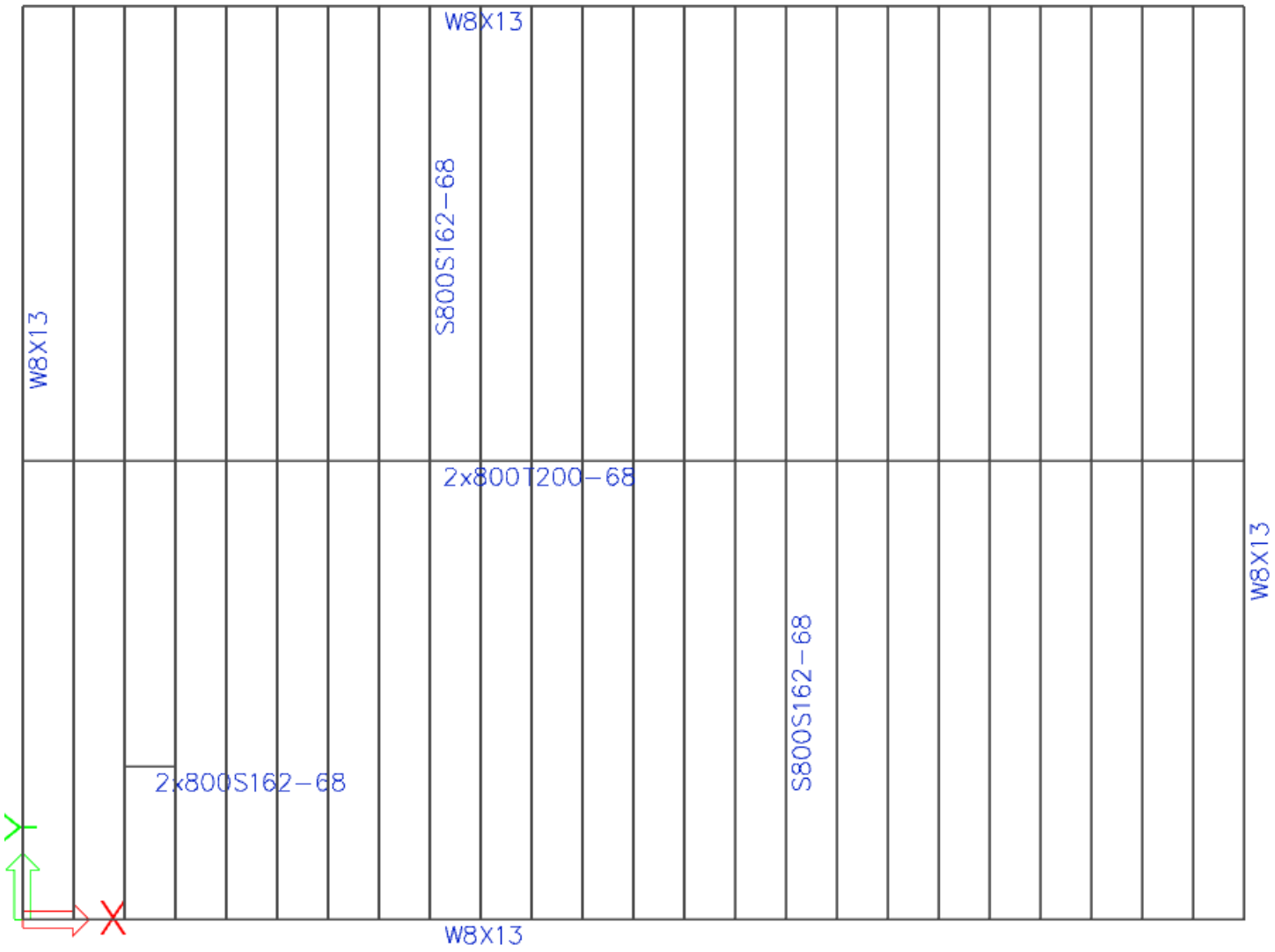
General view



Floor members number



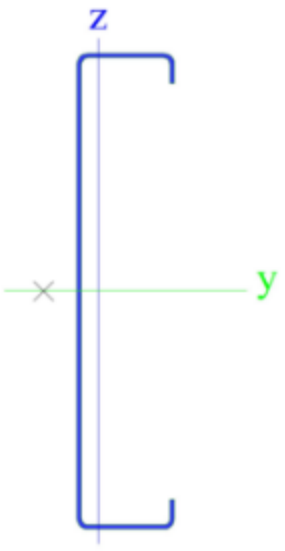
Floor members cross-sections



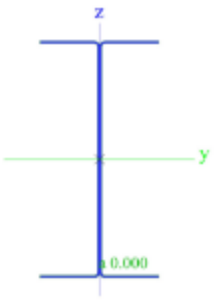
Cross-sections properties of W8x13

CS27		
Type	W8X13	
Formcode	1 - I section	
Shape type	Thin-walled	
Item material	A36	
Fabrication	rolled	
Colour		
A [inch^2]	3.840	
A_y [inch^2], A_z [inch^2]	2.131	1.858
A_L [inch^2/inch], A_D [inch^2/inch]	3.08e+01	3.08e+01
$c_{y,ucs}$ [inch], $c_{z,ucs}$ [inch]	2.000	3.995
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	39.600	2.730
i_y [inch], i_z [inch]	3.211	0.843
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	9.910	1.370
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	11.400	2.150
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	4.18e+02	4.18e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	7.80e+01	7.80e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.108	40.800
β_y [inch], β_z [inch]	0.000	0.000
Picture		

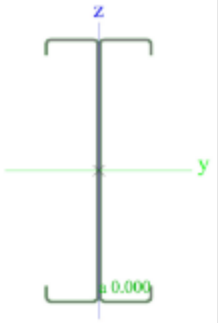
Cross-sections properties of 800S162-68

CS28		
Type	S800S162-68	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.840	
A_Y [inch^2], A_Z [inch^2]	0.230	0.565
A_L [inch^2/inch], A_D [inch^2/inch]	2.36e+01	2.36e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0.348	4.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	7.161	0.235
i_y [inch], i_z [inch]	2.920	0.529
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	1.771	0.184
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	2.175	0.269
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.09e+02	1.09e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.34e+01	1.34e+01
d_y [inch], d_z [inch]	-0.908	0.000
I_t [inch^4], I_w [inch^6]	0.001	3.093
β_y [inch], β_z [inch]	0.000	9.829
Picture		

Cross-sections properties of B2B (2)800T200-68

2x800T200-68		
Type	2CFUo	
Detailed	T800T200-68; 0.000	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.673	
A_y [inch^2], A_z [inch^2]	0.517	1.098
A_L [inch^2/inch], A_D [inch^2/inch]	3.19e+01	3.19e+01
$c_{y.ucs}$ [inch], $c_{z.ucs}$ [inch]	2.000	4.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	14.446	0.762
i_y [inch], i_z [inch]	2.938	0.675
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	3.612	0.381
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	4.391	0.610
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	2.20e+02	2.20e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	3.05e+01	3.05e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.008	11.863
β_y [inch], β_z [inch]	0.000	0.000
Picture		

Cross-sections properties of B2B (2)800S162-68

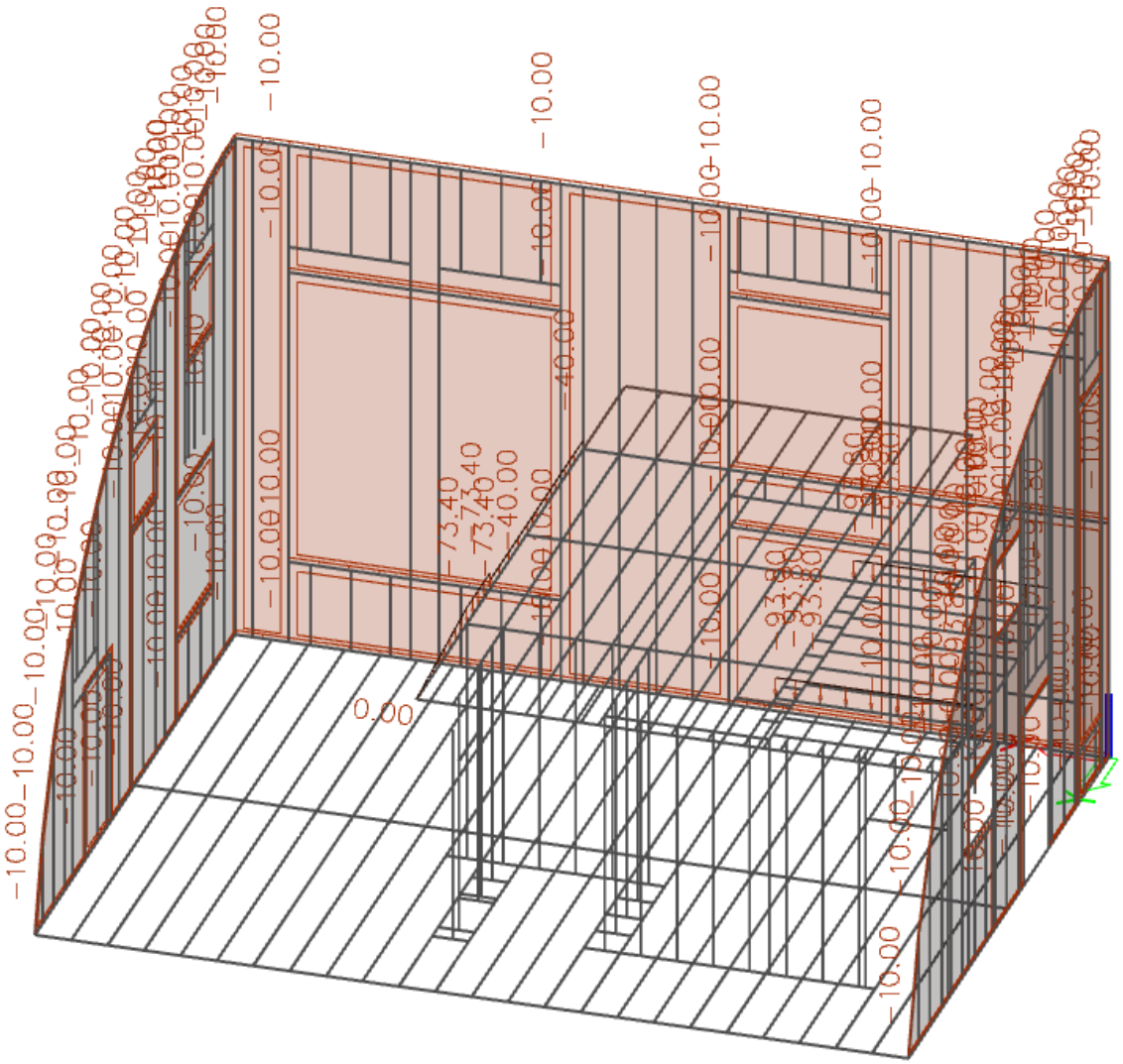
2x800S162-68		
Type	2CFCo	
Detailed	S800S162-68; 0.000	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1.671	
A_y [inch^2], A_z [inch^2]	0.471	1.108
A_L [inch^2/inch], A_D [inch^2/inch]	3.19e+01	3.19e+01
$c_{y,ucs}$ [inch], $c_{z,ucs}$ [inch]	1.625	4.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	14.171	0.672
i_y [inch], i_z [inch]	2.912	0.634
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	3.543	0.414
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	4.351	0.582
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	2.18e+02	2.18e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	2.91e+01	2.91e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.008	11.101
β_y [inch], β_z [inch]	0.000	0.000
Picture		

Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

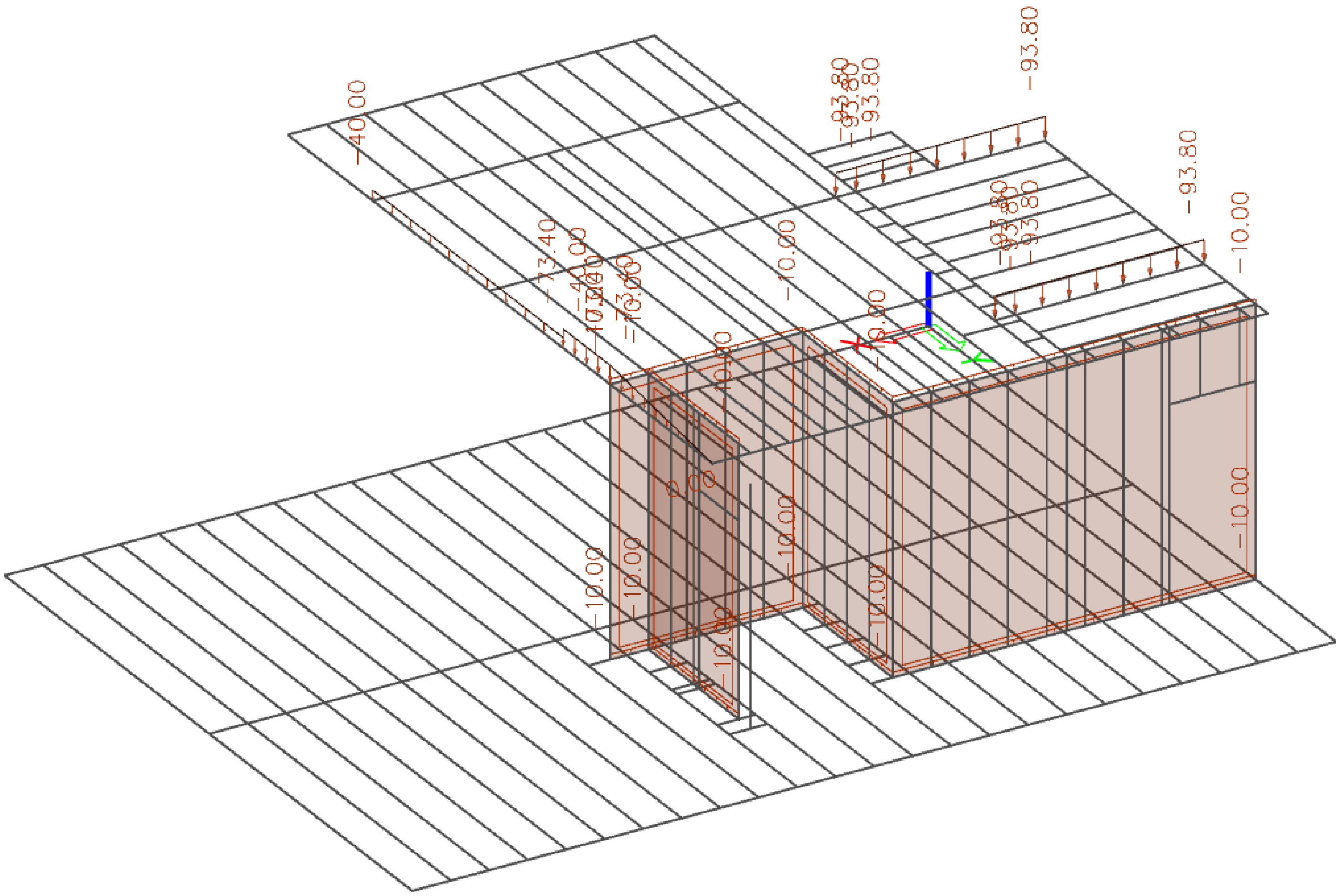
Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

APPLIED LOADS

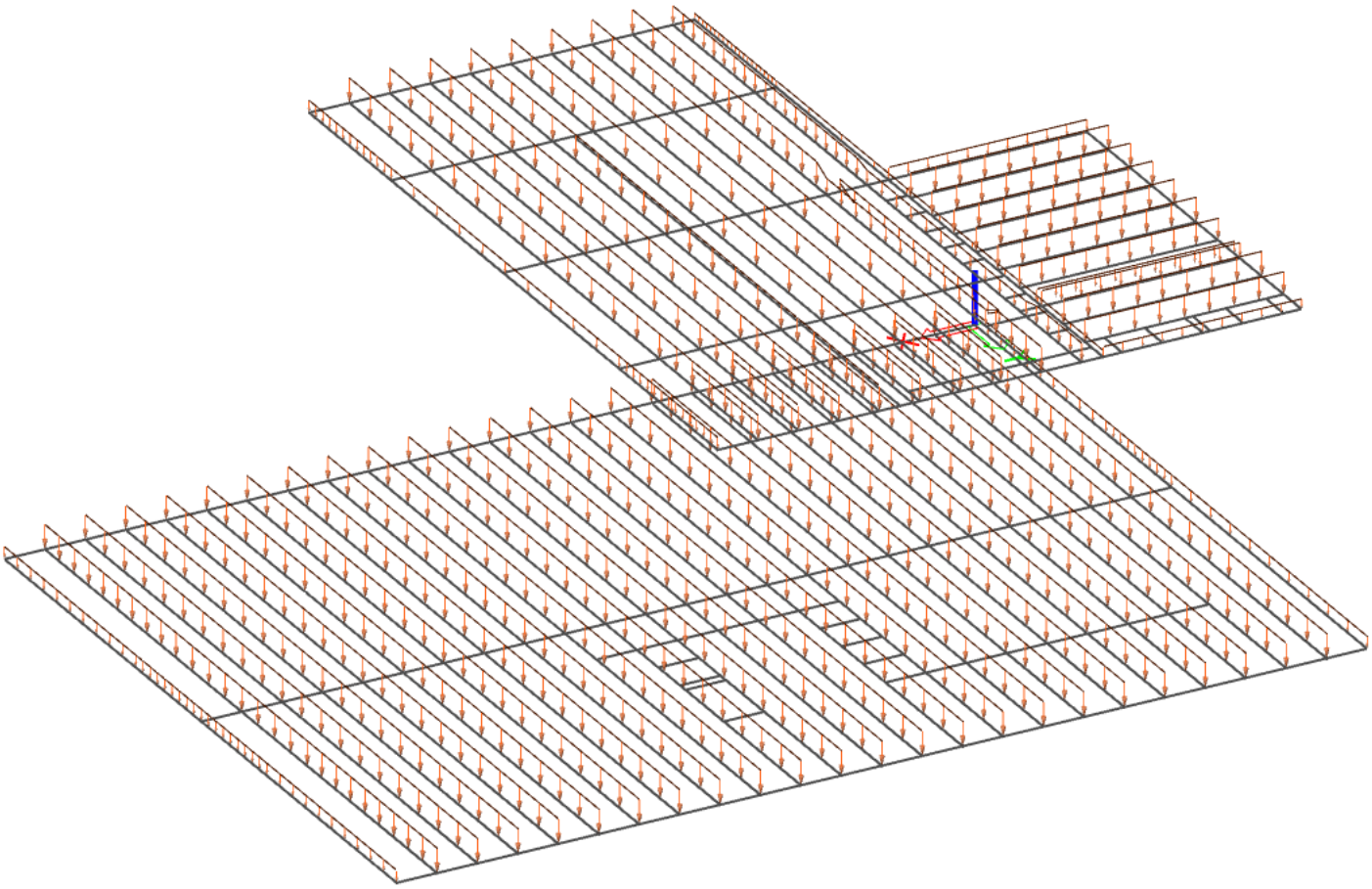
DL3 - Dead Load Wall.
Exterior wall load 10 psf



Interior wall load.

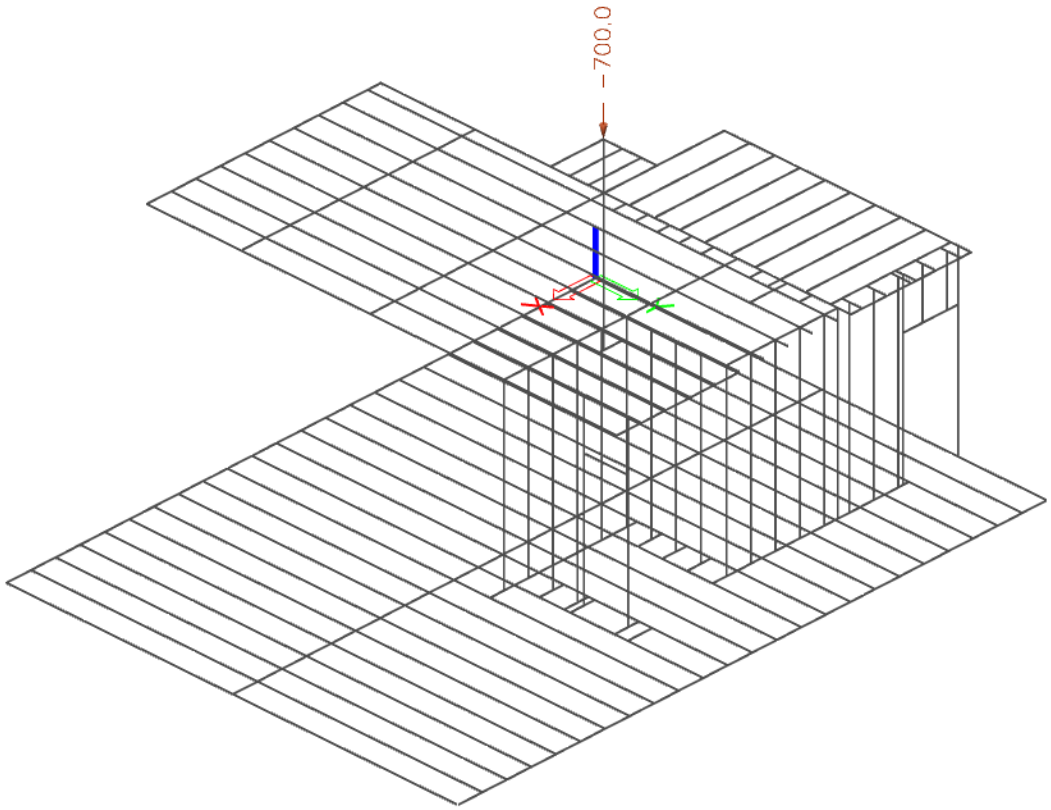


DL4 - Dead Load Floor.
8.6 psf * 16 in = 11.47 plf.

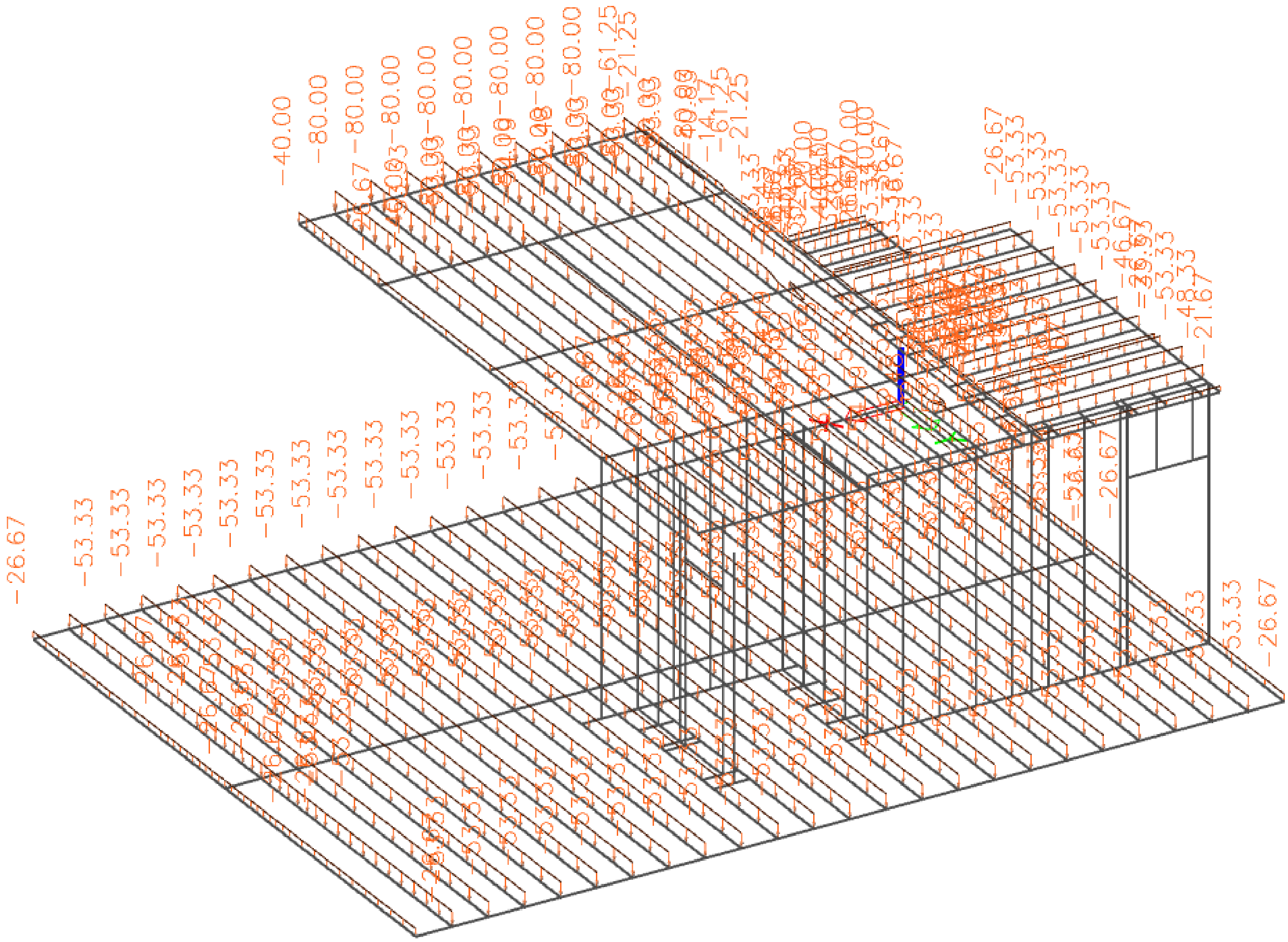


Dead load from spiral stairs, accepted 700 lbs.

Dead load from spiral stairs is included in load case DL4 - Dead Load Floor.

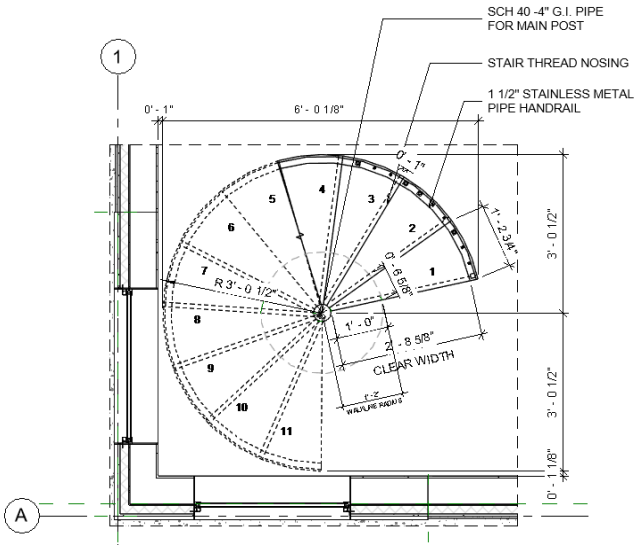


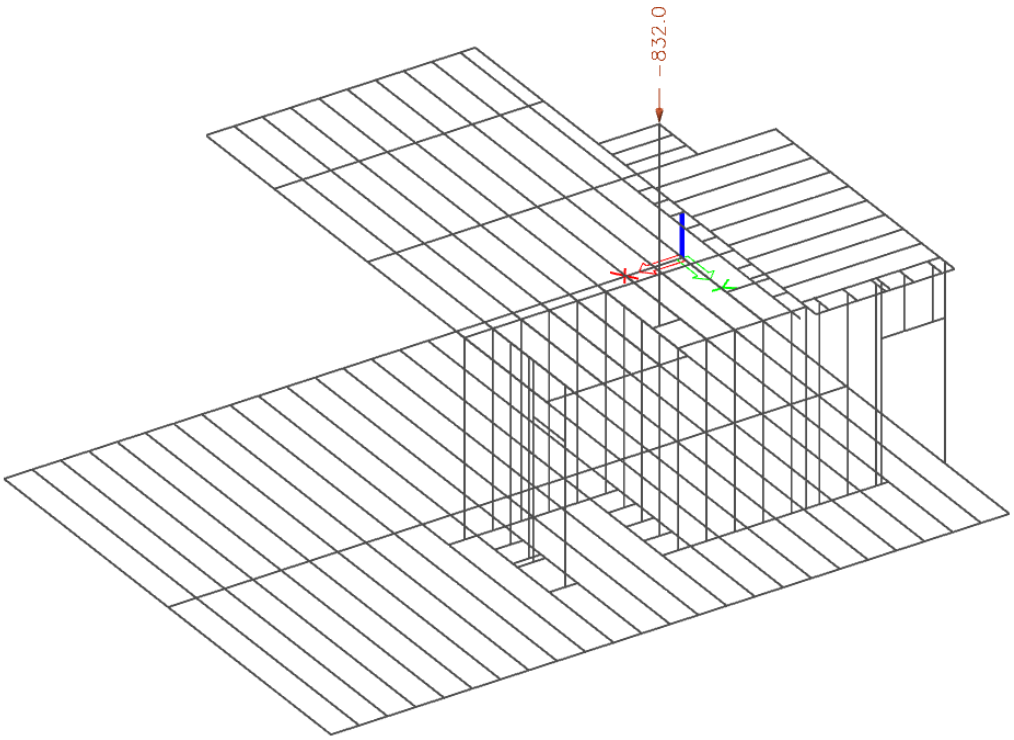
L - Floor Live Load, plf.
40 psf * 16 in = 53.33 plf.
50 psf * 16 in = 80 plf.



Load combinations see page 47

Live load from spiral stairs.
According to the sketch, the staircase has 11 steps. Single step area 1.89 sq ft.
Maximum live load 1.89 sq ft x 11 x 40 psf = 832 lb
Live load from spiral stairs is included in load case L - Floor Live Load.





Floor member maximum force

Shear force diagramVz,
LRFD-Ult (auto) 5 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1.6*L), lbf.

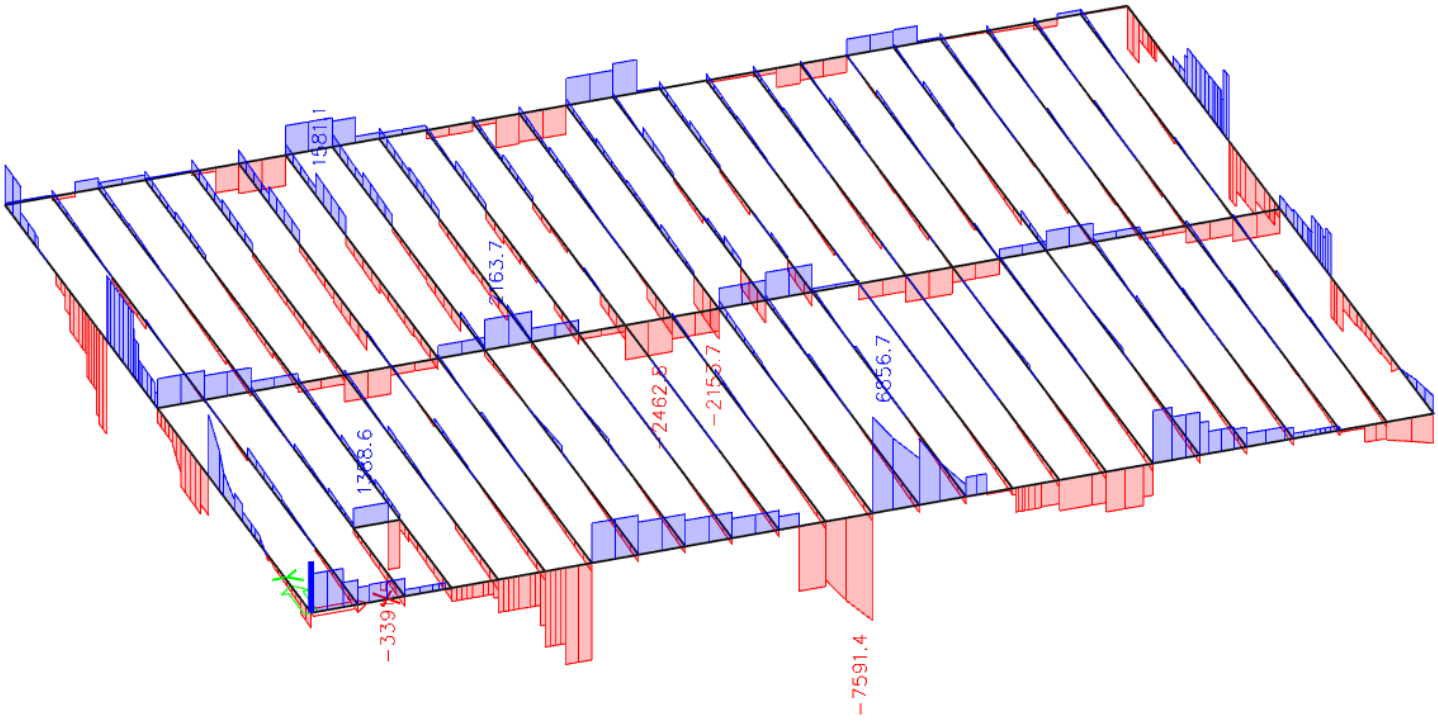
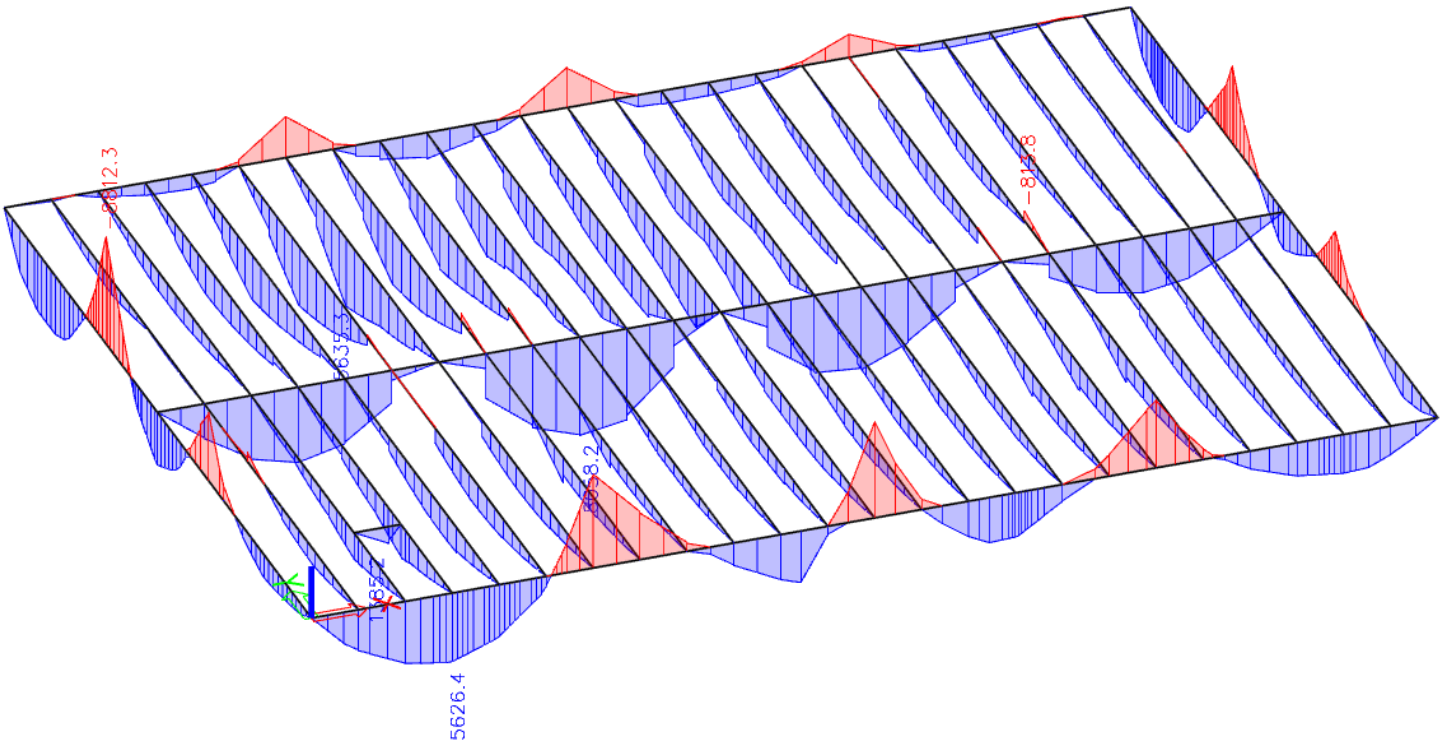
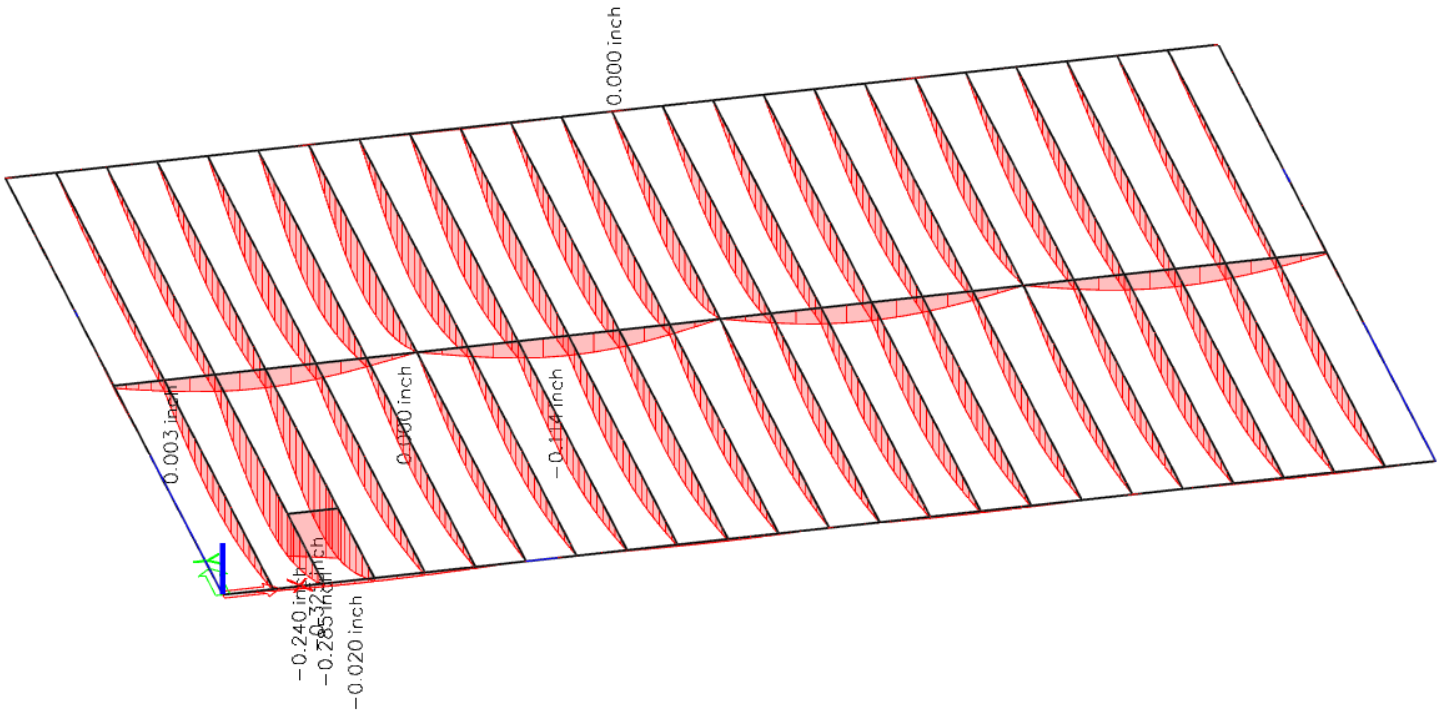


Diagram of bending moments My,
LRFD-Ult (auto) 5 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1.6*L), lbf*ft.



Floor member displacement
Load case L, inch:



The maximum deflection for beam W8x13 is 0.020”. According to TABLE R301.7 the code IRC 2018, maximum member deflection should not exceed - L/360.
L=8’=96”, 96”/360=0.267”. 0.020"< 0.267”. Deflection is OK!

The maximum deflection for joist 800S162-68 is 0.325”. According to TABLE R301.7 the code IRC 2018, maximum member deflection should not exceed - L/360.
L=11’-11”=143”, 143”/360=0.397”. 0.325"< 0.397”. Deflection is OK!

The maximum deflection for rim joist 800T200-68 is 0.114”. According to TABLE R301.7 the code IRC 2018, maximum member deflection should not exceed - L/360.
L=8’=96”, 96”/360=0.267”. 0.114"< 0.267”. Deflection is OK!

STEEL MEMBER B2982 CHECK

AISC 360-16 LRFD Check

Member B2982	W8X13	A36	LRFD-Ult (auto)	0.39

Material data		
Yield stress Fy	36.00	ksi
Tensile stress Fu	58.00	ksi
fabrication	rolled	

The critical check is on position 7.92 ft

Classification for Flexure

according to article B4 and Table B4.1b

Bending Mx	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Webs	29.07	106.73	161.80
Outstanding flanges	7.84	10.79	28.39

Section is classified as compact section.

Bending My	ratio	Lambda,p (Compact)	Lambda,r (Non-compact)
Outstanding flanges	7.84	10.79	28.39

Section is classified as compact section.

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	23.39	lbf
Vux	-1.34	lbf
Vuy	5126.53	lbf
Mut	25.67	lbfft
Mux	-10313.27	lbfft
Muy	1.35	lbfft

Normal force check

according to article D2 and formula (D2-1)

Table of values		
Pn	138239.77	lbf
Pu	23.39	lbf
Resistance factor	0.90	
unity check	0.00	

LTB data		
Lb	1.33	ft
Cb	1.36	

Strong axis bending check
according to article F2 and formula (F2-1)

Table of values		
Lr	12.32	ft
Lp	3.51	ft
Mn	34200.09	lbfft
Mu	-10313.27	lbfft
Resistance factor	0.90	
unity check	0.34	

Weak axis bending check
according to article F6 and formula (F6-1)

Table of values		
Mn	6450.00	lbfft
Mu	-1.35	lbfft
Resistance factor	0.90	
unity check	0.00	

Shear stress check
according to article G2 and formula (G2-1)
in buckling field 1

Table of values		
a	23.92	ft
h	0.56	ft
tw	0.23	inch
kv	5.34	
Vn	39694.33	lbf
Vu	5126.53	lbf
Resistance factor	1.00	
unity check	0.13	

Combined stresses check
according to article H1.2 and formula (H1-1b)

Table of values		
Pr	23.39	lbf
Mrx	-10313.27	lbfft
Mry	-1.35	lbfft
Pc	124415.80	lbf
Mcx	30780.08	lbfft
Mcy	5805.00	lbfft
Res. factor tension	0.90	
Res. factor flexure	0.90	

unity check = 0.00+0.34+0.00=0.34 (H1-1b)
Members under torsion and combined torsion. Flexure,shear and/or Axial force as per H3.
according to article H3 and formula (H3-7)

Table of values		
Mux	-10313.27	lbfft
Muy	-1.35	lbfft
fun	12.50	ksi
Fny	36.00	ksi
Res. factor torsion	0.90	

unity check = 12.50/32.40=0.39 (H3-7)
according to article H3 and formula (H3-8)
Critical fibre position 8

Table of values		
Vux	1.34	lbf
Vuy	5126.53	lbf
Mut	25.67	lbfft
fsx	0.000	lbf/ft^2
fsy	90.057	lbf/ft^2
fT	3675.930	lbf/ft^2
Stress	3.86	ksi
Fn	21.60	ksi
Res. factor torsion	0.90	

unity check = 3.86/19.44=0.20 (H3-8)
according to article H3 and formula (H3-9)

Table of values		
Pux	23.39	lbf
fua	0.01	ksi
Fnb	36.00	ksi
Res. factor torsion	0.90	

unity check = 0.01/32.40=0.00 (H3-9)

The member satisfies the check !

STEEL MEMBER B2990 CHECK

AISI S100-16 LRFD Check

Member B2990	S800S162-68	A913 grade 50	LRFD-Ult (auto)	0.96

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 4.71 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	0.22	lbf
Vux	-0.00	lbf
Vuy	-646.90	lbf
Mut	0.03	lbfft
Mux	5569.79	lbfft
Muy	0.01	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.322	-40.981 -44.903	-	-	-	-	-	- -	- -	-	- -	-
3	1.269	-46.641 -46.641	-	-	-	-	-	- -	- -	-	- -	-
5	7.644	48.262 -44.903	0.93	22.248	50.753	0.975	0.794	- 6.070	1.544 1.600	-	- -	-
7	1.269	50.000 50.000	1.00	3.279	271.583	0.429	1.000	1.269 -	0.634 0.634	30.83	0.000 0.000	0.322
9	0.322	48.262 44.340	0.92	0.459	591.004	0.286	1.000	0.322 -	- -	-	- -	-

Table of values		
Sxe	1.659	inch^3
Mnxo	6912.31	lbfft
Resistance factor	0.90	
Unity check	0.90	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	2 ft 8.000 in	ft
Sigma,ey	78.161	ksi
Kt	1.00	
Lt	2 ft 8.000 in	ft
Sigma,t	108.900	ksi
Cb	1.16	
Sfx	1.790	$inch^3$
Fcre	156.065	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
Distortional Buckling Strength
According to article F4 and formula F4.1-2.

Table of values		
Sfy	1.790	$inch^3$
My	7459.79	lbfft
L	1 ft 0.536 in	ft
Beta	1.10	
k,phi,fe	511.74	lbf
k,phi,we	522.36	lbf
k,phi	0.00	lbf
k,phi,fg	0.011	$inch^2$
k,phi,wg	0.006	$inch^2$
Fd	68.153	ksi

Table of values		
Sf	1.790	$inch^3$
Mcrd	10168.14	lbfft
Lambda,d	0.86	
Mn	6472.33	lbfft
Resistance factor	0.90	
Unity check	0.96	-

Data		
Lm	2 ft 8.000 in	ft
Lcr	1 ft 0.536 in	ft
h0	8.000	inch
Ixf	0.002	$inch^4$
Iyf	0.028	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.532	inch
hxf	-1.004	inch
Af	0.129	$inch^2$
y0f	0.050	inch
Ksi,web	2.00	

Number of compressed flanges: 1
Critical flange contains Initial shape parts: 8, 7, 9

...::Shear Strength::...

Shear Strength

According to article G2.1 and formula (G2.1.1)

Shear force Vy

Element ID	Aw [inch^2]	Vn [lbf]
3	0.000	0.00
5	0.545	6640.36
7	0.000	0.00

Table of values		
Vn,y	6640.36	lbf
Resistance factor	0.95	
Unity check	0.10	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	6912.31	lbfft
Vny	6640.36	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.80+0.01) = 0.90

Note: Web Crippling has been ignored due to user input

The member satisfies the check !

STEEL MEMBER B3233 CHECK

AISI S100-16 LRFD Check

Member B3233	2CFUo (T(SSMA)800T200-68; 0.000)	A913 grade 50	LRFD-Ult (auto)	0.86

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 4.00 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	0.01	lbf
Vux	0.06	lbf
Vuy	-455.61	lbf
Mut	-0.09	lbfft
Mux	8128.95	lbfft
Muy	-0.04	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	1.822	-37.392 -37.392	-	-	-	-	-	- -	- -	-	- -	-
3	7.644	48.429 -35.821	0.74	18.009	41.083	1.086	0.734	- 5.614	1.501 1.726	-	- -	-
5	1.822	50.000 50.000	1.00	0.430	17.268	1.702	0.512	0.932 -	- -	-	- -	-
6	1.822	50.000 50.000	1.00	0.430	17.268	1.702	0.512	0.932 -	- -	-	- -	-
8	7.644	48.429 -35.821	0.74	18.009	41.083	1.086	0.734	- 5.614	1.501 1.726	-	- -	-
10	1.822	-37.392 -37.392	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	2.510	inch^3
Mnxo	10458.66	lbfft
Resistance factor	0.90	
Unity check	0.86	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	1 ft 4.000 in	ft
Sigma,ey	509.638	ksi
Kt	1.00	
Lt	1 ft 4.000 in	ft
Sigma,t	878.302	ksi
Cb	1.03	
Sfx	3.612	$inch^3$
Fcre	963.551	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...
Shear Strength
According to article G2.1 and formula (G2.1.1)
Shear force Vy

Element ID	Aw [$inch^2$]	Vn [lbf]
1	0.000	0.00
3	0.545	6640.36
5	0.000	0.00
6	0.000	0.00
8	0.545	6640.36
10	0.000	0.00

Table of values		
Vn,y	13280.71	lbf
Resistance factor	0.95	
Unity check	0.04	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnxo	10458.66	lbfft
Vny	13280.71	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.75+0.00) = 0.86

The member satisfies the check !

STEEL MEMBER B3276 CHECK

AISI S100-16 LRFD Check

Member B3276	2CFCo (S(SSMA)800S162-68; 0.000)	A913 grade 50	LRFD-Ult (auto)	0.28

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 1.00 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	0.56	lbf
Vux	-0.06	lbf
Vuy	-3389.26	lbf
Mut	1.99	lbfft
Mux	1130.13	lbfft
Muy	0.02	lbfft

Combined Bending and Torsional Loading
According to article H4 and formula (H4-1)

Table of values		
Critical fibre	11	
Sigma Mx	3.949	ksi
Sigma My	0.000	ksi
f bending	3.949	ksi
Tau t	-0.210	ksi
f torsion	-0.210	ksi
Composed Stress	3.966	ksi
R	1.00	-

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.322	-40.981 -44.903	-	-	-	-	-	- -	- -	-	- -	-
3	1.269	-46.641 -46.641	-	-	-	-	-	- -	- -	-	- -	-
5	7.644	48.262 -44.903	0.93	22.248	50.753	0.975	0.794	- 6.070	1.544 1.600	-	- -	-
7	1.269	50.000 50.000	1.00	3.279	271.583	0.429	1.000	1.269 -	0.634 0.634	30.83	0.000 0.000	0.322

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
9	0.322	48.262 44.340	0.92	0.459	591.004	0.286	1.000	0.322 -	- -	-	- -	-
10	0.322	48.262 44.340	0.92	0.459	591.004	0.286	1.000	0.322 -	- -	-	- -	-
12	1.269	50.000 50.000	1.00	3.279	271.583	0.429	1.000	1.269 -	0.634 0.634	30.83	0.000 0.000	0.322
14	7.644	48.262 -44.903	0.93	22.248	50.753	0.975	0.794	- 6.070	1.544 1.600	-	- -	-
16	1.269	-46.641 -46.641	-	-	-	-	-	- -	- -	-	- -	-
18	0.322	-40.981 -44.903	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	3.318	inch^3
Mnxo	13766.6	lbfft
Resistance factor	0.90	
Unity check	0.09	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	4.000 in	ft
Sigma,ey	7201.372	ksi
Kt	1.00	
Lt	4.000 in	ft
Sigma,t	13387.793	ksi
Cb	1.67	
Sfx	3.543	inch^3
Fcre	22987.463	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
Distortional Buckling Strength
According to article F4 and formula F4.3-1.

Table of values		
Sfy	3.543	inch^3
My	14761.6	lbfft
L	4.000 in	ft
Beta	1.00	
k,phi,fe	35608.6	lbf
k,phi,we	2497.2	lbf
k,phi	0.0	lbf
k,phi,fg	0.105	inch^2
k,phi,wg	0.052	inch^2
Fd	241.564	ksi
Sf	3.543	inch^3

Mcrd	71317.1	lbfft
Lambda,d	0.45	
Cyd	1.22	
Mp	18127.3	lbfft
cyt	3.00	
Myc	14761.6	lbfft
Myt3	0.0	lbfft
Mn	17753.3	lbfft
Resistance factor	0.90	
Unity check	0.07	-

Data		
Lm	4.000 in	ft
Lcr	1 ft 0.385 in	ft
h0	7.644	inch
Ixf	0.002	$inch^4$
Iyf	0.028	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.532	inch
hxf	-1.004	inch
Af	0.129	$inch^2$
y0f	0.050	inch
Ksi,web	2.00	

Number of compressed flanges: 2
Critical flange contains Initial shape parts: 8, 7, 9

...::Shear Strength::...
Shear Strength
According to article G2.1 and formula (G2.1.1)
Shear force Vy

Element ID	Aw [$inch^2$]	Vn [lbf]
3	0.000	0.0
5	0.545	6640.4
7	0.000	0.0
12	0.000	0.0
14	0.545	6640.4
16	0.000	0.0

Table of values		
Vn,y	13280.7	lbf
Resistance factor	0.95	
Unity check	0.27	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnxo	13766.6	lbfft
Vny	13280.7	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.01+0.07) = 0.28

Note: Web Crippling has been ignored due to user input

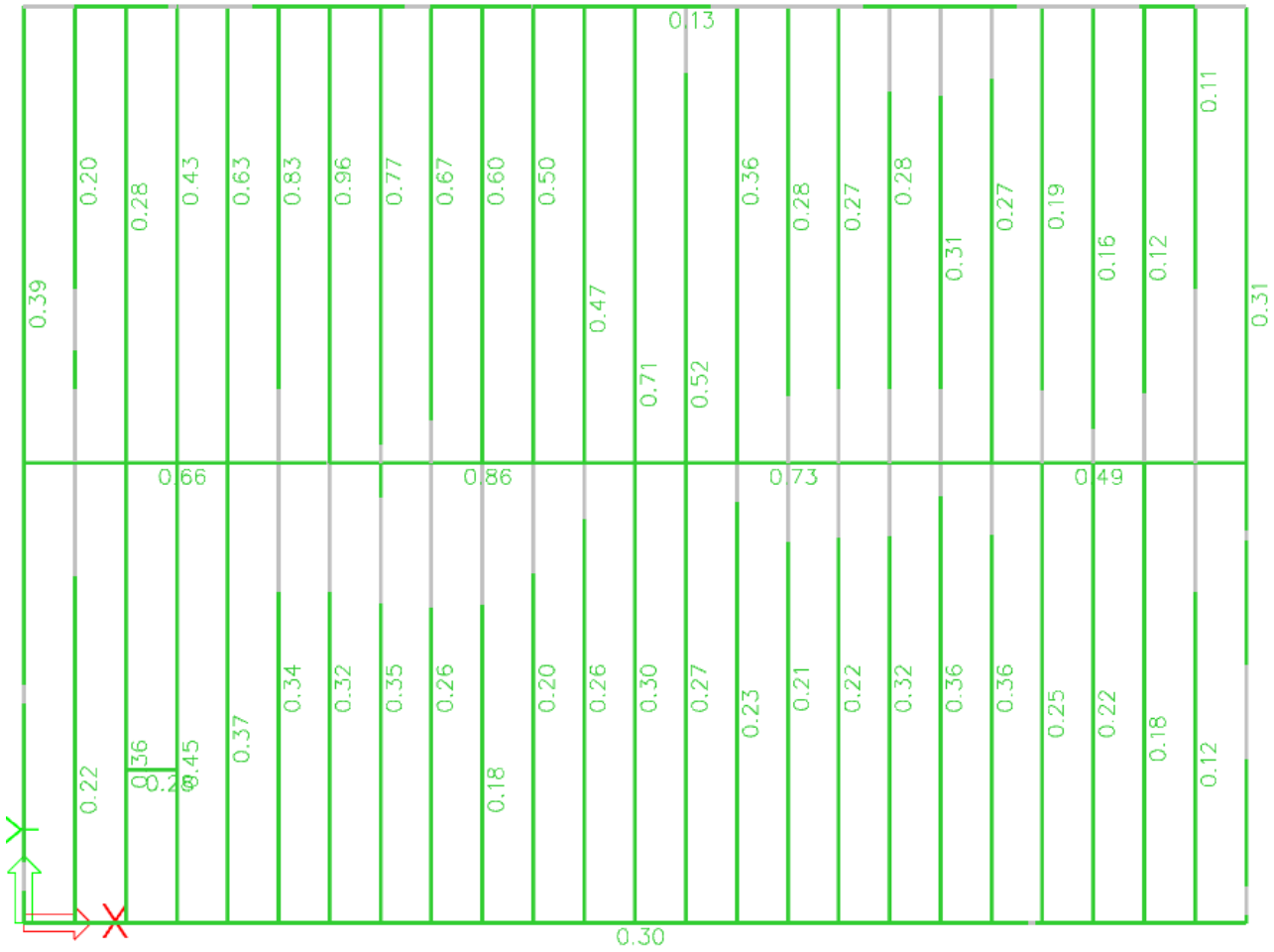
Combined Tensile Axial Load and Bending
According to article H1.1 and formulas (H1.1-1), (H1.1-2)

Table of values		
Sftx	3.543	$inch^3$
Mnxt	14761.6	lbfft
Mnx	13766.6	lbfft
Tn	83531.7	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	

Unity check = 0.09+0.00+0.00 = 0.09 - (H1.1-1)
Unity check = 0.09+0.00-0.00 = 0.09 - (H1.1-2)

The member satisfies the check !

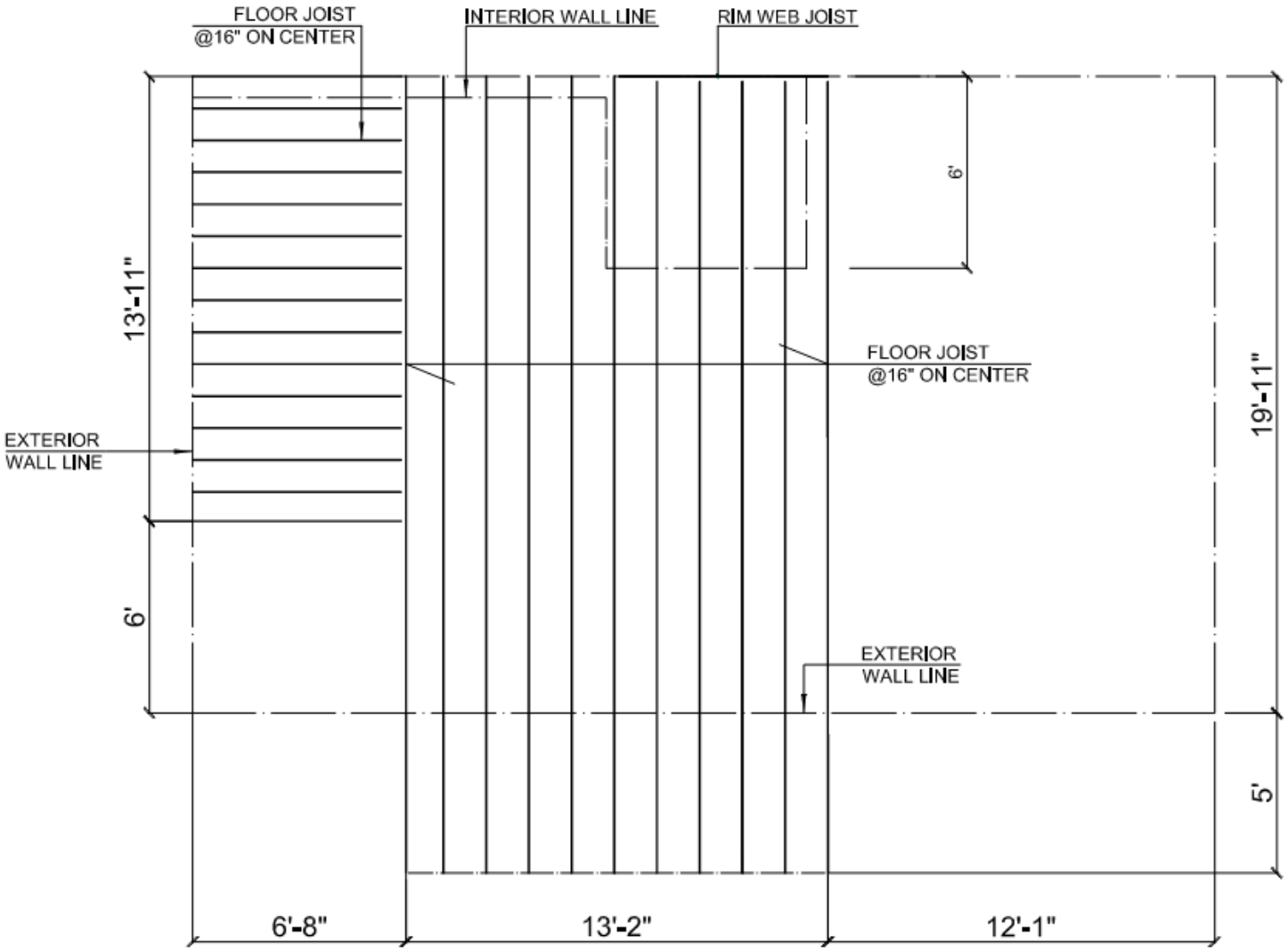
Unity check



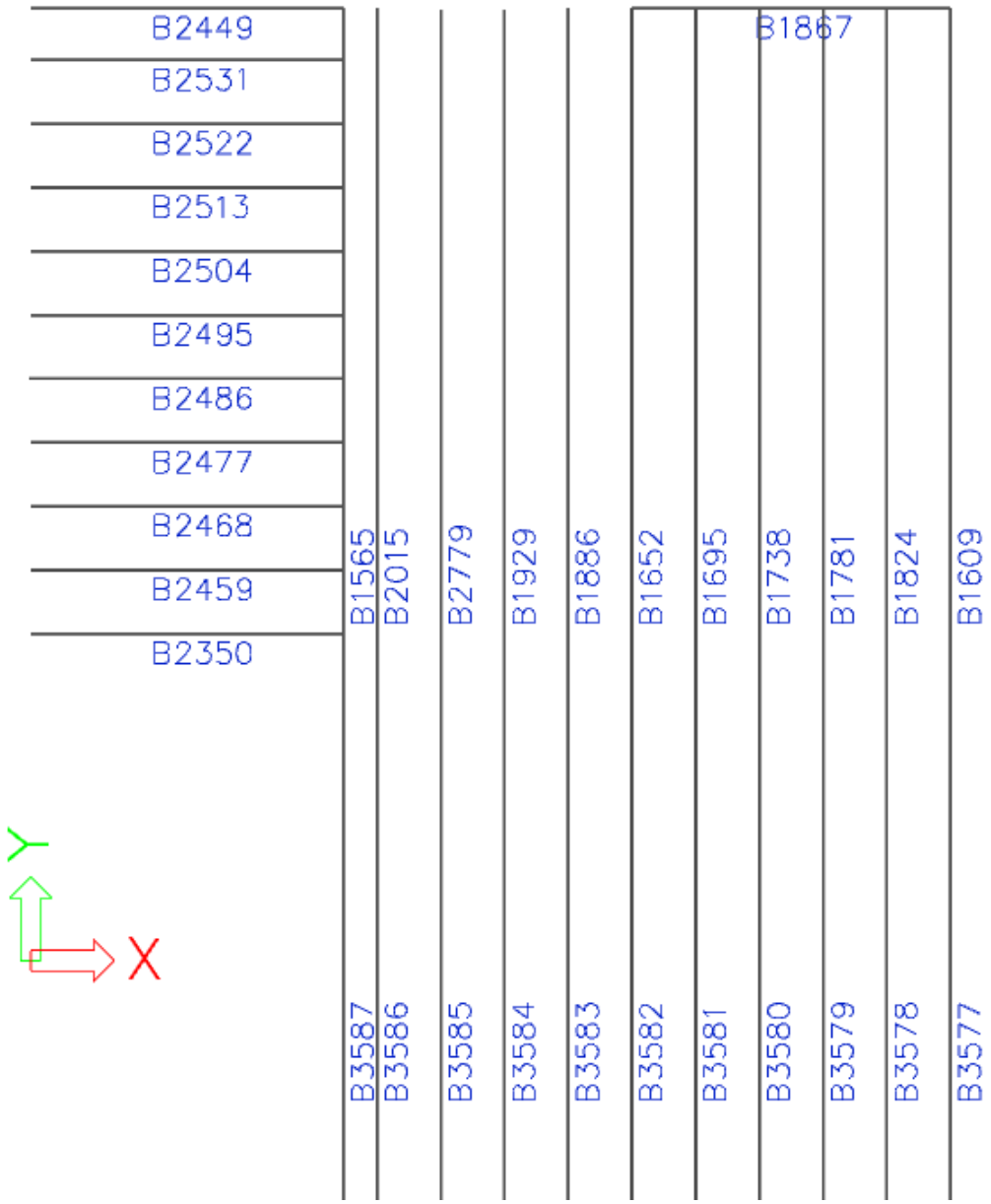
2.4.2.2 SECOND-STOREY FLOOR FRAMING CHECK

Floor joist spacing - 16";

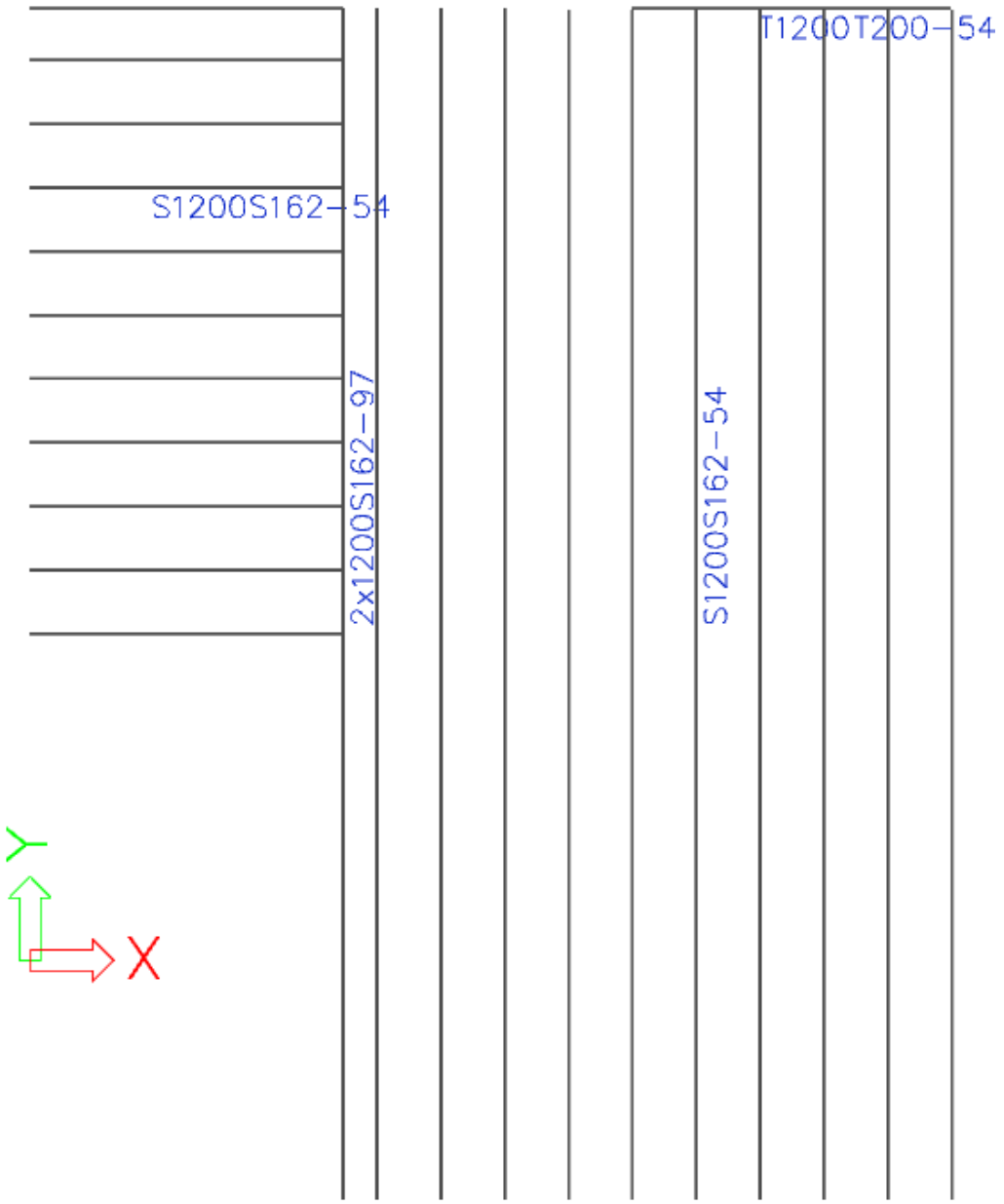
General view



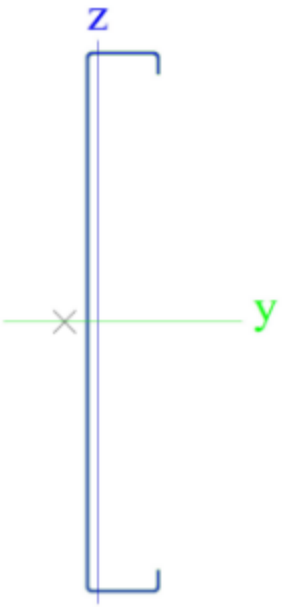
Floor members number



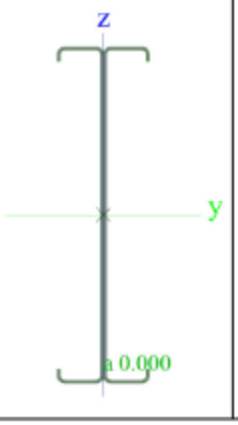
Floor members cross-sections



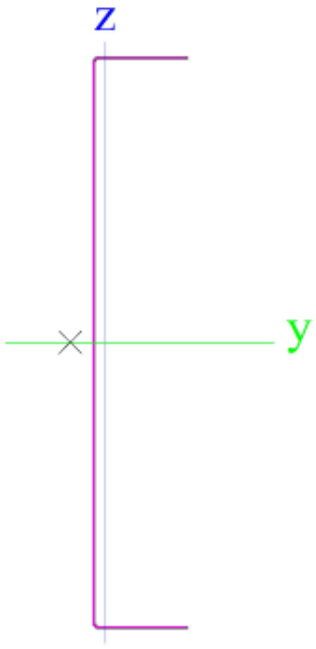
Cross-sections properties of 1200S162-54

CS33		
Type	S1200S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.899	
A_y [inch^2], A_z [inch^2]	0.188	0.659
A_L [inch^2/inch], A_D [inch^2/inch]	3.18e+01	3.18e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0.267	6.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	15.835	0.212
i_y [inch], i_z [inch]	4.198	0.486
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	2.621	0.156
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	3.318	0.223
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.66e+02	1.66e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.11e+01	1.11e+01
d_y [inch], d_z [inch]	-0.739	0.000
I_t [inch^4], I_w [inch^6]	0.001	6.340
β_y [inch], β_z [inch]	0.000	19.591
Picture		

Cross-sections properties of 2x1200S162-97

2x1200S162-97		
Type	2CFCo	
Detailed	S1200S162-97; 0.000	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	3.150	
A_Y [inch^2], A_Z [inch^2]	0.698	2.300
A_L [inch^2/inch], A_D [inch^2/inch]	3.94e+01	3.94e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	1.625	6.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	53.895	0.898
i_y [inch], i_z [inch]	4.136	0.534
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	8.982	0.553
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	11.499	0.855
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	5.75e+02	5.75e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	4.28e+01	4.28e+01
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.034	32.474
β_y [inch], β_z [inch]	0.000	0.000
Picture		

Cross-sections properties of 1200T200-54

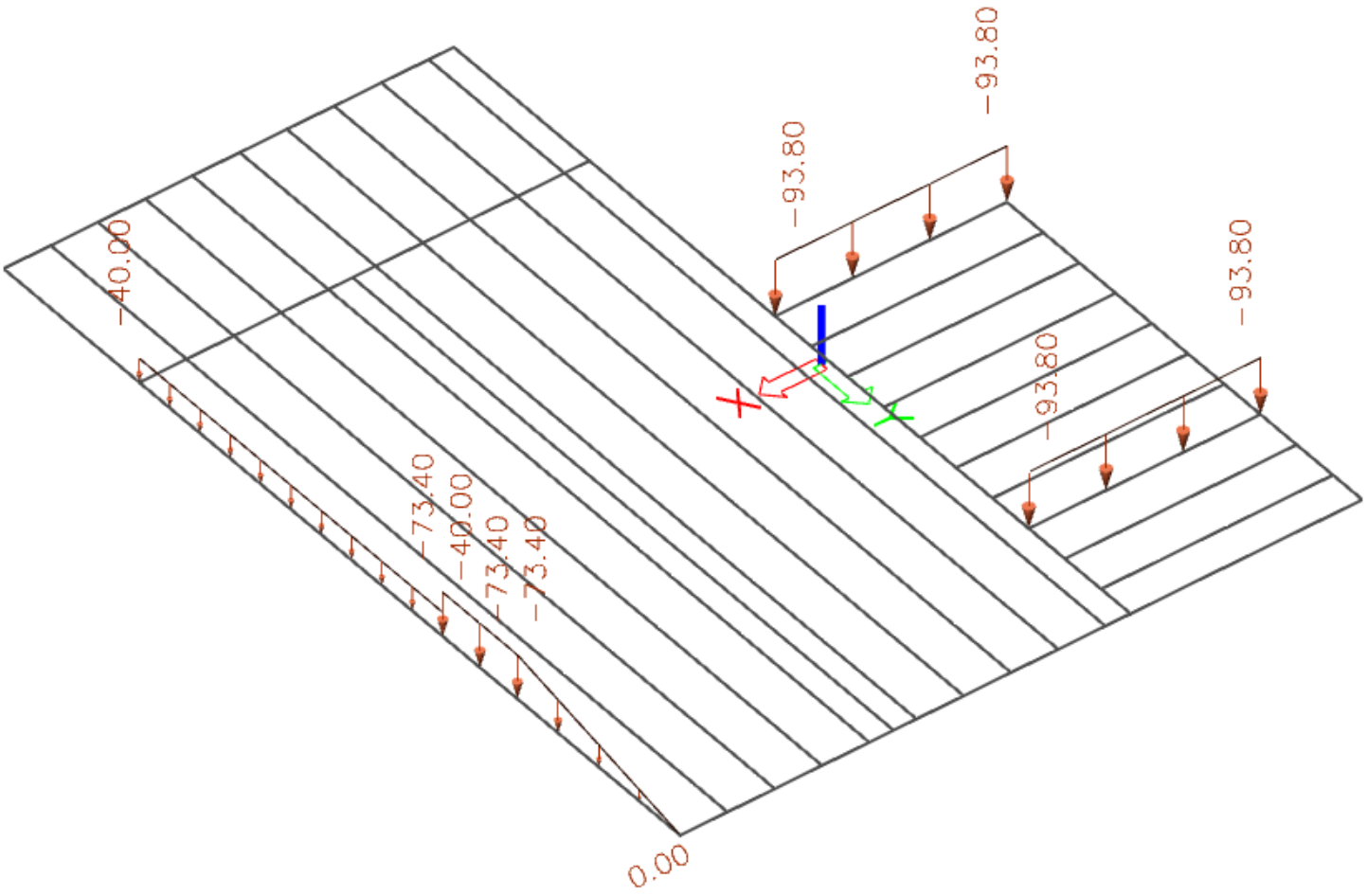
CS35		
Type	T1200T200-54	
Formcode	112 - Cold formed channel section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0.895	
A_y [inch^2], A_z [inch^2]	0.211	0.662
A_L [inch^2/inch], A_D [inch^2/inch]	3.17e+01	3.17e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0.275	6.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	15.853	0.235
i_y [inch], i_z [inch]	4.209	0.513
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	2.632	0.136
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	3.318	0.229
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.66e+02	1.66e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.14e+01	1.14e+01
d_y [inch], d_z [inch]	-0.735	0.000
I_t [inch^4], I_w [inch^6]	0.001	6.465
β_y [inch], β_z [inch]	0.000	19.325
Picture		

Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

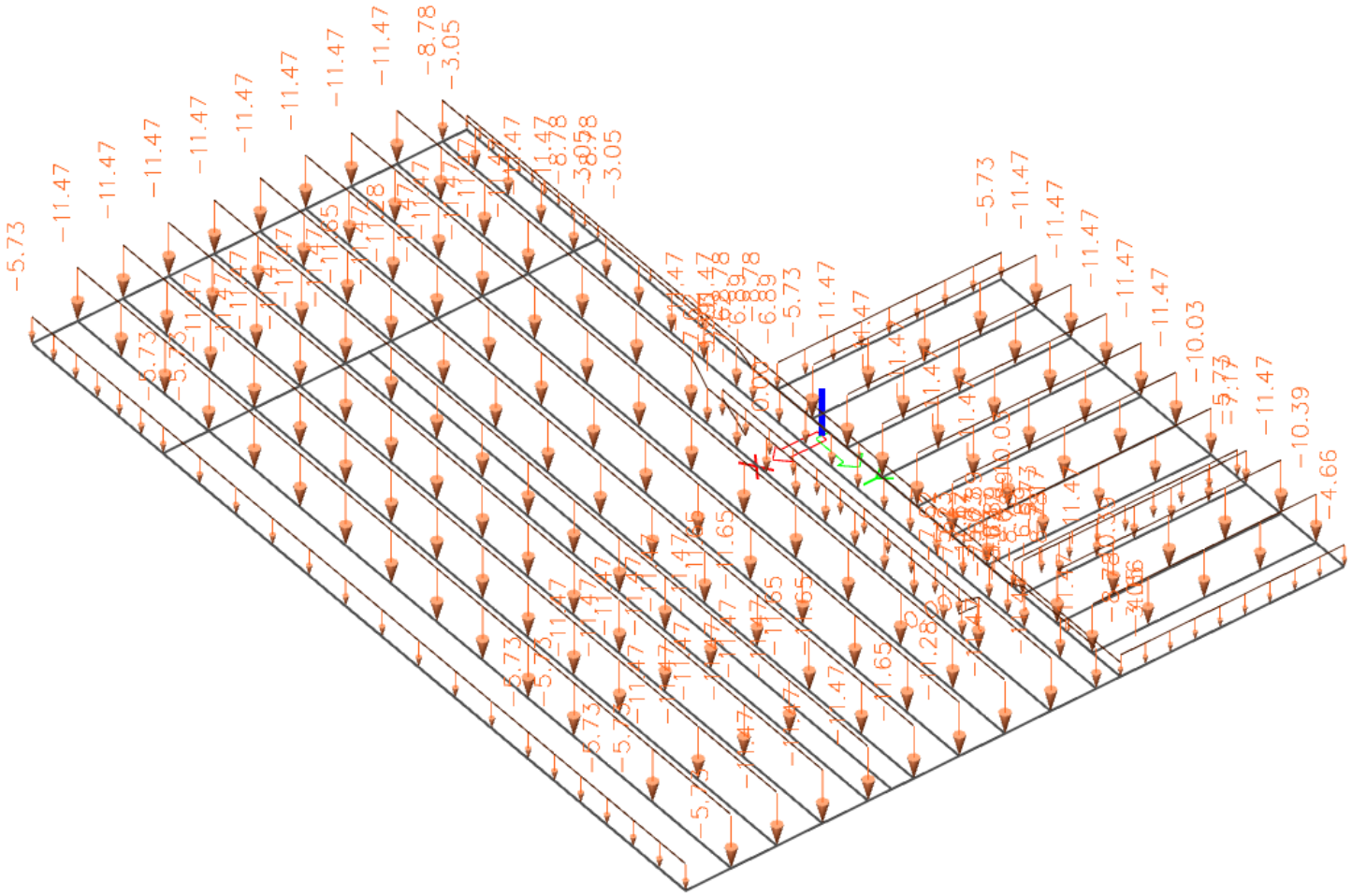
Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

APPLIED LOADS

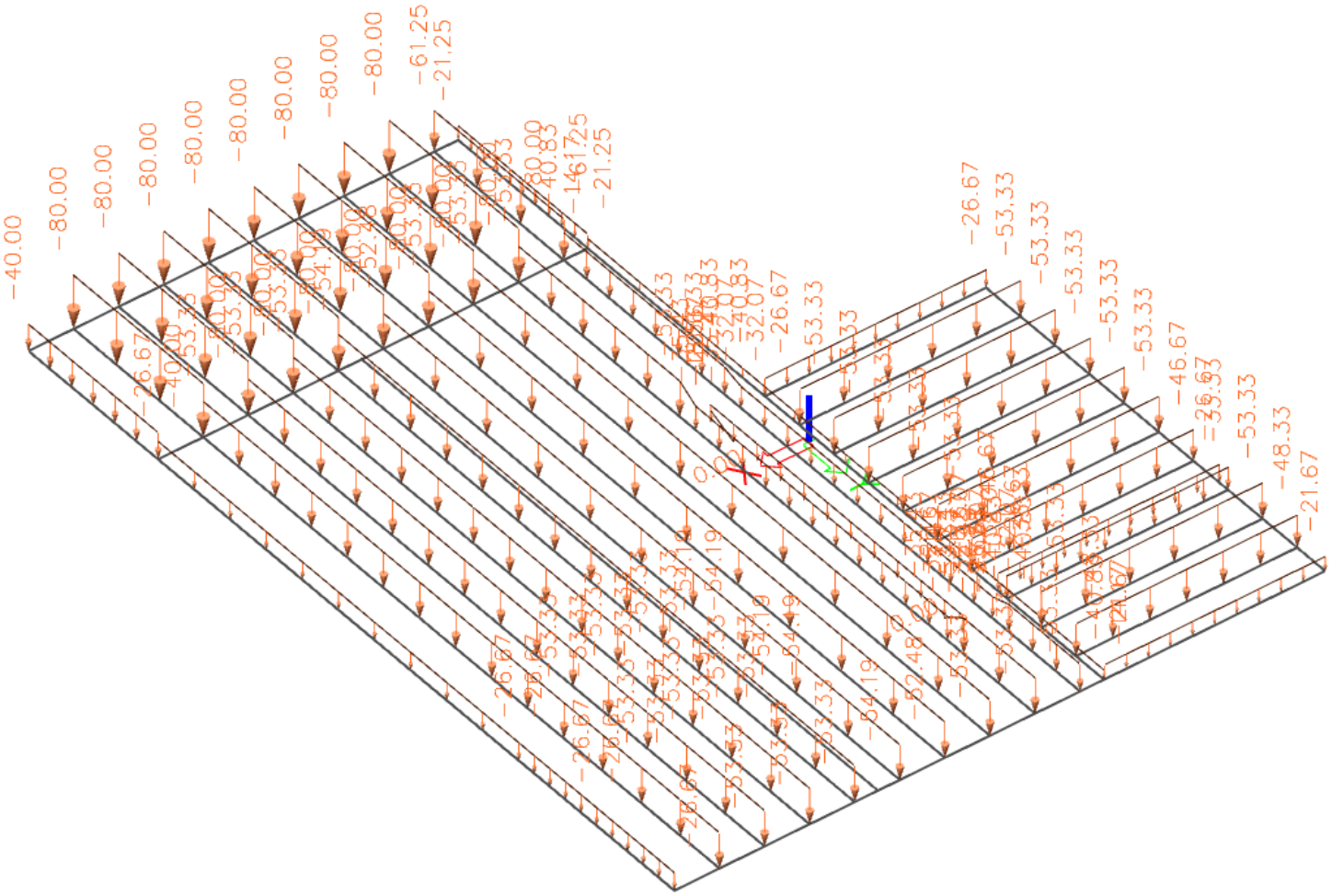
DL3 - Dead Load Wall.
Interior wall load.



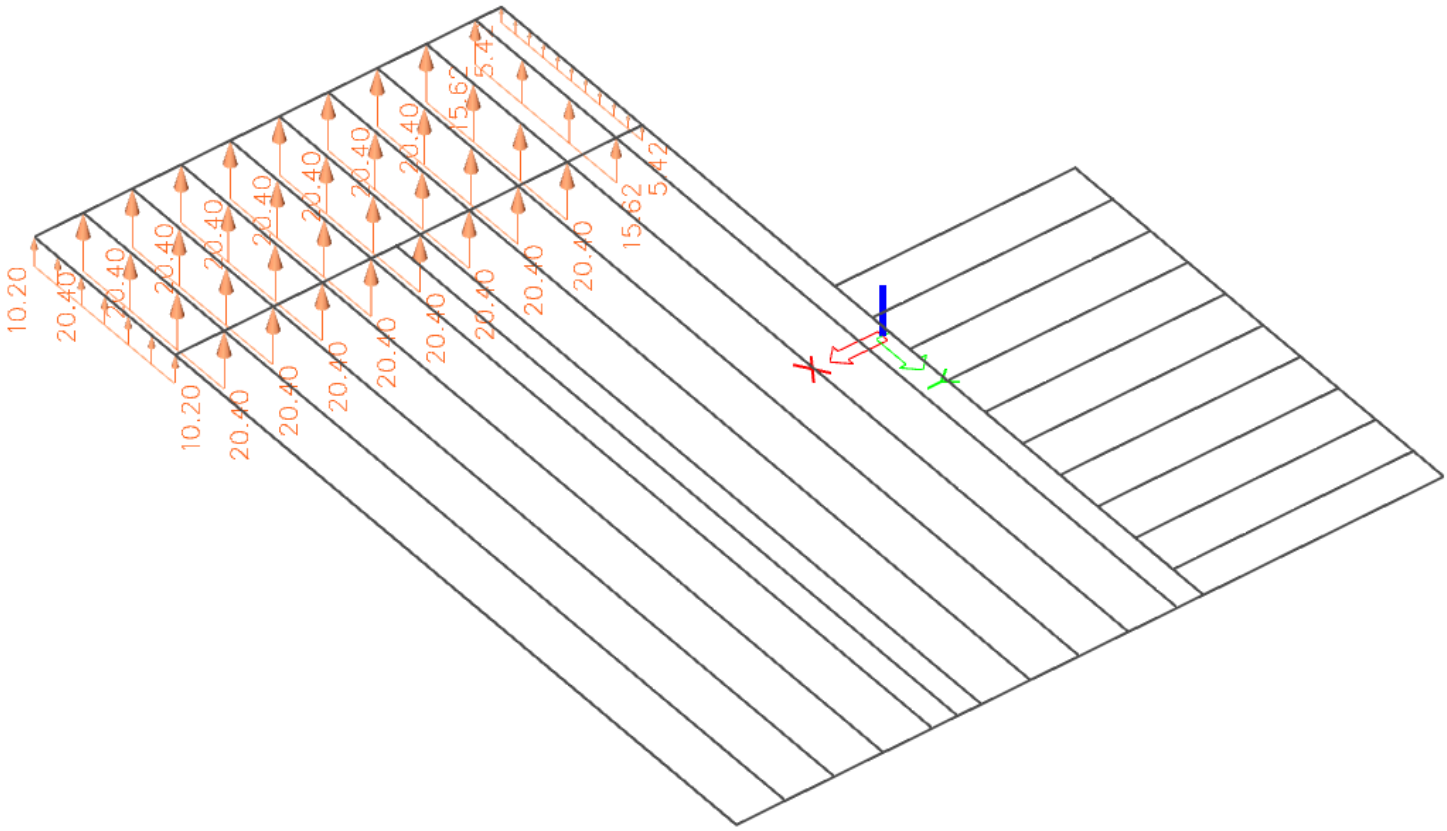
DL4 - Dead Load Floor.
8.6 psf * 16 in = 11.47 plf.



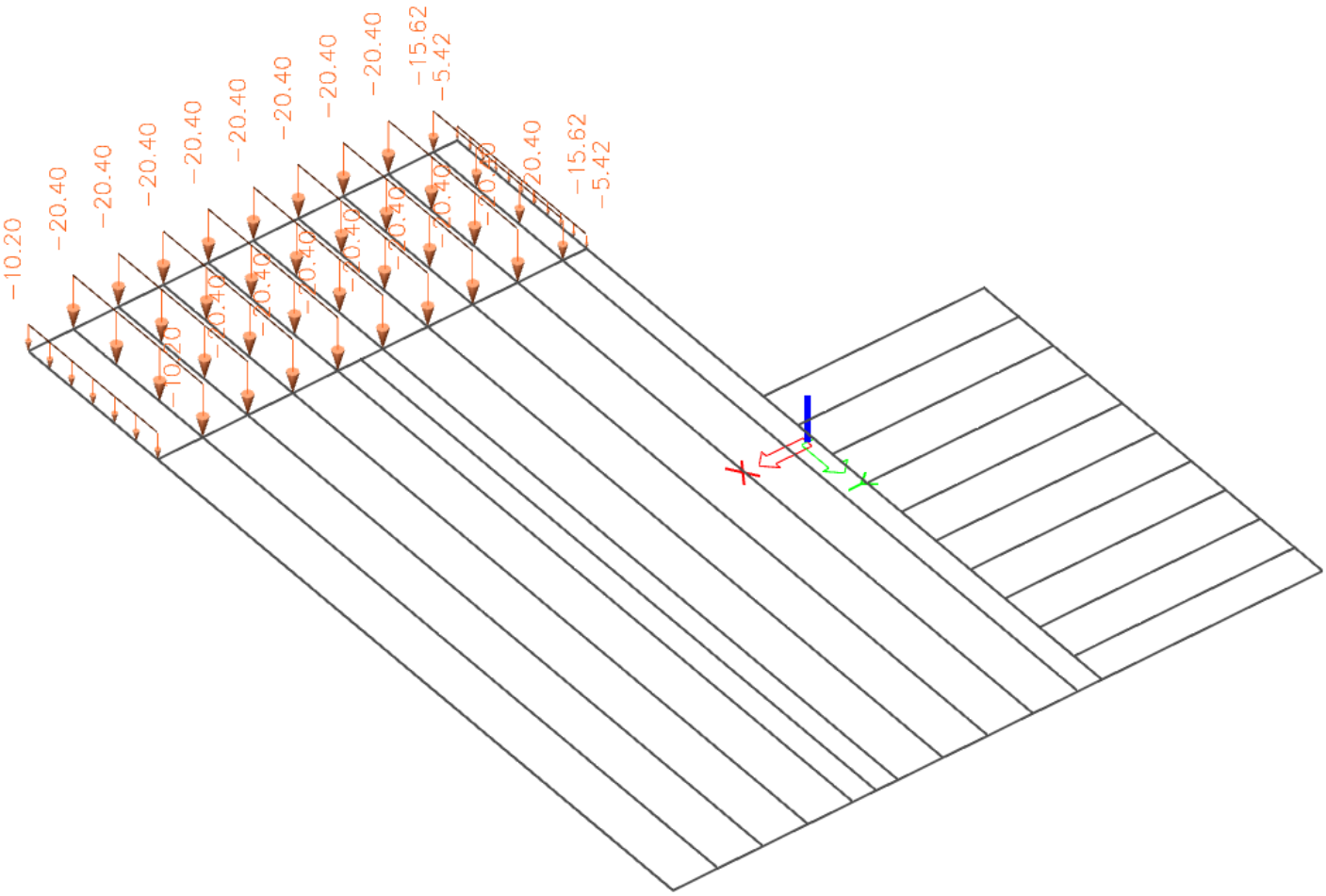
L - Floor Live Load, plf.
40 psf * 16 in = 53.33 plf.
50 psf * 16 in = 80 plf.



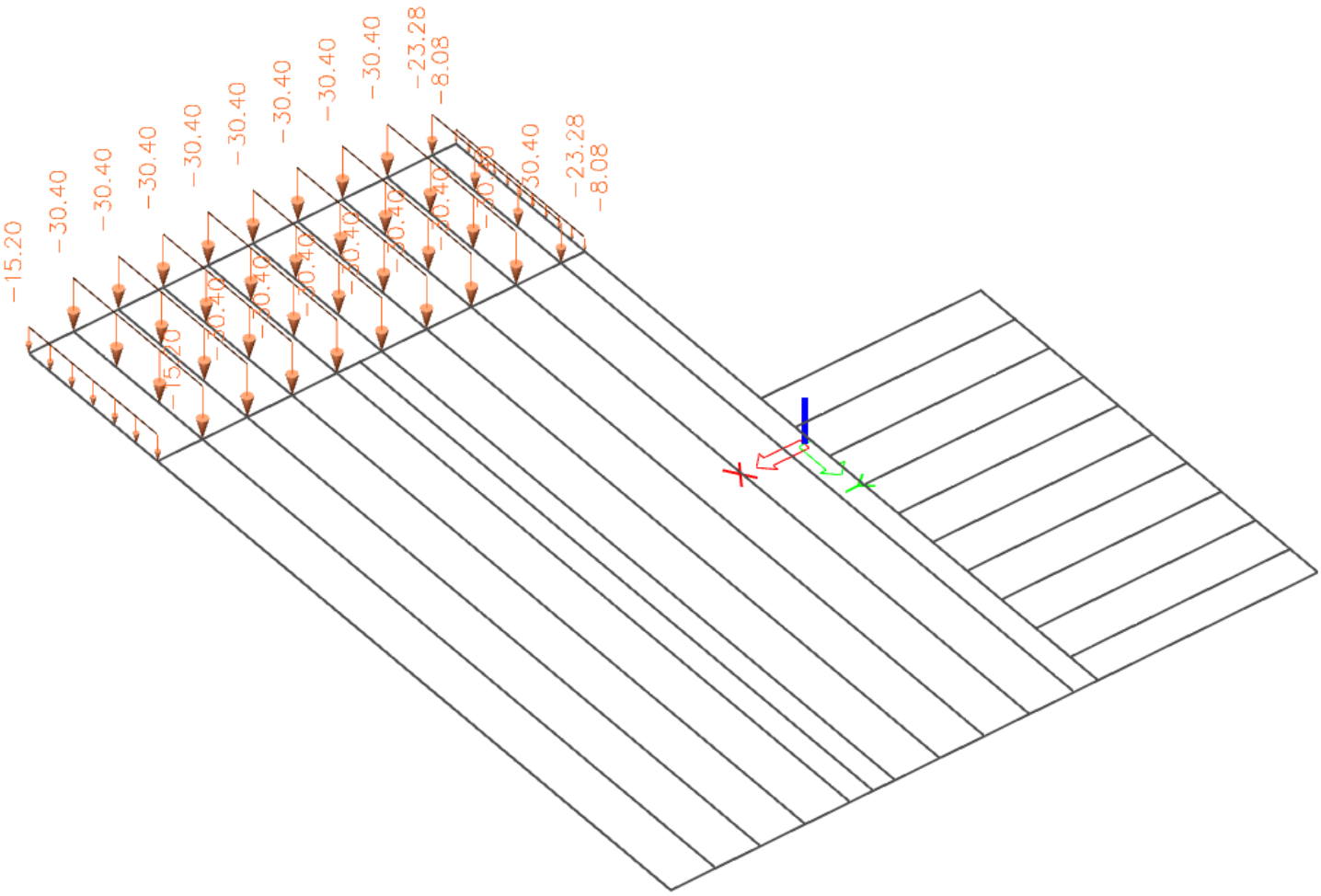
Wx+(0.18), Wx-(0.18) - Wind load, plf.



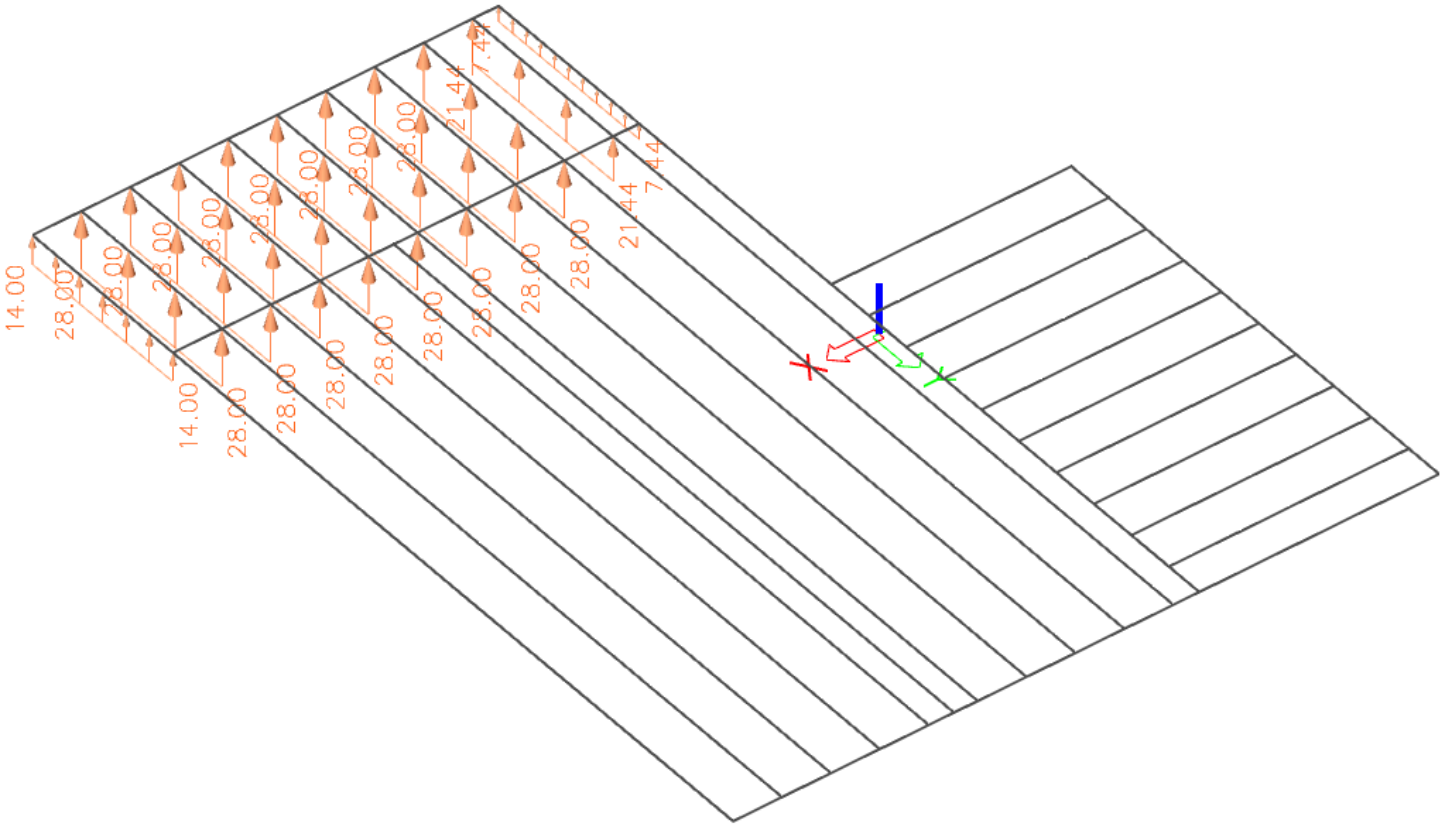
Wx+(-0.18), Wx-(-0.18) - Wind load, plf.



Wy+(0.18), Wy-(0.18) - Wind load, plf.



Wy+(-0.18), Wy-(-0.18) - Wind load, plf.



Load combinations see page 47

Floor member maximum force

Shear force diagramVz,
LRFD-Ult (auto) 5 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1.6*L), lbf.

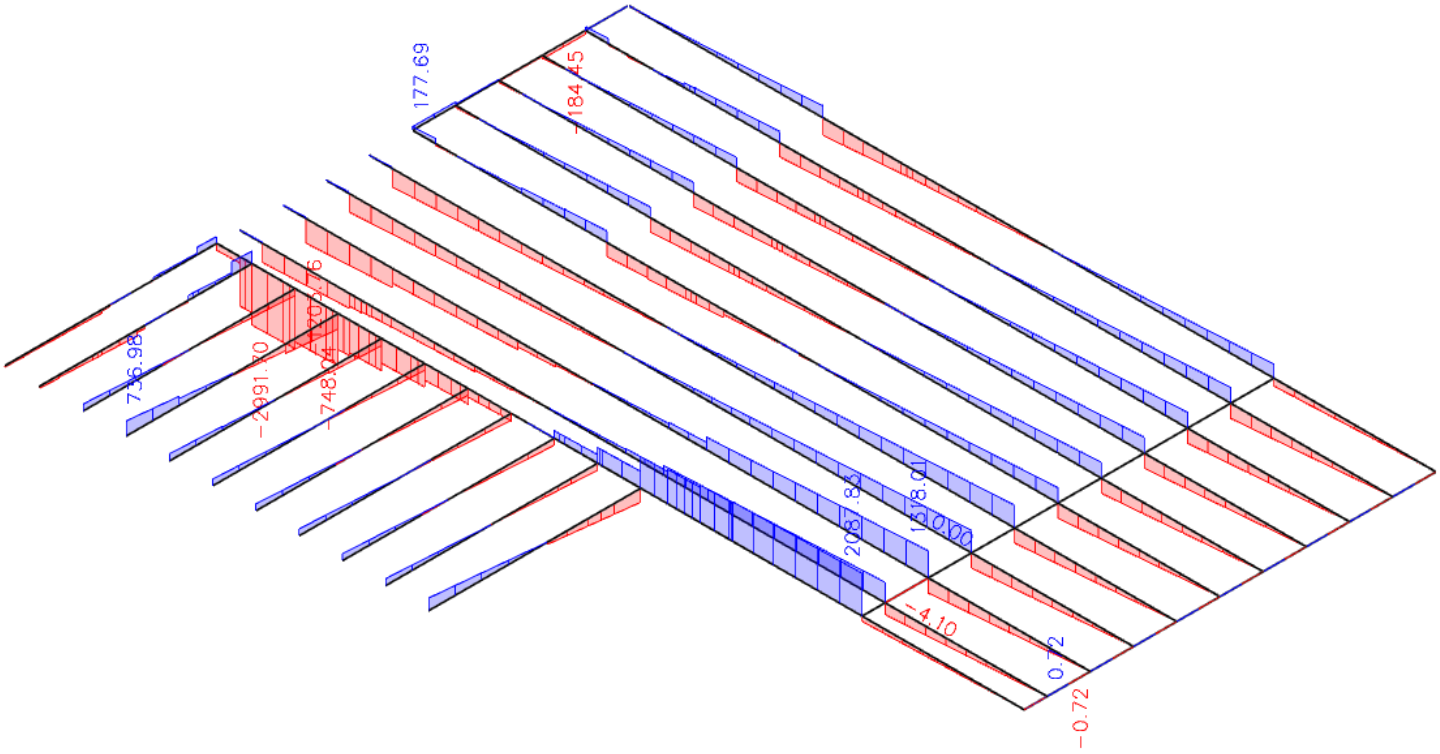
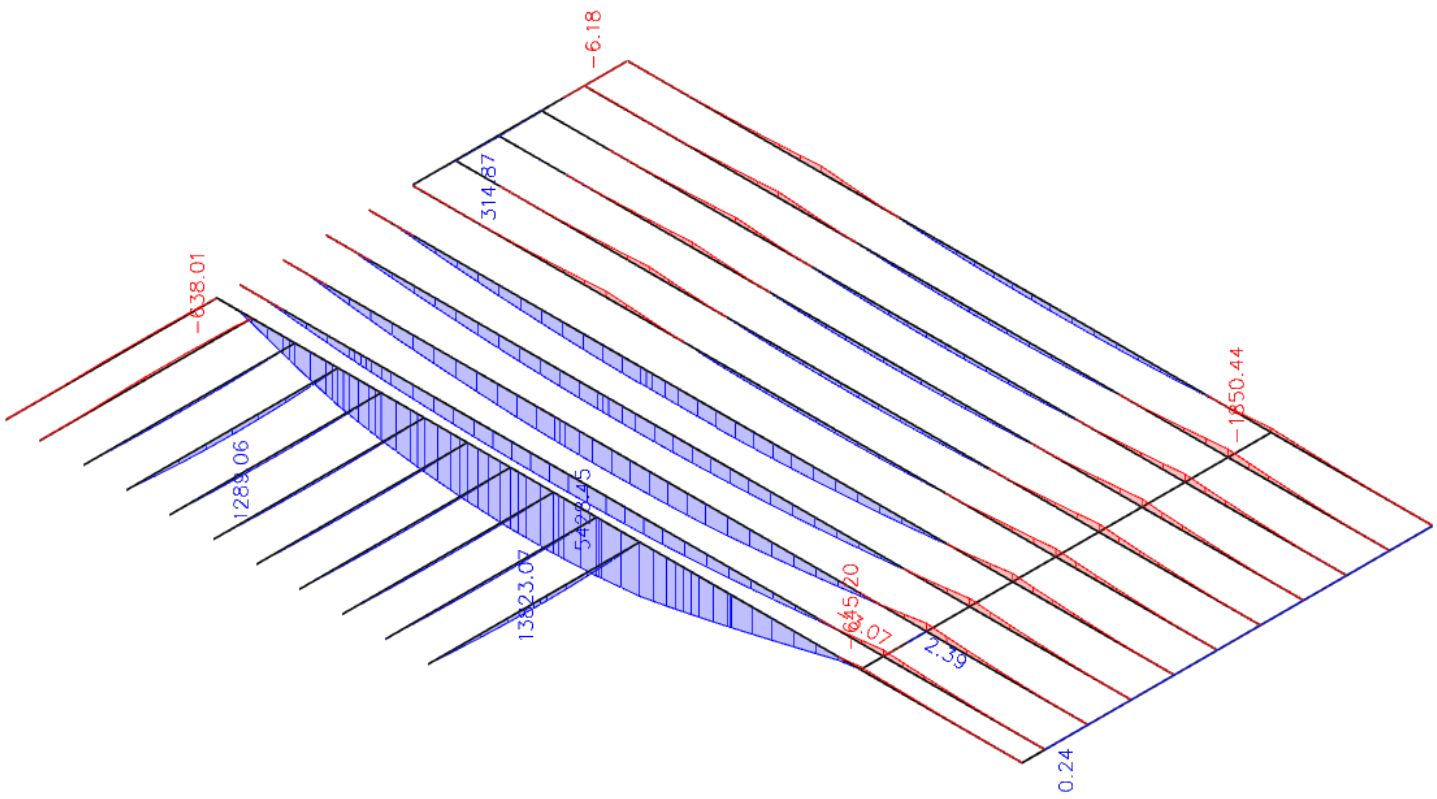
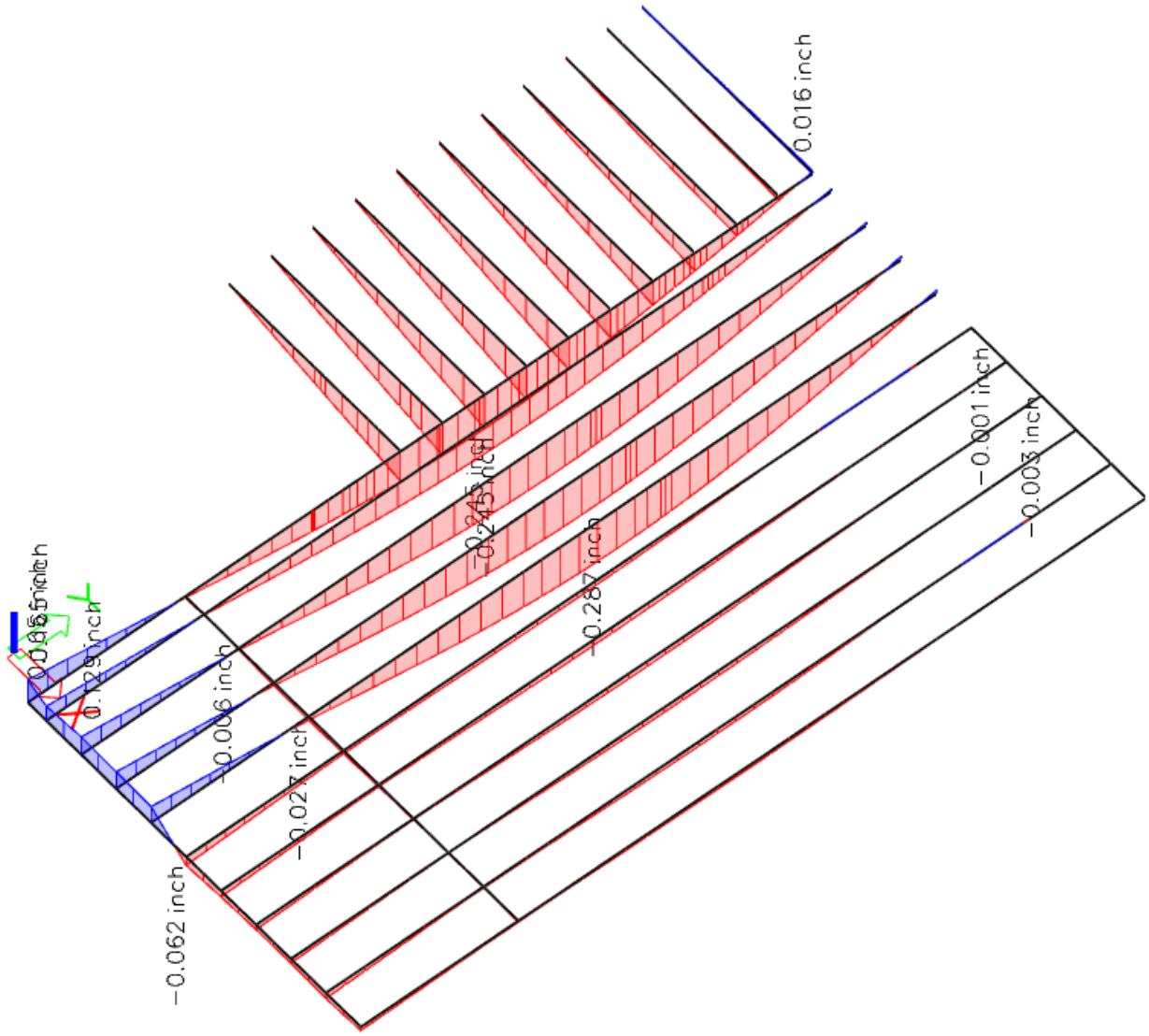


Diagram of bending moments My,
LRFD-Ult (auto) 5 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1.6*L), lbf*ft.



Floor member displacement
Load case L, inch:



The maximum deflection for floor joist 1200S162-54 is 0.287”. According to TABLE R301.7 the code IRC 2018, maximum member deflection should not exceed - L/360.
L=19’-11”=239”, 239”/360=0.663”. 0.287"< 0.663”. Deflection is OK!

STEEL MEMBER B1565 CHECK

AISI S100-16 LRFD Check

Member B1565	2CFCo (S(SSMA)1200S162-97; 0.000)	A913 grade 50	LRFD-Ult (auto)	0.49

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 9.50 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-3.78	lbf
Vux	3.70	lbf
Vuy	27.07	lbf
Mut	-0.04	lbfft
Mux	13814.15	lbfft
Muy	-2.31	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.246	-41.724 -43.694	-	-	-	-	-	- -	- -	-	- -	-
3	1.117	-45.323 -45.323	-	-	-	-	-	- -	- -	-	- -	-
5	11.492	48.371 -43.694	0.90	21.596	44.345	1.044	0.756	- 8.685	2.225 2.338	-	- -	-
7	1.117	50.000 50.000	1.00	3.011	654.869	0.276	1.000	1.117 -	0.558 0.558	30.83	0.000 0.000	0.246
9	0.246	48.371 46.402	0.96	0.445	1996.587	0.156	1.000	0.246 -	- -	-	- -	-
10	0.246	48.371 46.402	0.96	0.445	1996.587	0.156	1.000	0.246 -	- -	-	- -	-
12	1.117	50.000 50.000	1.00	3.011	654.869	0.276	1.000	1.117 -	0.558 0.558	30.83	0.000 0.000	0.246
14	11.492	48.371 -43.694	0.90	21.596	44.345	1.044	0.756	- 8.685	2.225 2.338	-	- -	-
16	1.117	-45.323 -45.323	-	-	-	-	-	- -	- -	-	- -	-
18	0.246	-41.724 -43.694	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	8.153	$inch^3$
Mnxo	33972.41	lbfft
Resistance factor	0.90	
Unity check	0.45	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	10.000 in	ft
Sigma,ey	816.092	ksi
Kt	1.00	
Lt	10.000 in	ft
Sigma,t	1703.736	ksi
Cb	1.00	
Sfx	8.982	$inch^3$
Fcre	1724.872	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

Distortional Buckling Strength

According to article F4 and formula F4.1-2.

Table of values		
Sfy	8.982	$inch^3$
My	37426.88	lbfft
L	10.000 in	ft
Beta	1.00	
k,phi,fe	1725.63	lbf
k,phi,we	1905.08	lbf
k,phi	0.00	lbf
k,phi,fg	0.022	$inch^2$
k,phi,wg	0.037	$inch^2$
Fd	61.671	ksi
Sf	8.982	$inch^3$
Mcrd	46163.00	lbfft
Lambda,d	0.90	
Mn	31410.20	lbfft
Resistance factor	0.90	
Unity check	0.49	-

Data		
Lm	10.000 in	ft
Lcr	11.360 in	ft
h0	11.492	inch
Ixf	0.003	$inch^4$
Iyf	0.032	$inch^4$
Ixyf	-0.005	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.001	$inch^4$
x0f	0.494	inch
hxf	-1.004	inch
Af	0.171	$inch^2$
y0f	0.046	inch
Ksi,web	2.00	

Number of compressed flanges: 2

Critical flange contains Initial shape parts: 8, 7, 9

...::Flexural Strength about Y-axis::...
Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	12.925	ksi
Kt	1.00	
Lt	10.000 in	ft
Sigma,t	1703.736	ksi
Cb	1.00	
Sfy	0.553	$inch^3$
Fcre	3527.996	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

Note: Web Crippling has been ignored due to user input

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	non-sway	
Unbraced Length L	19 1/4	7/8	ft
Effective Length factor K	2.67	0.99	
Effective Length	51 3/8	7/8	ft
Slenderness	148.83	18.58	
Flexural Buckling stress Fcre	12.925	829.201	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	12.925	ksi
Sigma,ey	829.201	ksi
Kt	1.00	
Lt	7/8	ft
Sigma,t	1703.736	ksi
Sigma,TF	12.925	ksi
Torsional (-Flexural) buckling stress Fcre	12.925	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.246	11.335 11.335	1.00	0.430	1929.899	0.077	1.000	0.246 -	- -	-	- -	-
3	1.117	11.335 11.335	1.00	4.000	869.951	0.114	1.000	1.117 -	0.558 0.558	64.75	- 0.000	0.246
5	11.492	11.335 11.335	1.00	4.000	8.214	1.175	0.692	7.950 -	- -	-	- -	-
7	1.117	11.335 11.335	1.00	4.000	869.951	0.114	1.000	1.117 -	0.558 0.558	64.75	- 0.000	0.246
9	0.246	11.335 11.335	1.00	0.430	1929.899	0.077	1.000	0.246 -	- -	-	- -	-
10	0.246	11.335 11.335	1.00	0.430	1929.899	0.077	1.000	0.246 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
10	0.246	11.335 11.335	1.00	0.430	1929.899	0.077	1.000	0.246 -	- -	-	- -	-
12	1.117	11.335 11.335	1.00	4.000	869.951	0.114	1.000	1.117 -	0.558 0.558	64.75	- 0.000	0.246
14	11.492	11.335 11.335	1.00	4.000	8.214	1.175	0.692	7.950 -	- -	-	- -	-
16	1.117	11.335 11.335	1.00	4.000	869.951	0.114	1.000	1.117 -	0.558 0.558	64.75	- 0.000	0.246
18	0.246	11.335 11.335	1.00	0.430	1929.899	0.077	1.000	0.246 -	- -	-	- -	-

Table of values		
Fe	12.925	ksi
lambda, c	1.97	
Fn	11.335	ksi
Ae	2.431	inch^2
Pn	27553.40	lbf
Resistance factor	0.85	
Unity check	0.00	-

Combined Compressive Axial Load and Bending

According to article H1.2 and formulas (C5.2.1-3)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.246	0.001 0.001	1.00	0.430	1929.899	0.001	1.000	0.246 -	- -	-	- -	-
3	1.117	0.001 0.001	1.00	4.000	869.951	0.001	1.000	1.117 -	0.558 0.558	6290.88	- 0.000	0.246
5	11.492	0.001 0.001	1.00	4.000	8.214	0.012	1.000	11.492 -	- -	-	- -	-
7	1.117	0.001 0.001	1.00	4.000	869.951	0.001	1.000	1.117 -	0.558 0.558	6290.88	- 0.000	0.246
9	0.246	0.001 0.001	1.00	0.430	1929.899	0.001	1.000	0.246 -	- -	-	- -	-
10	0.246	0.001 0.001	1.00	0.430	1929.899	0.001	1.000	0.246 -	- -	-	- -	-
12	1.117	0.001 0.001	1.00	4.000	869.951	0.001	1.000	1.117 -	0.558 0.558	6290.88	- 0.000	0.246
14	11.492	0.001 0.001	1.00	4.000	8.214	0.012	1.000	11.492 -	- -	-	- -	-
16	1.117	0.001 0.001	1.00	4.000	869.951	0.001	1.000	1.117 -	0.558 0.558	6290.88	- 0.000	0.246
18	0.246	0.001 0.001	1.00	0.430	1929.899	0.001	1.000	0.246 -	- -	-	- -	-

Table of values		
Mnx	31410.20	lbfft
Mny	1295.56	lbfft
Pn	27553.40	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.00+0.49+0.00 = 0.49 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B CHECK

AISI S100-16 LRFD Check

Member B1867	T1200T200-54	A913 grade 50	LRFD-Ult (auto)	0.10

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

Warning: Part 3 exceeds dimensional limit h/t <= 200! (art. B1.2(a))

The critical check is on position 4.00 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	7.06	lbf
Vux	-0.77	lbf
Vuy	-179.87	lbf
Mut	-0.00	lbfft
Mux	237.00	lbfft
Muy	0.09	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	1.858	-27.903 -27.903	-	-	-	-	-	- -	- -	-	- -	-
3	11.717	49.262 -27.165	0.55	14.571	8.914	2.351	0.386	- 4.518	1.272 1.640	-	- -	-
5	1.858	50.000 50.000	1.00	0.430	10.456	2.187	0.411	0.764 -	- -	-	- -	-

Table of values		
Sxe	1.317	$inch^3$
Mnxo	5485.92	lbfft
Resistance factor	0.90	
Unity check	0.05	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	1 ft 4.000 in	ft
Sigma,ey	293.867	ksi
Kt	1.00	
Lt	1 ft 4.000 in	ft
Sigma,t	436.911	ksi
Cb	1.69	
Sfx	2.642	$inch^3$
Fcre	882.686	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...
Shear Strength
According to article G2.1 and formula (G2.1.1)
Shear force Vy

Element ID	Aw [$inch^2$]	Vn [lbf]
1	0.000	0.00
3	0.663	2166.97
5	0.000	0.00

Table of values		
Vn,y	2166.97	lbf
Resistance factor	0.95	
Unity check	0.09	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnxo	5485.92	lbfft
Vny	2166.97	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.00+0.01) = 0.10

Note: Web Crippling has been ignored due to user input

Combined Tensile Axial Load and Bending
According to article H1.1 and formulas (H1.1-1), (H1.1-2)

Table of values		
Sftx	2.642	$inch^3$
Mnxt	11009.17	lbfft
Mnx	5485.92	lbfft
Tn	44753.01	lbf
Resistance factor tension	0.95	
Resistance factor bending x	0.90	

Unity check = 0.02+0.00+0.00 = 0.02 - (H1.1-1)
Unity check = 0.05+0.00-0.00 = 0.05 - (H1.1-2)

The member satisfies the check !

STEEL MEMBER B2779 CHECK

AISI S100-16 LRFD Check

Member B2779	S1200S162-54	A913 grade 50	LRFD-Ult (auto)	0.91

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

Warning: Part 5 exceeds dimensional limit h/t <= 200! (art. B1.2(a))

The critical check is on position 10.33 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-7.15	lbf
Vux	0.02	lbf
Vuy	46.83	lbf
Mut	-0.00	lbfft
Mux	5428.44	lbfft
Muy	-0.19	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	-33.249 -35.851	-	-	-	-	-	- -	- -	-	- -	-
3	1.342	-36.673 -36.673	-	-	-	-	-	- -	- -	-	- -	-
5	11.717	49.179 -35.851	0.73	17.795	10.887	2.125	0.422	- 4.942	1.325 1.533	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	49.179 46.577	0.95	0.449	293.467	0.409	1.000	0.222 -	- -	-	- -	-

Table of values		
Sxe	1.903	inch^3
Mnxo	7928.51	lbfft
Resistance factor	0.90	
Unity check	0.76	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	1 ft 2.000 in	ft
Sigma,ey	344.418	ksi
Kt	1.00	
Lt	1 ft 2.000 in	ft
Sigma,t	560.674	ksi
Cb	1.01	
Sfx	2.639	$inch^3$
Fcre	645.960	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
Distortional Buckling Strength
According to article F4 and formula F4.1-2.

Table of values		
Sfy	2.639	$inch^3$
My	10996.39	lbfft
L	1 ft 2.000 in	ft
Beta	1.02	
k,phi,fe	261.29	lbf
k,phi,we	221.37	lbf
k,phi	0.00	lbf
k,phi,fg	0.007	$inch^2$
k,phi,wg	0.012	$inch^2$

Table of values		
Fd	25.461	ksi
Sf	2.639	$inch^3$
Mcrd	5599.63	lbfft
Lambda,d	1.40	
Mn	6615.10	lbfft
Resistance factor	0.90	
Unity check	0.91	-

Data		
Lm	1 ft 2.000 in	ft
Lcr	1 ft 3.985 in	ft
h0	12.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch
Ksi,web	2.00	

Number of compressed flanges: 1
Critical flange contains Initial shape parts: 8, 7, 9

...::Shear Strength::...

Shear Strength

According to article G2.1 and formula (G2.1.1)

Shear force Vy

Element ID	Aw [inch^2]	Vn [lbf]
3	0.000	0.00
5	0.663	2166.97
7	0.000	0.00

Table of values		
Vn,y	2166.97	lbf
Resistance factor	0.95	
Unity check	0.02	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	7928.51	lbfft
Vny	2166.97	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.58+0.00) = 0.76

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	19 1/4	1 1/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	19 1/4	1 1/4	ft
Slenderness	54.91	28.83	
Flexural Buckling stress Fcre	94.947	344.418	ksi

Torsional (-Flexural) Buckling Strength

According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	94.947	ksi
Sigma,ey	344.418	ksi
Kt	1.00	
Lt	1 1/4	ft
Sigma,t	560.674	ksi
Sigma,TF	94.381	ksi
Torsional (-Flexural) buckling stress Fcre	94.381	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	40.056 40.056	1.00	0.430	281.003	0.378	1.000	0.359 -	- -	-	- -	-
3	1.342	40.056 40.056	1.00	3.387	157.959	0.504	1.000	1.342 -	0.671 0.671	34.45	0.000 0.000	0.359
5	11.717	40.056 40.056	1.00	4.000	2.447	4.046	0.234	2.739 -	- -	-	- -	-
7	1.342	40.056 40.056	1.00	3.387	157.959	0.504	1.000	1.342 -	0.671 0.671	34.45	0.000 0.000	0.359
9	0.359	40.056 40.056	1.00	0.430	281.003	0.378	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	94.381	ksi
lambda, c	0.73	
Fn	40.056	ksi
Ae	0.388	inch^2
Pn	15530.07	lbf
Resistance factor	0.85	
Unity check	0.00	-

Combined Compressive Axial Load and Bending

According to article H1.2 and formulas (C5.2.1-3)

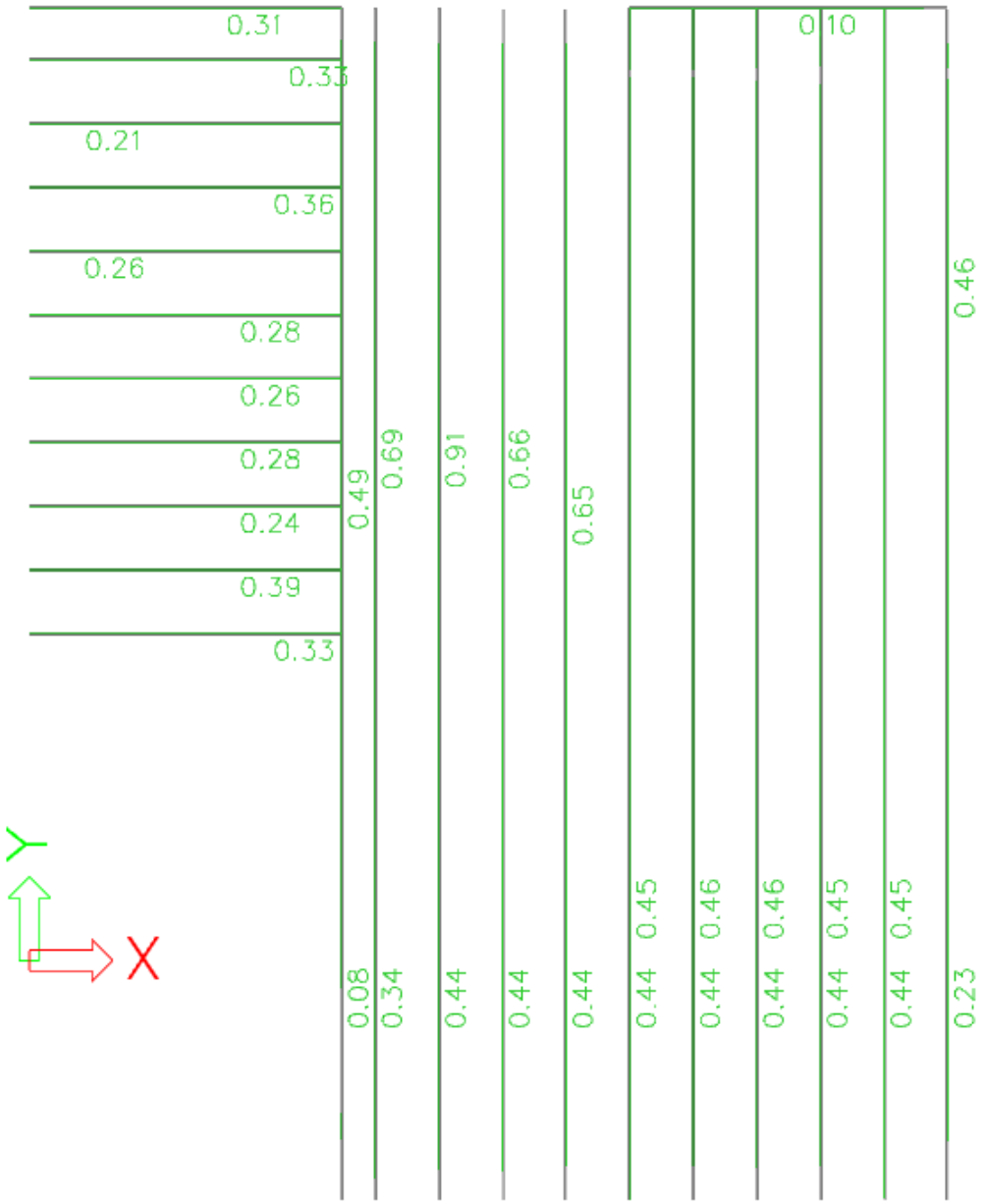
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	0.008 0.008	1.00	0.430	281.003	0.005	1.000	0.359 -	- -	-	- -	-
3	1.342	0.008 0.008	1.00	4.000	186.542	0.007	1.000	1.342 -	0.671 0.671	2444.65	- 0.000	0.359
5	11.717	0.008 0.008	1.00	4.000	2.447	0.057	1.000	11.717 -	- -	-	- -	-
7	1.342	0.008 0.008	1.00	4.000	186.542	0.007	1.000	1.342 -	0.671 0.671	2444.65	- 0.000	0.359
9	0.359	0.008 0.008	1.00	0.430	281.003	0.005	1.000	0.359 -	- -	-	- -	-

Table of values		
Mnx	6615.10	lbfft
Pn	9102.26	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	

Unity check = 0.00+0.91+0.00 = 0.91 - (C5.2.1-3)

The member satisfies the check !

Unity check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0.65
0.10
0.46
Y
X
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0.45
0.23

2-ND JOISTS SCREWS CONNECTIONS DESIGN

Maximum vertical suuport reactions from combination
LRFD-Ult (auto) 5 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*Lr + 1.6*L) - 737 lbs.

For connected joist to wall stud use Simson Strong-Tie X-Metal screw #14 with reference shear 375 lb for steel 20 ga (33mil)

X Metal Screw — Cold-Formed Steel Connection Loads

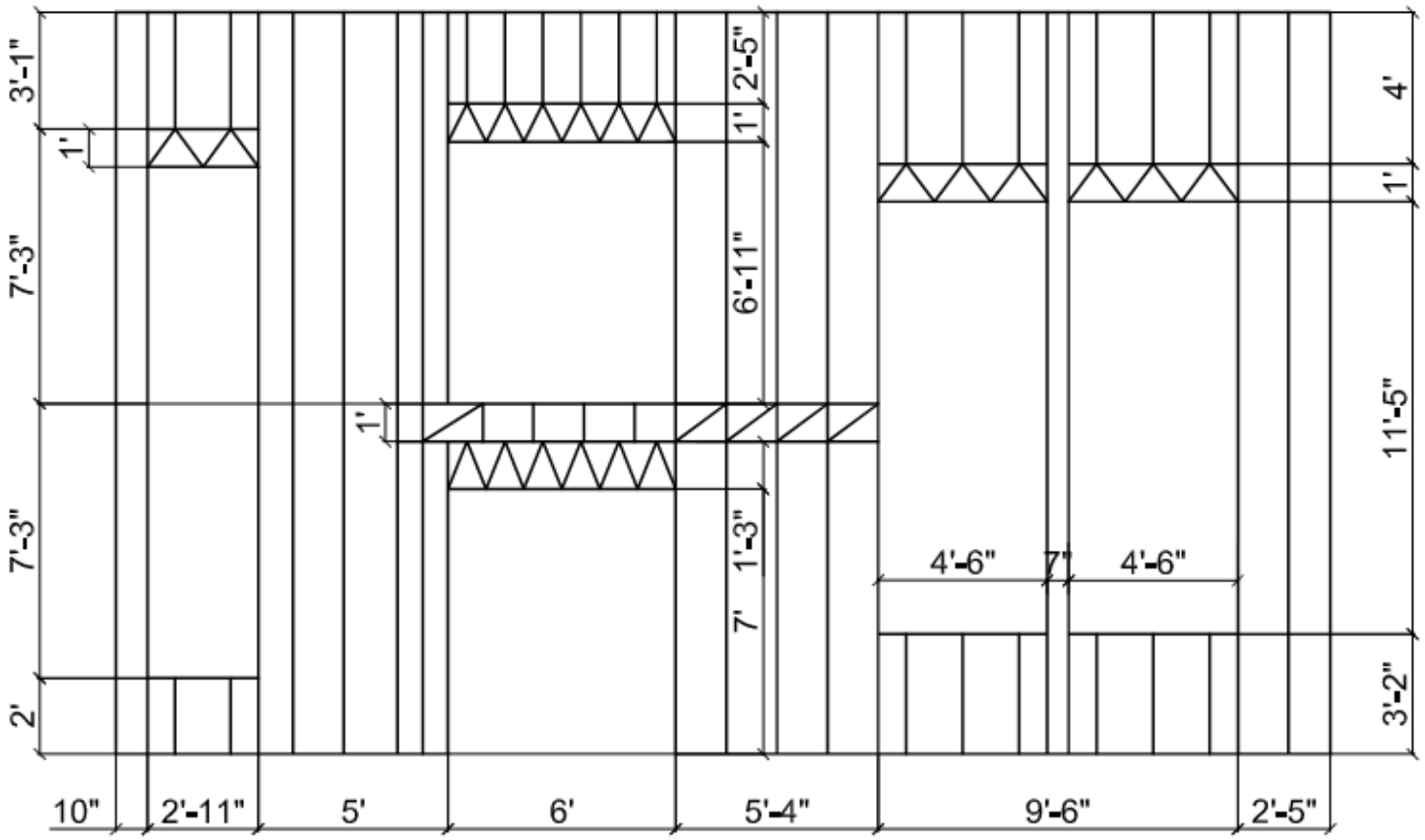
Size (in.)	Model No.	Nominal Dia. (in.)[7]	Load Description	Reference Shear (lb.)						Reference Pull-Over (lb.)						Reference Pull-Out (lb.)					
				Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]					
				27	33	43	54	68	97	27	33	43	54	68	97	27	33	43	54	68	97
				(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)
#12-14 x 1	XQ1S1214 X1S1214	0.216	ASD	176	235	385	595	840	840	295	375	525	785	1,045	1,210	74	96	147	215	325	500
			LRFD	280	375	610	950	1,265	1,265	470	600	835	1,255	1,670	1,875	117	154	235	340	520	795
			Nominal strength	400	535	870	1,350	2,135	2,135	720	920	1,285	1,925	2,565	2,965	180	235	360	520	800	1,220

Screws required 737 lb /375 lb = 1.96 pcs.

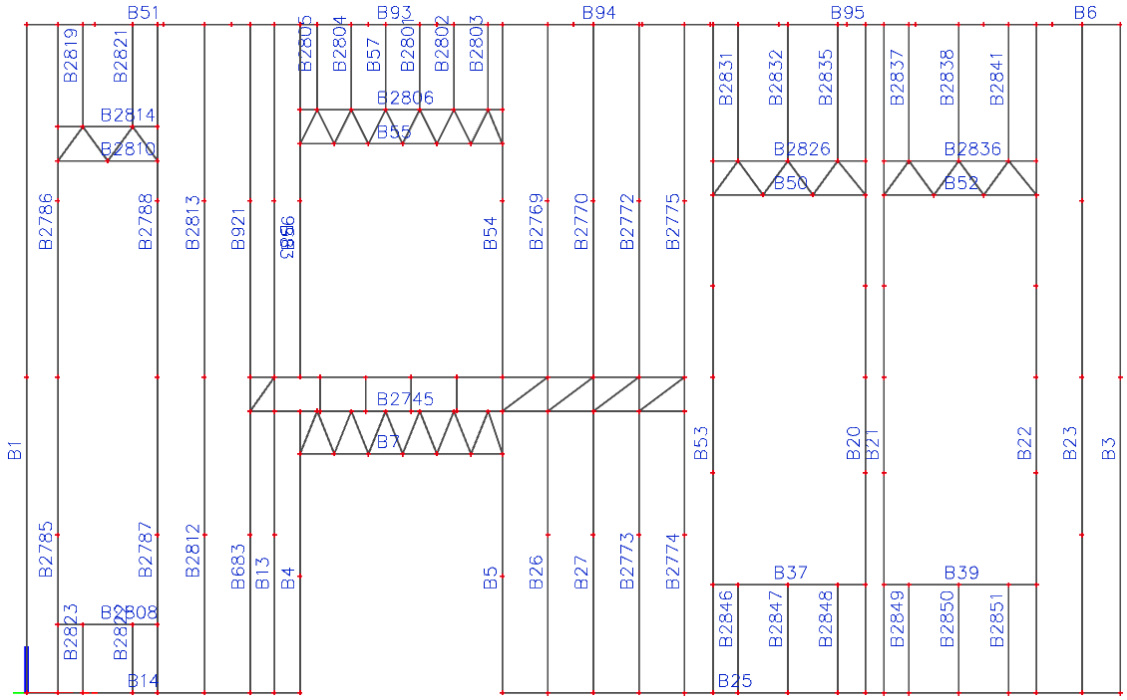
Structurally accepts 4 pcs.

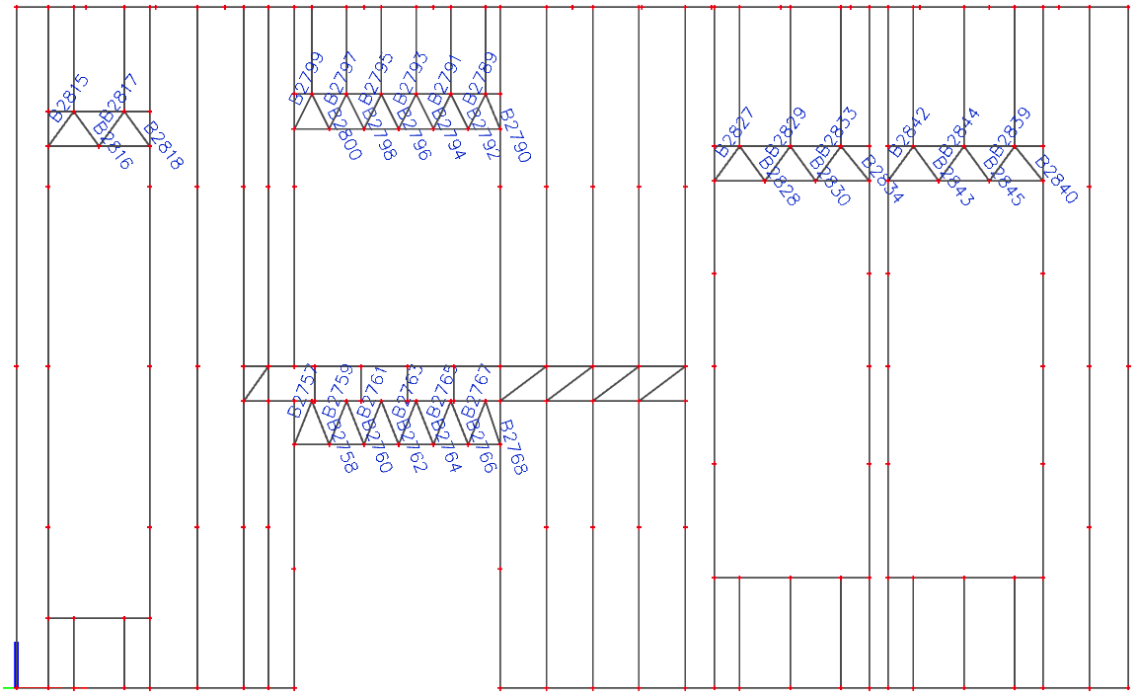
2.4.3 FRONT WALL CHECK

Wall stud spacing - 16”;
First story wall stud height - 9’-3”;
Second story wall stud height - 10’-4”.

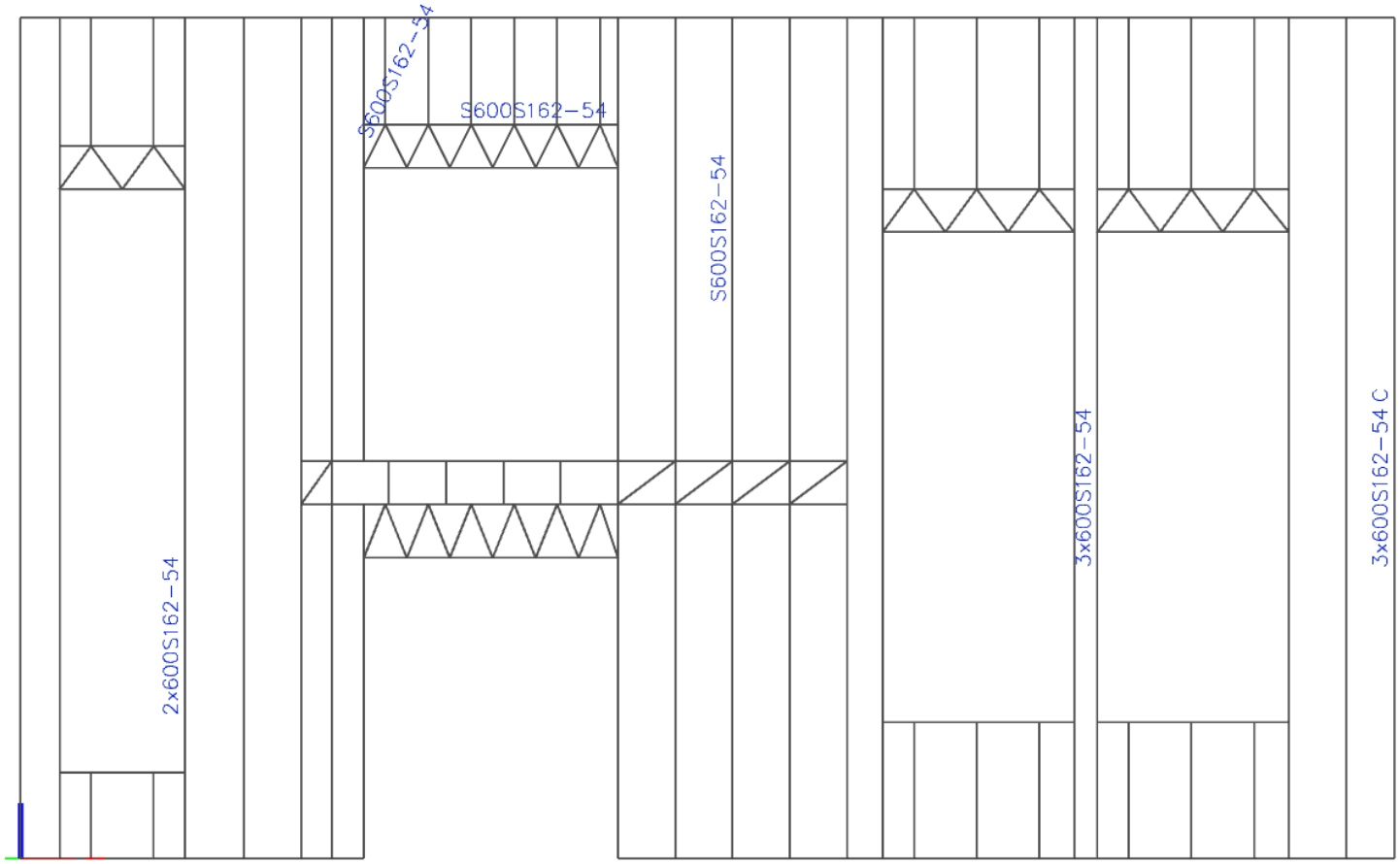


Numbers of members

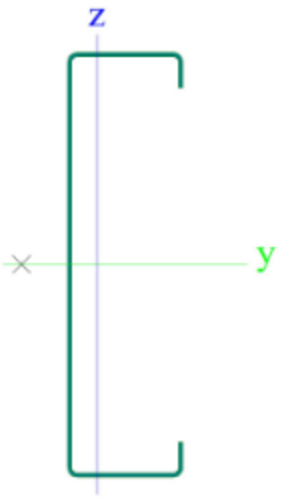




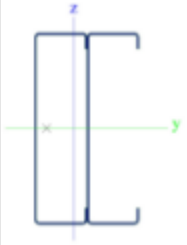
Cross-sections of members



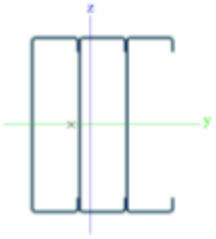
Cross-sections properties of 600S162-54

CS14		
Type	S600S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0,559	
A_y [inch^2], A_z [inch^2]	0,186	0,346
A_L [inch^2/inch], A_D [inch^2/inch]	1,98e+01	1,98e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0,413	3,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	2,886	0,181
i_y [inch], i_z [inch]	2,272	0,568
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0,953	0,149
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	1,140	0,217
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	5,70e+01	5,70e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,08e+01	1,08e+01
d_y [inch], d_z [inch]	-1,056	0,000
I_t [inch^4], I_w [inch^6]	0,001	1,337
β_y [inch], β_z [inch]	0,000	6,508
Picture		

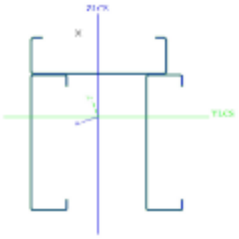
Cross-sections properties of 2x600S162-54

CS11		
Type	2x600S162-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,112	
A_y [inch^2], A_z [inch^2]	0,346	0,676
A_L [inch^2/inch], A_D [inch^2/inch]	2,33e+01	3,81e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	2,948	3,968
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	5,718	1,095
i_y [inch], i_z [inch]	2,267	0,992
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	1,906	0,541
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	2,280	0,904
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,14e+02	1,14e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	4,52e+01	4,52e+01
d_y [inch], d_z [inch]	-0,860	0,000
I_t [inch^4], I_w [inch^6]	1,381	3,828
β_y [inch], β_z [inch]	0,000	3,169
Picture		

Cross-sections properties of 3x600S162-54

CS17		
Type	3x600S162-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,668	
A_y [inch^2], A_z [inch^2]	0,507	1,014
A_L [inch^2/inch], A_D [inch^2/inch]	2,69e+01	5,64e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	3,760	3,968
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	8,577	3,479
i_y [inch], i_z [inch]	2,267	1,444
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	2,859	1,226
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	3,420	2,024
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,71e+02	1,71e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,01e+02	1,01e+02
d_y [inch], d_z [inch]	-0,676	0,000
I_t [inch^4], I_w [inch^6]	4,201	4,273
β_y [inch], β_z [inch]	0,000	2,036
Picture		

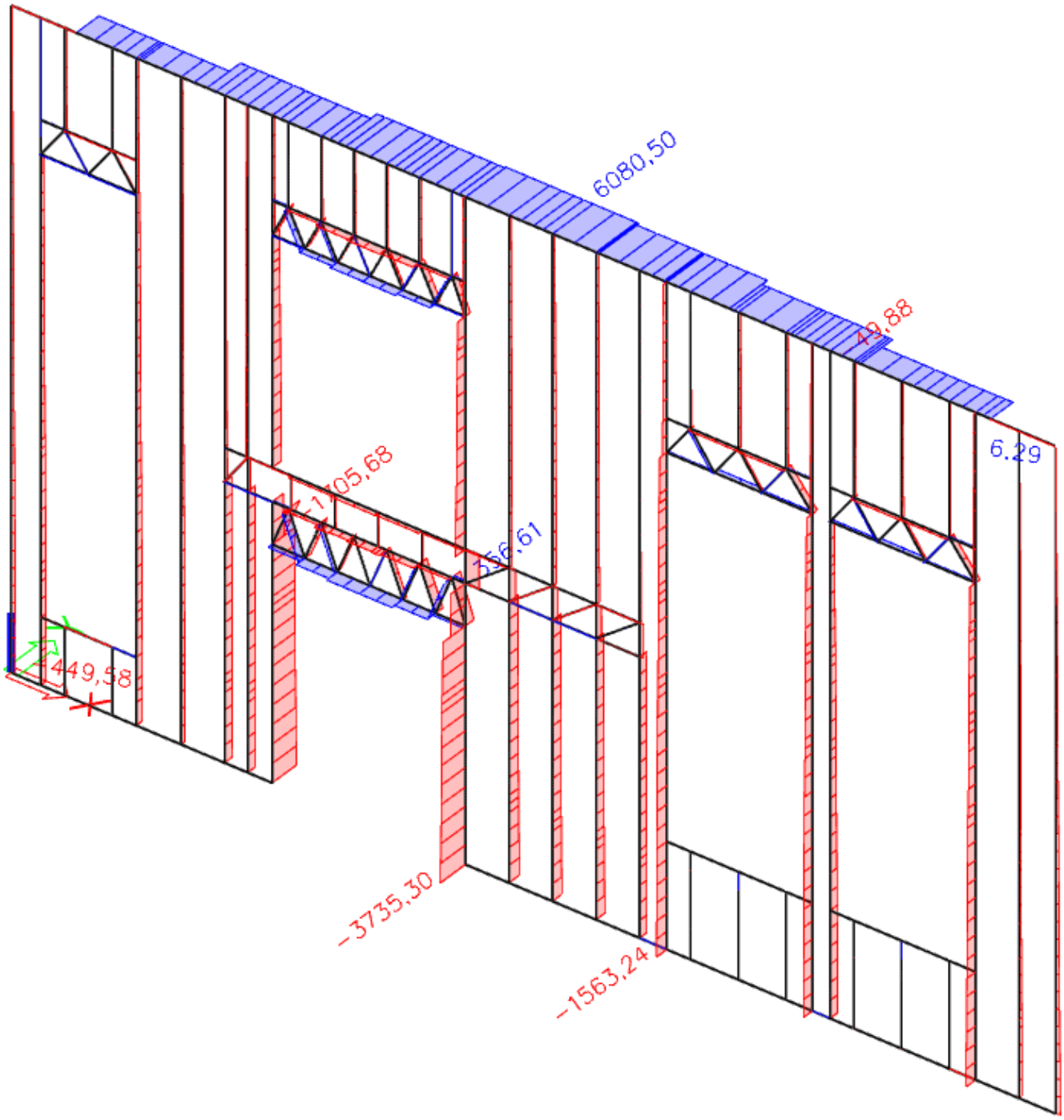
Cross-sections properties of 3x600S162-54 C

CS21		
Type	3x600S162-54 C	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,668	
A_y [inch^2], A_z [inch^2]	1,079	0,840
A_L [inch^2/inch], A_D [inch^2/inch]	5,53e+01	5,53e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	4,407	4,242
$I_{Y,LCS}$ [inch^4], $I_{Z,LCS}$ [inch^4]	10,218	10,372
$I_{YZ,LCS}$ [inch^4]	0,064	
α [deg]	110,01	
I_y [inch^4], I_z [inch^4]	10,396	10,195
i_y [inch], i_z [inch]	2,496	2,472
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	2,497	1,989
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	3,722	3,380
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,86e+02	1,86e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,69e+02	1,69e+02
d_y [inch], d_z [inch]	3,751	-0,439
I_t [inch^4], I_w [inch^6]	0,002	77,572
β_y [inch], β_z [inch]	1,226	-8,898
Picture		

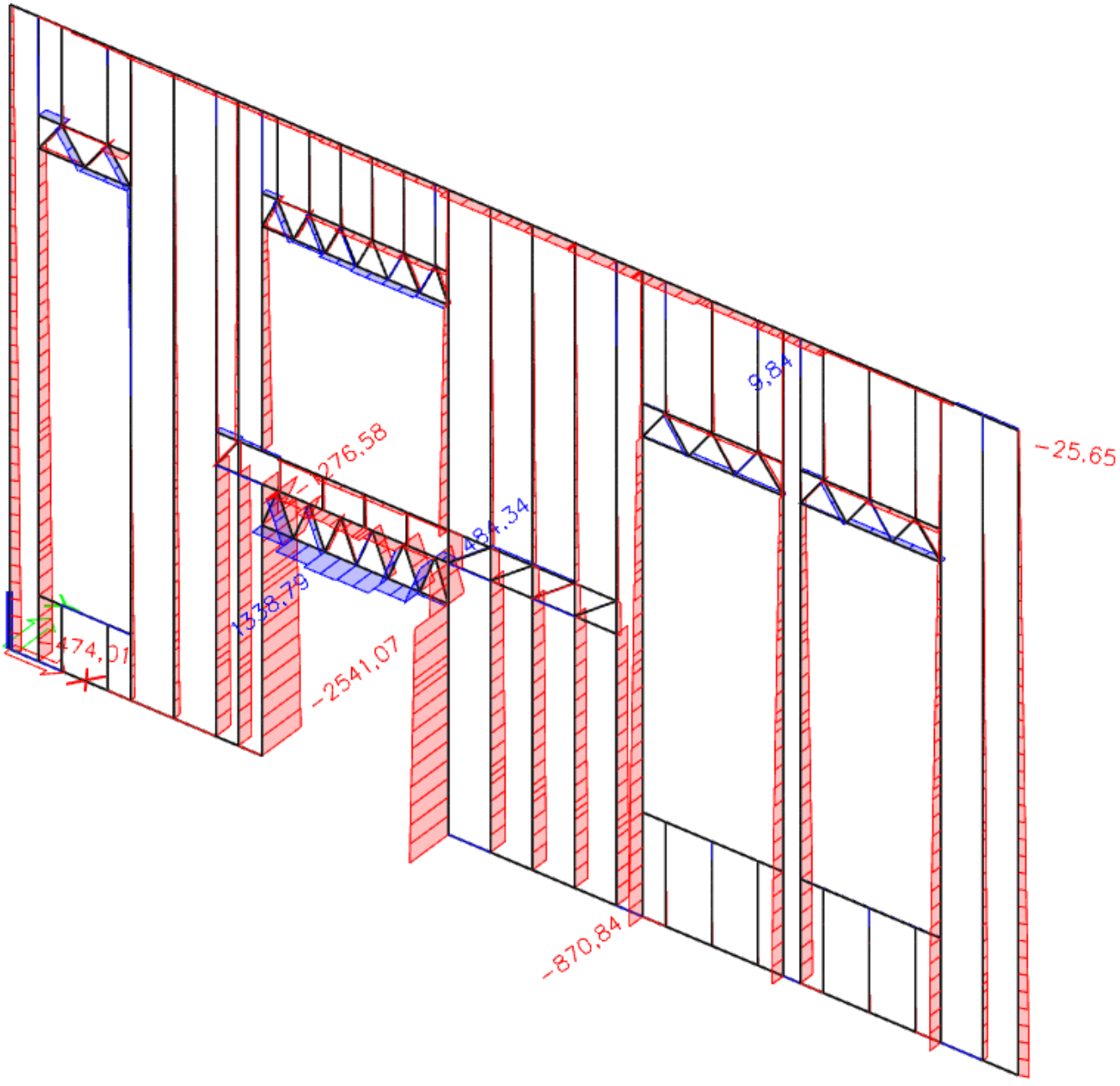
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Axial force diagram N,
LRFD-Ult (auto) 8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1.6*Lr), lbf.



Axial force diagram N,
LRFD-Ult (auto) 55 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 0.5*Lr + Wy+(-0.18)), lbf.



Shear force diagram Vz,
LRFD-Ult (auto) 55 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 0.5*Lr + Wy+(-0.18)), lbf.

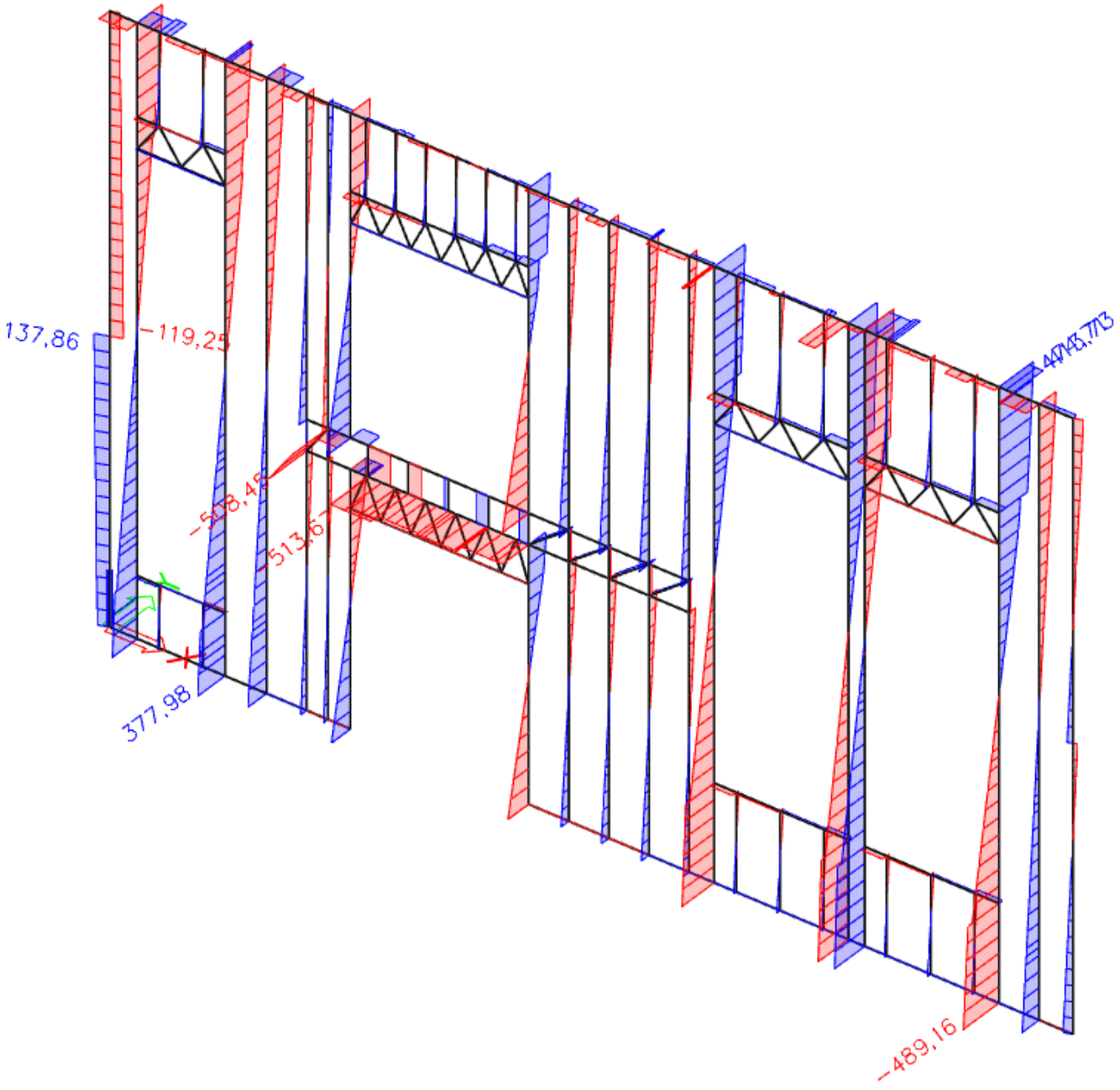
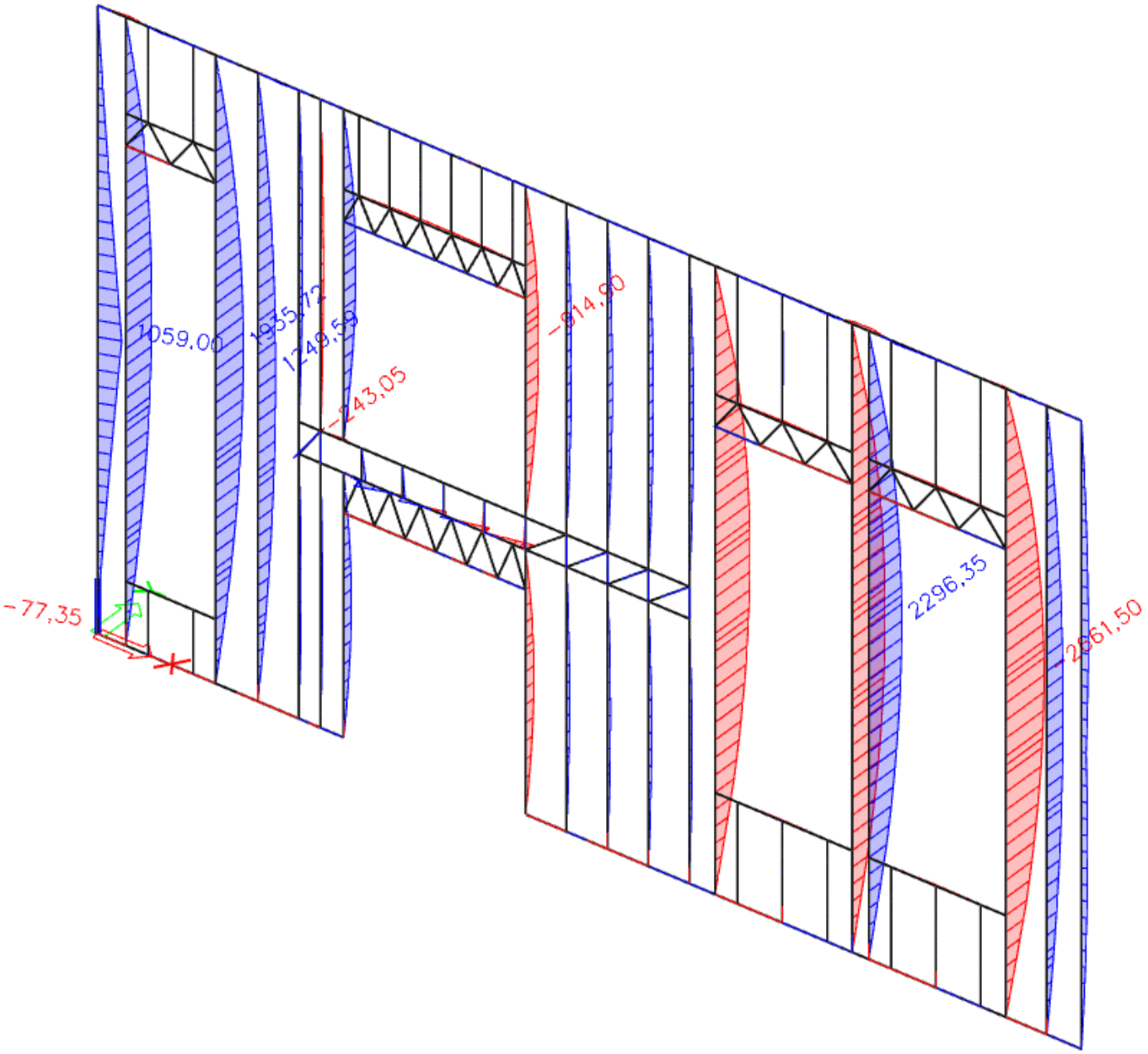
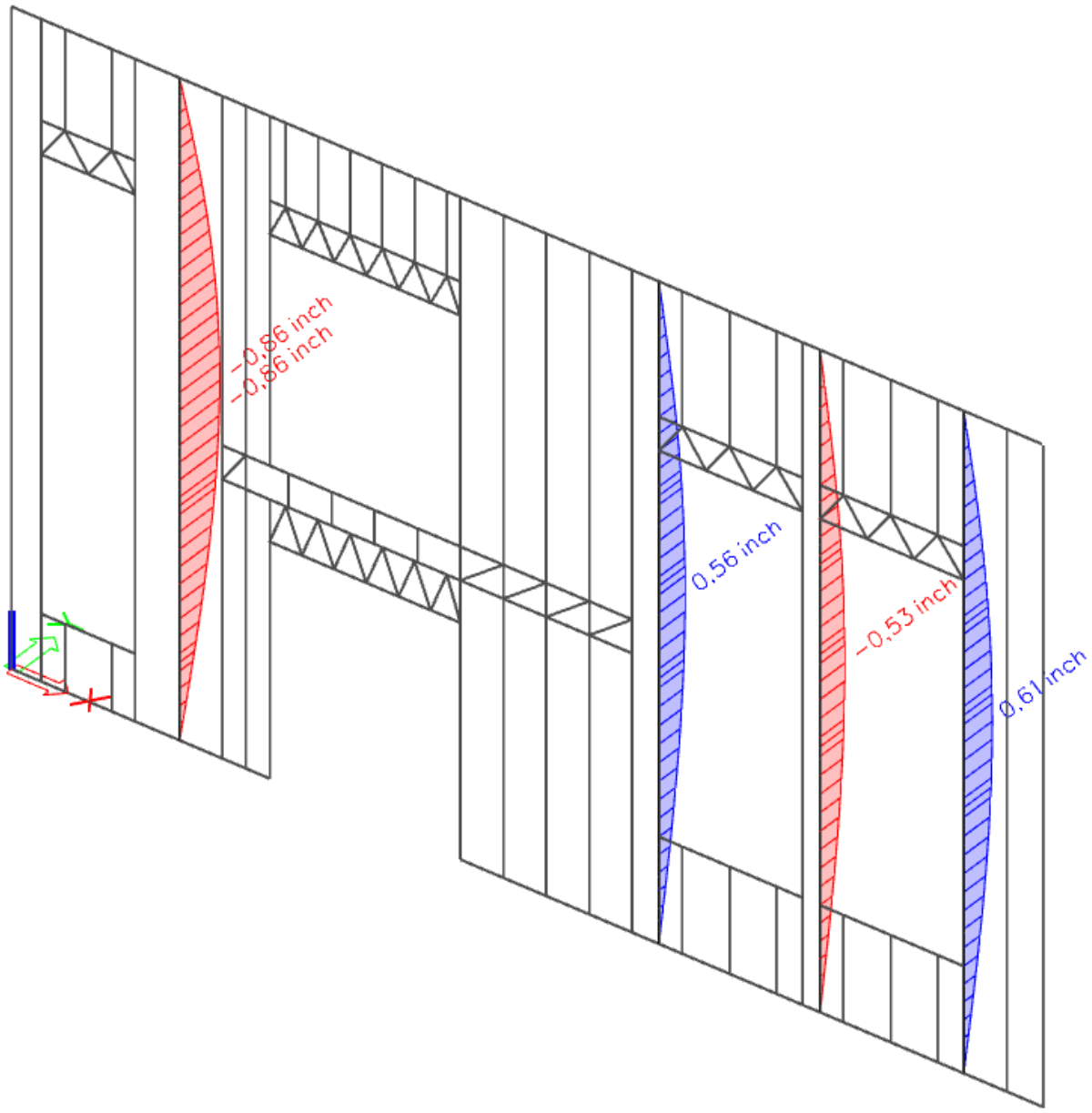


Diagram of bending moments My,
LRFD-Ult (auto) 55 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 0.5*Lr + Wy+(-0.18)), lbf*ft.



Wall stud deflection by Wind Load Wy+ (0.18), inch.



The maximum horizontal deflection of 0.85”. According to TABLE R301.7 the code IRC 2018, maximum wall member deflection should not exceed - H/240. H = stud height.
19’-7”=235”, 235”/240=0.979”. 0.85"< 0.979”. Deflection is OK!

STEEL MEMBER B22 CHECK

AISI S100-16 LRFD Check

Member B22	3x600S162-54	A913 grade 50	LRFD-Ult (auto)	0.31

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 9.25 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-424.21	lbf
Vux	-37.25	lbf
Vuy	-29.09	lbf
Mut	2.60	lbfft
Mux	-2653.91	lbfft
Muy	-160.11	lbfft

Combined Bending and Torsional Loading
According to article H4 and formula (H4-1)

Table of values		
Critical fibre	68	
Sigma Mx	11.139	ksi
Sigma My	1.978	ksi
f bending	13.117	ksi
Tau t	0.000	ksi
f torsion	0.000	ksi
Composed Stress	13.117	ksi
R	1.00	-

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-

...::Flexural Strength about X-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
5	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-
6	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-
11	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
12	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
13	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
14	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
15	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-

Table of values		
Sxe	2.859	inch^3
Mnxo	11912.61	lbfft
Resistance factor	0.90	
Unity check	0.25	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	2' 9.628"	ft
Sigma,ey	527.865	ksi
Kt	1.00	
Lt	2' 9.628"	ft
Sigma,t	3740.644	ksi
Cb	1.03	
Sfx	2.859	inch^3
Fcre	2346.897	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	14.313 14.313	1.00	4.000	6039.613	0.049	1.000	0.472 -	- -	-	- -	-
2	1.568	46.188 14.878	0.32	5.979	204.134	0.476	1.000	- 1.568	0.586 0.983	-	- -	-
3	5.943	46.188 46.188	1.00	4.000	9.511	2.204	0.408	2.428 -	- -	-	- -	-
4	1.568	46.188 14.878	0.32	5.979	204.134	0.476	1.000	- 1.568	0.586 0.983	-	- -	-
5	0.472	14.313 14.313	1.00	4.000	6039.613	0.049	1.000	0.472 -	- -	-	- -	-
6	0.472	-18.126 -18.126	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	13.748 -17.561	1.28	32.175	1098.573	0.112	1.000	- 1.568	0.367 0.784	-	- -	-
8	5.000	13.748 13.748	1.00	4.000	13.438	1.011	0.774	3.868 -	- -	-	- -	-
9	1.568	13.748 -17.561	1.28	32.175	1098.573	0.112	1.000	- 1.568	0.367 0.784	-	- -	-
10	0.472	-18.126 -18.126	-	-	-	-	-	- -	- -	-	- -	-
11	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
12	1.568	-18.691 -50.000	-	-	-	-	-	- -	- -	-	- -	-
13	5.000	-18.691 -18.691	-	-	-	-	-	- -	- -	-	- -	-
14	1.568	-18.691 -50.000	-	-	-	-	-	- -	- -	-	- -	-
15	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	1.034	inch^3
Mnyo	4308.89	lbfft
Resistance factor	0.90	
Unity check	0.04	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	26.651	ksi
Kt	1.00	
Lt	2' 9.628"	ft
Sigma,t	3740.644	ksi
Cb	1.00	
Sfy	1.707	inch^3
Fcre	855.597	ksi

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 50.000	1.00	4.000	9.511	2.293	0.394	2.343 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
11	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
12	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
13	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
14	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
15	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	1.194	inch^2
Pno	59677.33	lbf
Resistance factor	0.85	
Unity check	0.01	-

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	19 5/8	2 7/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	19 5/8	2 7/8	ft
Slenderness	103.64	23.29	
Flexural Buckling stress Fcre	26.651	527.865	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	26.651	ksi
Sigma,ey	527.865	ksi
Kt	1.00	
Lt	2 7/8	ft
Sigma,t	3740.644	ksi
Sigma,TF	26.640	ksi
Torsional (-Flexural) buckling stress Fcre	26.640	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	22.793 22.793	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
2	1.568	22.793 22.793	1.00	4.000	136.574	0.409	1.000	1.568 -	- -	-	- -	-
3	5.943	22.793 22.793	1.00	4.000	9.511	1.548	0.554	3.294 -	- -	-	- -	-
4	1.568	22.793 22.793	1.00	4.000	136.574	0.409	1.000	1.568 -	- -	-	- -	-
5	0.472	22.793 22.793	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
6	0.472	22.793 22.793	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
7	1.568	22.793 22.793	1.00	4.000	136.574	0.409	1.000	1.568 -	- -	-	- -	-
8	5.000	22.793 22.793	1.00	4.000	13.438	1.302	0.638	3.191 -	- -	-	- -	-
9	1.568	22.793 22.793	1.00	4.000	136.574	0.409	1.000	1.568 -	- -	-	- -	-
10	0.472	22.793 22.793	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
11	0.472	22.793 22.793	1.00	0.430	162.315	0.375	1.000	0.472 -	- -	-	- -	-
12	1.568	22.793 22.793	1.00	4.000	136.574	0.409	1.000	1.568 -	- -	-	- -	-
13	5.000	22.793 22.793	1.00	4.000	13.438	1.302	0.638	3.191 -	- -	-	- -	-
14	1.568	22.793 22.793	1.00	4.000	136.574	0.409	1.000	1.568 -	- -	-	- -	-
15	0.472	22.793 22.793	1.00	0.430	162.315	0.375	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	26.640	ksi
lambda, c	1.37	
Fn	22.793	ksi
Ae	1.349	inch^2
Pn	30737.33	lbf
Resistance factor	0.85	
Unity check	0.02	-

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (C5.2.1-3)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.472	0.254 0.254	1.00	4.000	6039.613	0.006	1.000	0.472 -	- -	-	- -	-
2	1.568	0.254 0.254	1.00	4.000	136.574	0.043	1.000	1.568 -	- -	-	- -	-
3	5.943	0.254 0.254	1.00	4.000	9.511	0.164	1.000	5.943 -	- -	-	- -	-
4	1.568	0.254 0.254	1.00	4.000	136.574	0.043	1.000	1.568 -	- -	-	- -	-
5	0.472	0.254 0.254	1.00	4.000	6039.613	0.006	1.000	0.472 -	- -	-	- -	-
6	0.472	0.254 0.254	1.00	4.000	6039.613	0.006	1.000	0.472 -	- -	-	- -	-
7	1.568	0.254 0.254	1.00	4.000	136.574	0.043	1.000	1.568 -	- -	-	- -	-
8	5.000	0.254 0.254	1.00	4.000	13.438	0.138	1.000	5.000 -	- -	-	- -	-
9	1.568	0.254 0.254	1.00	4.000	136.574	0.043	1.000	1.568 -	- -	-	- -	-
10	0.472	0.254 0.254	1.00	4.000	6039.613	0.006	1.000	0.472 -	- -	-	- -	-
11	0.472	0.254 0.254	1.00	0.430	162.315	0.040	1.000	0.472 -	- -	-	- -	-
12	1.568	0.254 0.254	1.00	4.000	136.574	0.043	1.000	1.568 -	- -	-	- -	-
13	5.000	0.254 0.254	1.00	4.000	13.438	0.138	1.000	5.000 -	- -	-	- -	-
14	1.568	0.254 0.254	1.00	4.000	136.574	0.043	1.000	1.568 -	- -	-	- -	-
15	0.472	0.254 0.254	1.00	0.430	162.315	0.040	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	11912.61	lbfft
Mny	4308.89	lbfft
Pn	30737.33	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.02+0.25+0.04 = 0.31 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B2787 CHECK

AISI S100-16 LRFD Check

Member B2787	2x600S162-54	A913 grade 50	LRFD-Ult (auto)	0.66

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 9.25 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-1239.33	lbf
Vux	-119.51	lbf
Vuy	-16.54	lbf
Mut	6.81	lbfft
Mux	-1705.49	lbfft
Muy	-580.47	lbfft

Combined Bending and Torsional Loading
According to article H4 and formula (H4-1)

Table of values		
Critical fibre	36	
Sigma Mx	10.231	ksi
Sigma My	17.817	ksi
f bending	28.048	ksi
Tau t	-0.003	ksi
f torsion	-0.003	ksi
Composed Stress	28.048	ksi
R	1.00	-

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
6	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-

Table of values		
Sxe	1.906	inch^3
Mnxo	7941.74	lbfft
Resistance factor	0.90	
Unity check	0.24	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	4' 7.500"	ft
Sigma,ey	91.530	ksi
Kt	1.00	
Lt	4' 7.500"	ft
Sigma,t	2064.279	ksi
Cb	1.08	
Sfx	1.906	inch^3
Fcre	715.479	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	-4.257 -4.257	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	41.486 -3.446	0.08	8.707	297.289	0.374	1.000	- 1.568	0.509 1.060	-	- -	-
3	5.943	41.486 41.486	1.00	4.000	9.511	2.089	0.428	2.546 -	- -	-	- -	-
4	1.568	41.486 -3.446	0.08	8.707	297.289	0.374	1.000	- 1.568	0.509 1.060	-	- -	-
5	0.472	-4.257 -4.257	-	-	-	-	-	- -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
6	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	-5.068 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	5.000	-5.068 -5.068	-	-	-	-	-	- -	- -	-	- -	-
9	1.568	-5.068 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.447	inch^3
Mnyo	1862.75	lbfft
Resistance factor	0.90	
Unity check	0.35	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	26.651	ksi
Kt	1.00	
Lt	4' 7.500"	ft
Sigma,t	2064.279	ksi
Cb	1.00	
Sfy	0.893	inch^3
Fcre	765.066	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...

Shear Strength

According to article G2.1 and formula (G2.1.1)

Shear force Vx

Element ID	Aw [inch^2]	Vn [lbf]
1	0.000	0.00
2	0.089	2663.14
3	0.000	0.00
4	0.089	2663.14
5	0.000	0.00
6	0.000	0.00
7	0.089	2663.14
8	0.000	0.00
9	0.089	2663.14
10	0.000	0.00

Table of values		
Vn,x	10652.57	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnyo	1862.75	lbfft
Vnx	10652.57	lbf
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

...::Axial Compression Strength::...
Nominal Axial Strength
According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 50.000	1.00	4.000	9.511	2.293	0.394	2.343 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.779	inch^2
Pno	38928.07	lbf
Resistance factor	0.85	
Unity check	0.04	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	19 5/8	4 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	19 5/8	4 5/8	ft
Slenderness	103.64	55.93	
Flexural Buckling stress Fcre	26.651	91.530	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	26.651	ksi
Sigma,ey	91.530	ksi
Kt	1.00	
Lt	4 5/8	ft
Sigma,t	2064.279	ksi
Sigma,TF	26.614	ksi
Torsional (-Flexural) buckling stress Fcre	26.614	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	22.775 22.775	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
2	1.568	22.775 22.775	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
3	5.943	22.775 22.775	1.00	4.000	9.511	1.547	0.554	3.295 -	- -	-	- -	-
4	1.568	22.775	1.00	4.000	136.574	0.408	1.000	1.568	-	-	-	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
		22.775						-	-		-	
5	0.472	22.775 22.775	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
6	0.472	22.775 22.775	1.00	0.430	162.315	0.375	1.000	0.472 -	- -	-	- -	-
7	1.568	22.775 22.775	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
8	5.000	22.775 22.775	1.00	4.000	13.438	1.302	0.638	3.192 -	- -	-	- -	-
9	1.568	22.775 22.775	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
10	0.472	22.775 22.775	1.00	0.430	162.315	0.375	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	26.614	ksi
lambda, c	1.37	
Fn	22.775	ksi
Ae	0.883	inch²
Pn	20112.19	lbf
Resistance factor	0.85	
Unity check	0.07	-

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (C5.2.1-3)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	1.114 1.114	1.00	4.000	6039.613	0.014	1.000	0.472 -	- -	-	- -	-
2	1.568	1.114 1.114	1.00	4.000	136.574	0.090	1.000	1.568 -	- -	-	- -	-
3	5.943	1.114 1.114	1.00	4.000	9.511	0.342	1.000	5.943 -	- -	-	- -	-
4	1.568	1.114 1.114	1.00	4.000	136.574	0.090	1.000	1.568 -	- -	-	- -	-
5	0.472	1.114 1.114	1.00	4.000	6039.613	0.014	1.000	0.472 -	- -	-	- -	-
6	0.472	1.114 1.114	1.00	0.430	162.315	0.083	1.000	0.472 -	- -	-	- -	-
7	1.568	1.114 1.114	1.00	4.000	136.574	0.090	1.000	1.568 -	- -	-	- -	-
8	5.000	1.114 1.114	1.00	4.000	13.438	0.288	1.000	5.000 -	- -	-	- -	-
9	1.568	1.114 1.114	1.00	4.000	136.574	0.090	1.000	1.568 -	- -	-	- -	-
10	0.472	1.114 1.114	1.00	0.430	162.315	0.083	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	7941.74	lbfft
Mny	1862.75	lbfft
Pn	20112.19	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.07+0.24+0.35 = 0.66 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B2813 CHECK

AISI S100-16 LRFD Check

Member B2813	S600S162-54	A913 grade 50	LRFD-Ult (auto)	0.67

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 0.00 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-247.18	lbf
Vux	5.58	lbf
Vuy	-12.48	lbf
Mut	0.00	lbfft
Mux	-1100.15	lbfft
Muy	-57.71	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	48.118 42.159	0.88	0.475	310.586	0.394	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	5.717	48.118 -46.914	0.97	23.357	60.020	0.895	0.842	- 4.816	1.212 2.408	-	- -	-
7	1.342	-48.795 -48.795	-	-	-	-	-	- -	- -	-	- -	-
9	0.359	-40.954 -46.914	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.925	inch^3
Mnxo	3854.26	lbfft
Resistance factor	0.90	
Unity check	0.32	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	35.379 30.942	0.87	0.476	310.987	0.337	1.000	0.359 -	- -	-	- -	-
3	1.342	36.780 36.780	1.00	3.387	157.959	0.483	1.000	1.342 -	0.671 0.671	35.95	0.000 0.000	0.359
5	5.717	35.379 -35.379	1.00	24.000	61.673	0.757	0.937	- 5.356	1.339 2.678	-	- -	-
7	1.342	-36.780 -36.780	-	-	-	-	-	- -	- -	-	- -	-
9	0.359	-30.942 -35.379	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Lltb	5' 2.000"	ft
Sigma,ey	24.050	ksi
Kt	1.00	
Lt	5' 2.000"	ft
Sigma,t	28.819	ksi
Cb	1.05	
Sfx	0.962	inch^3
Fcre	41.097	ksi
Fc	36.780	ksi
Scx	0.953	inch^3
Mnx	2922.36	lbfft

Table of values		
Resistance factor	0.90	
Unity check	0.42	-

Distortional Buckling Strength

According to article F4 and formula F4.1-2.

Table of values		
Sfy	0.962	inch^3
My	4008.12	lbfft
L	1' 1.314"	ft
Beta	1.05	
k,phi,fe	311.83	lbf
k,phi,we	293.13	lbf
k,phi	0.00	lbf
k,phi,fg	0.008	inch^2
k,phi,wg	0.002	inch^2
Fd	64.864	ksi
Sf	0.962	inch^3
Mcrd	5199.65	lbfft
Lambda,d	0.88	
Mn	3421.25	lbfft
Resistance factor	0.90	
Unity check	0.36	-

Data		
Lm	5' 2.000"	ft
Lcr	1' 1.314"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	-0.052	inch
Ksi,web	2.00	

Number of compressed flanges: 1
Critical flange contains Initial shape parts: 2, 3, 1

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	1.342	19.276 -44.611	2.31	83.443	3891.418	0.070	1.000	- 1.342	0.253 0.671	-	- -	-
5	5.717	24.665 24.665	1.00	4.000	10.279	1.549	0.554	3.166 -	- -	-	- -	-
7	1.342	19.276 -44.611	2.31	83.443	3891.418	0.070	1.000	- 1.342	0.253 0.671	-	- -	-
9	0.359	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.141	$inch^3$
Mnyo	587.12	lbfft
Resistance factor	0.90	
Unity check	0.11	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.2-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	-77.486 -77.486	-	-	-	-	-	- -	- -	-	- -	-
3	1.342	36.361 -68.630	1.89	57.922	2701.239	0.116	1.000	- 1.342	0.275 0.671	-	- -	-
5	5.717	45.218 45.218	1.00	4.000	10.279	2.097	0.427	2.440 -	- -	-	- -	-
7	1.342	36.361 -68.630	1.89	57.922	2701.239	0.116	1.000	- 1.342	0.275 0.671	-	- -	-
9	0.359	-77.486 -77.486	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sigma,ex	4.282	ksi
Kt	1.00	
Lt	5' 2.000"	ft
Sigma,t	28.819	ksi
Cs	1.00	
CTF	0.80	
Sfy	0.437	$inch^3$
j	3.407	inch
Fcre	74.637	ksi
Fc	45.218	ksi
Scy	0.230	$inch^3$
Mny	865.79	lbfft
Resistance factor	0.90	
Unity check	0.07	-

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	5.717	50.000 50.000	1.00	4.000	10.279	2.206	0.408	2.334 -	- -	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.349	$inch^2$
Pno	17465.16	lbf
Resistance factor	0.85	
Unity check	0.02	-

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	19 5/8	5 1/4	ft
Effective Length factor K	2.50	1.00	
Effective Length	49	5 1/4	ft
Slenderness	258.59	109.11	
Flexural Buckling stress Fcre	4.282	24.050	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	4.282	ksi
Sigma,ey	24.050	ksi
Kt	1.00	
Lt	5 1/4	ft
Sigma,t	28.819	ksi
Sigma,TF	4.163	ksi
Torsional (-Flexural) buckling stress Fcre	4.163	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	3.651 3.651	1.00	0.430	281.003	0.114	1.000	0.359 -	- -	-	- -	-
3	1.342	3.651 3.651	1.00	4.000	186.542	0.140	1.000	1.342 -	0.671 0.671	114.10	- 0.000	0.359
5	5.717	3.651 3.651	1.00	4.000	10.279	0.596	1.000	5.717 -	- -	-	- -	-
7	1.342	3.651 3.651	1.00	4.000	186.542	0.140	1.000	1.342 -	0.671 0.671	114.10	- 0.000	0.359
9	0.359	3.651 3.651	1.00	0.430	281.003	0.114	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	4.163	ksi
lambda, c	3.47	
Fn	3.651	ksi
Ae	0.556	inch^2
Pn	2030.84	lbf
Resistance factor	0.85	
Unity check	0.14	-

Distortional Buckling Strength
According to article E4 and formula (E4.1-2).

Table of values		
Py	27953.53	lbf
L	1' 2.713"	ft
k,phi,fe	219.79	lbf
k,phi,we	160.55	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	inch^2
k,phi,wg	0.009	inch^2
Fd	24.162	ksi
Pcrd	13508.21	lbf
Lambda,d	1.44	
Pn	15149.05	lbf
Resistance factor	0.85	
Unity check	0.02	-

Data		
Lm	5' 2.000"	ft
Lcr	1' 2.713"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

Number of compressed flanges: 2
Critical flange contains Initial shape parts: 8, 7, 9

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (C5.2.1-3)

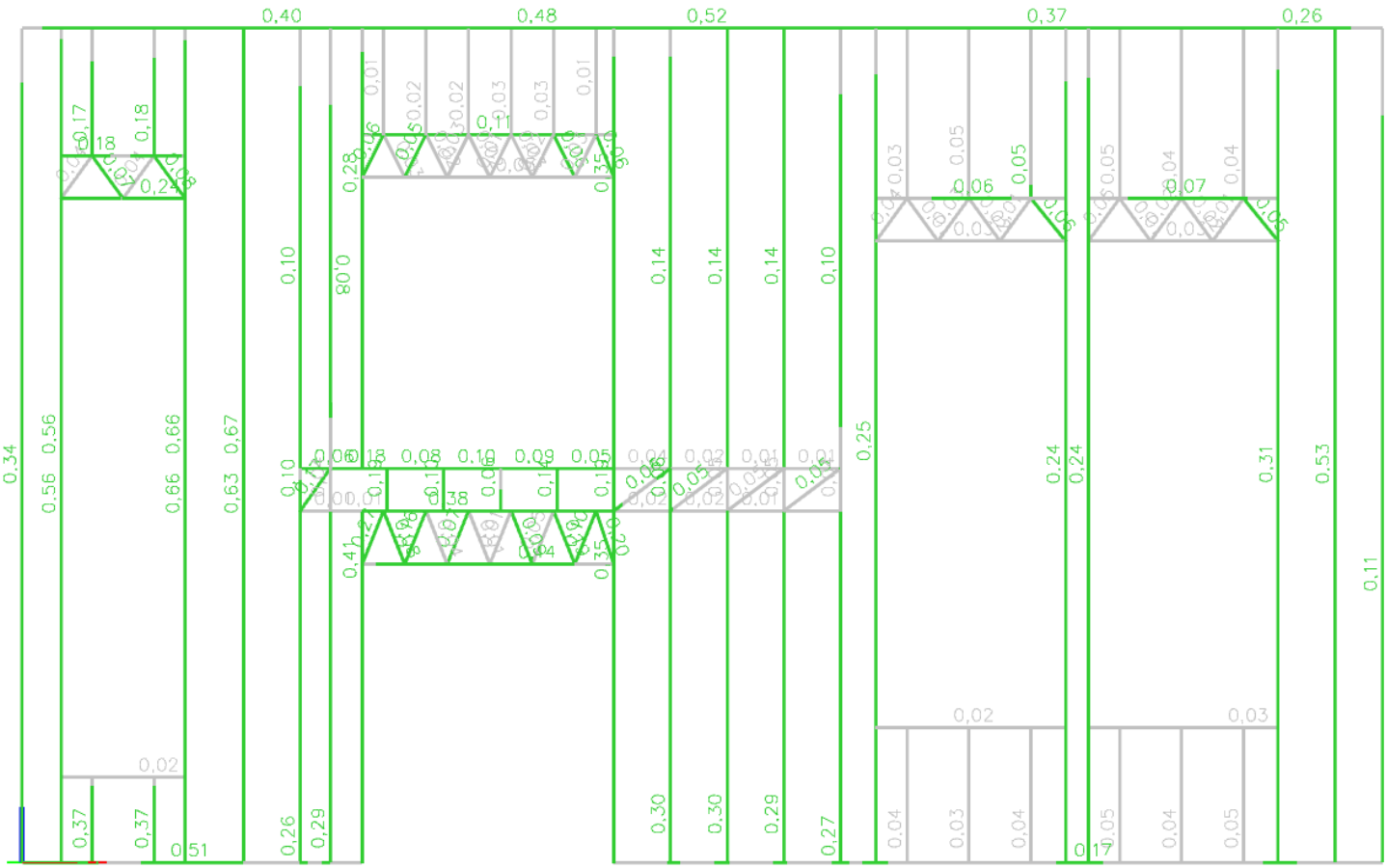
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	0.442 0.442	1.00	0.430	281.003	0.040	1.000	0.359 -	- -	-	- -	-
3	1.342	0.442 0.442	1.00	4.000	186.542	0.049	1.000	1.342 -	0.671 0.671	327.86	- 0.000	0.359
5	5.717	0.442 0.442	1.00	4.000	10.279	0.207	1.000	5.717 -	- -	-	- -	-
7	1.342	0.442 0.442	1.00	4.000	186.542	0.049	1.000	1.342 -	0.671 0.671	327.86	- 0.000	0.359
9	0.359	0.442 0.442	1.00	0.430	281.003	0.040	1.000	0.359 -	- -	-	- -	-

Table of values		
Mnx	2922.36	lbfft
Mny	587.12	lbfft
Pn	2030.84	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.14+0.42+0.11 = 0.67 - (C5.2.1-3)

The member satisfies the check !

Unity check


0,40
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0,37
0,26
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0,66
0,66
0,63
0,67
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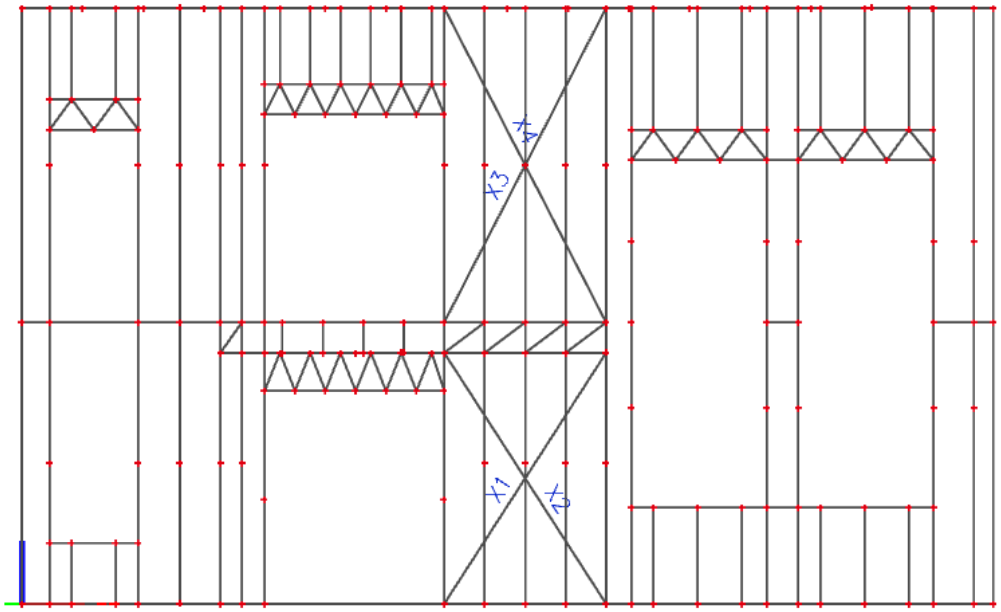
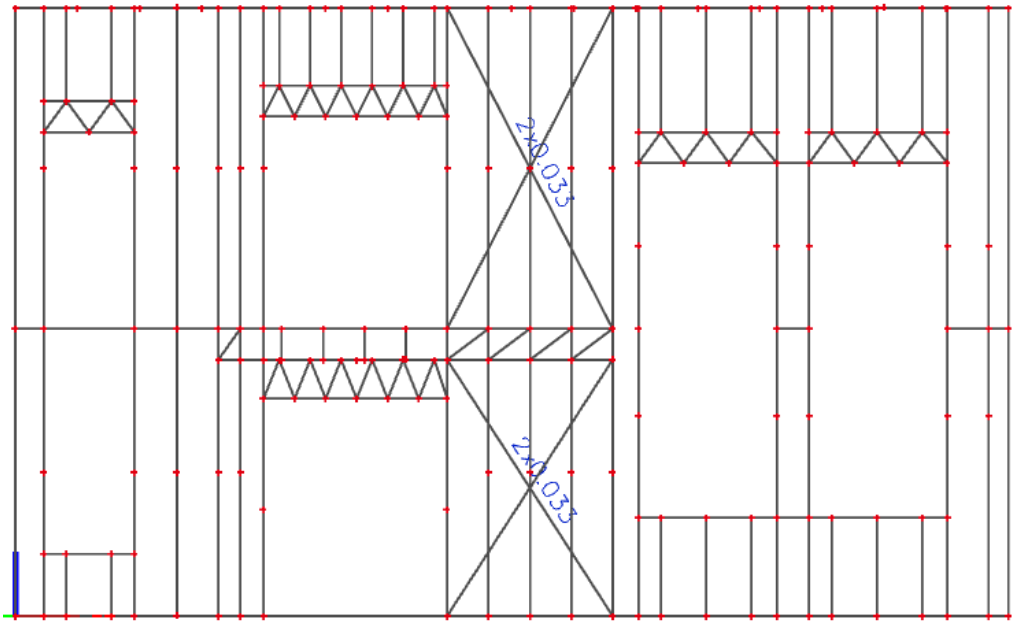
0,10
0,14
0,14
0,14
0,10
0,25
0,24
0,24
0,31
0,53
0,11
0,02
0,03
0,37
0,37
0,51
0,26
0,29
0,30
0,30
0,29
0,27
0,17

X-BRACING STRAP CHECK

Numbers of members



Cross-sections of members



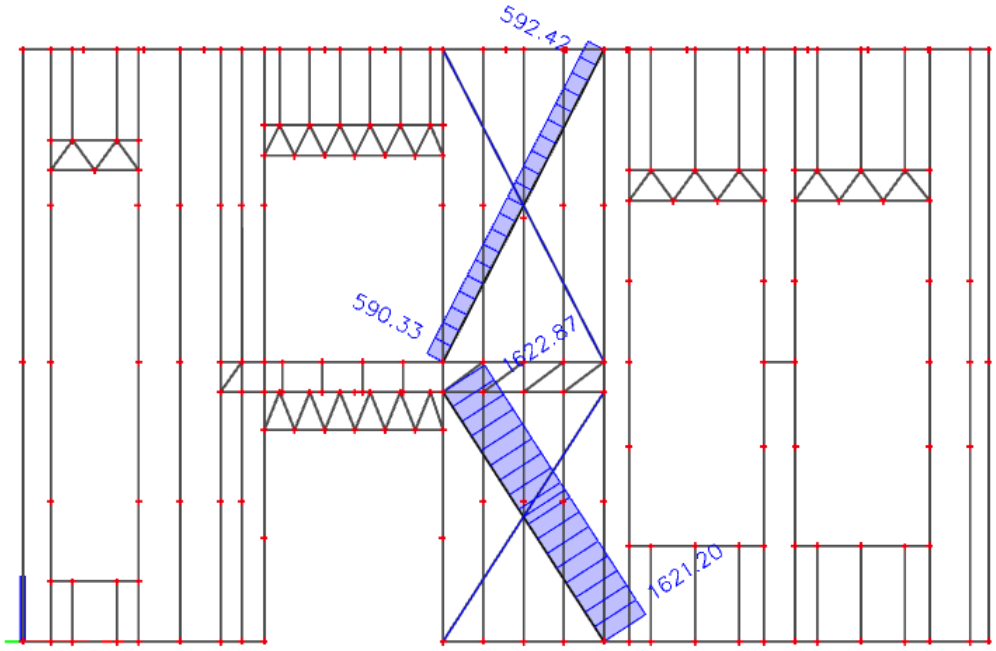
Cross-sections properties of X-BRACING STRAP 2”x0.033”

CS26		
Type	2x0.033	
Shape type	Thin-walled	
Item material	A653 grade 33	
Fabrication	cold formed	
Colour		
A [inch^2]	0.066	
A_y [inch^2], A_z [inch^2]	0.055	0.055
A_L [inch^2/inch], A_D [inch^2/inch]	4.07e+00	4.07e+00
$c_{y,UCS}$ [inch], $c_{z,UCS}$ [inch]	0.000	1.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.022	0.000
i_y [inch], i_z [inch]	0.577	0.010
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0.022	0.000
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	0.033	0.001
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.09e+00	1.09e+00
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.80e-02	1.80e-02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.000
β_y [inch], β_z [inch]	0.000	0.000
Picture		

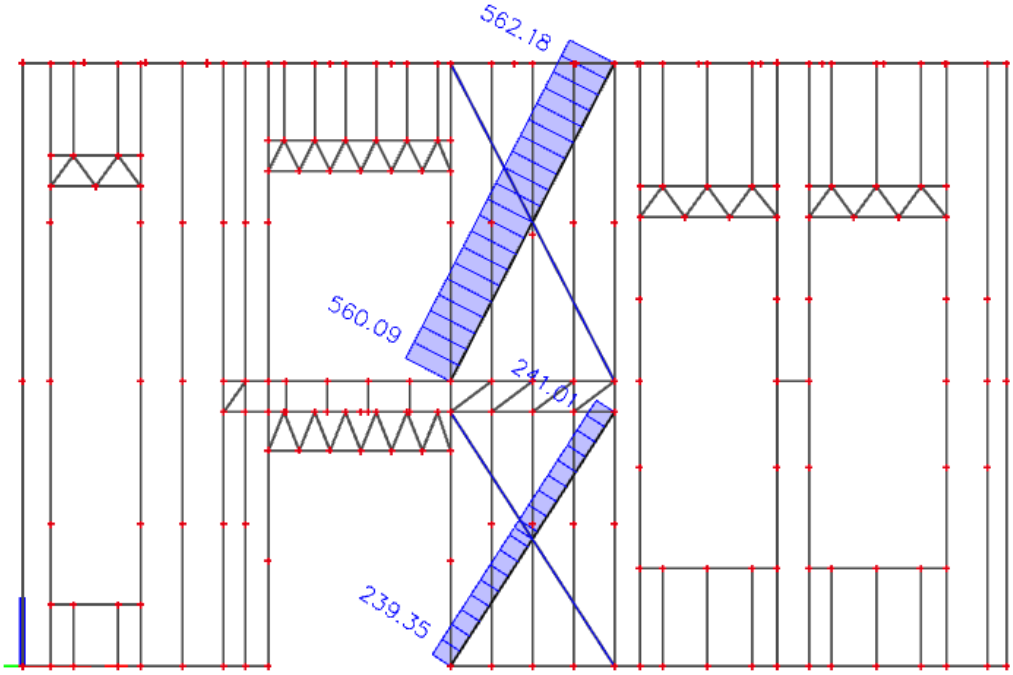
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

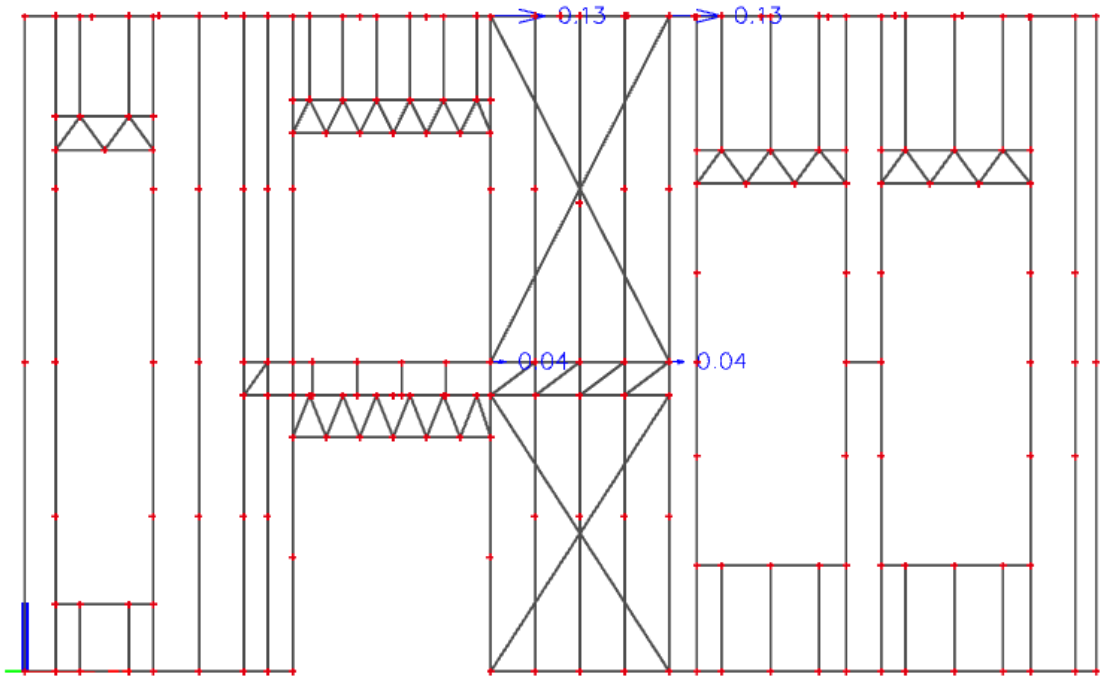
Maximum axial force N,
LRFD-Ult (auto) 70 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-(-0.18)), lbf.



LRFD-Ult (auto) 68 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+(-0.18)), lbf.



Wall Wind load deflection
Value: Ux - Wx+(-0.18), (inch).



The maximum deflection is 0.13" according to Table R301.7 the code IRC-2019 - the deflection limits H/360. L = 19'-7"= 19'*12"+7=235" 235/360 = 0.652"

0.13" < 0.652"

<u>Deflection is OK!</u>

STEEL MEMBER X2 CHECK

AISI S100-16 LRFD Check

Member X2	2x0.033	A653 grade 33	NC_LRFD-Ult (au	0.83

Material data		
Yield stress Fy	33.00	ksi
Tensile stress Fu	45.00	ksi
fabrication	cold formed	

Warning: Part 1 exceeds dimensional limit w/t <= 60! (art. B1.1(3))

The critical check is on position 9.82 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

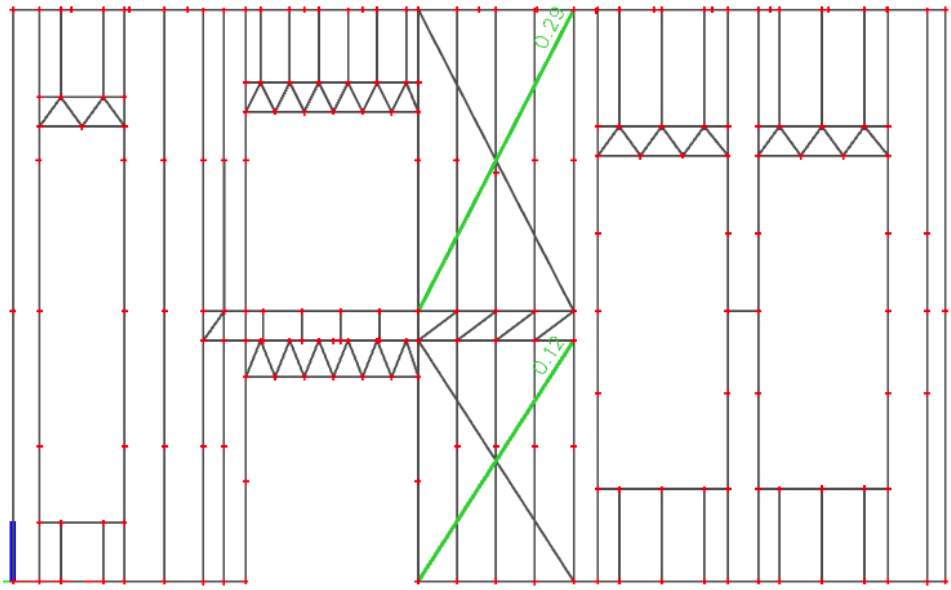
Internal forces		
Pu	1622.87	lbf
Vux	-0.00	lbf
Vuy	-0.00	lbf
Mut	-0.00	lbfft
Mux	0.00	lbfft
Muy	0.00	lbfft

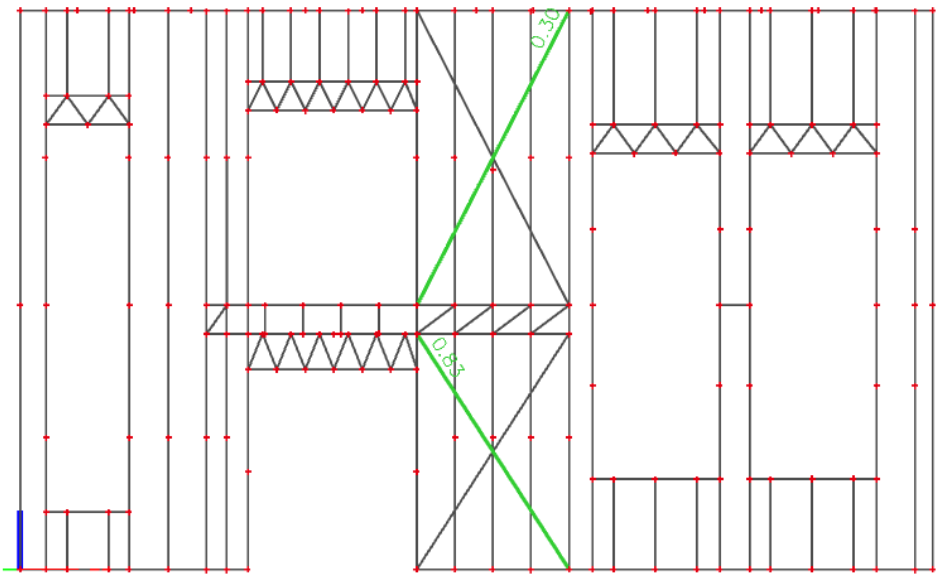
Nominal Tensile Strength
According to article D2 and formula (D2-1).

Table of values		
Tn	2178.00	lbf
Resistance factor	0.90	
Unity check	0.83	-

The member satisfies the check !

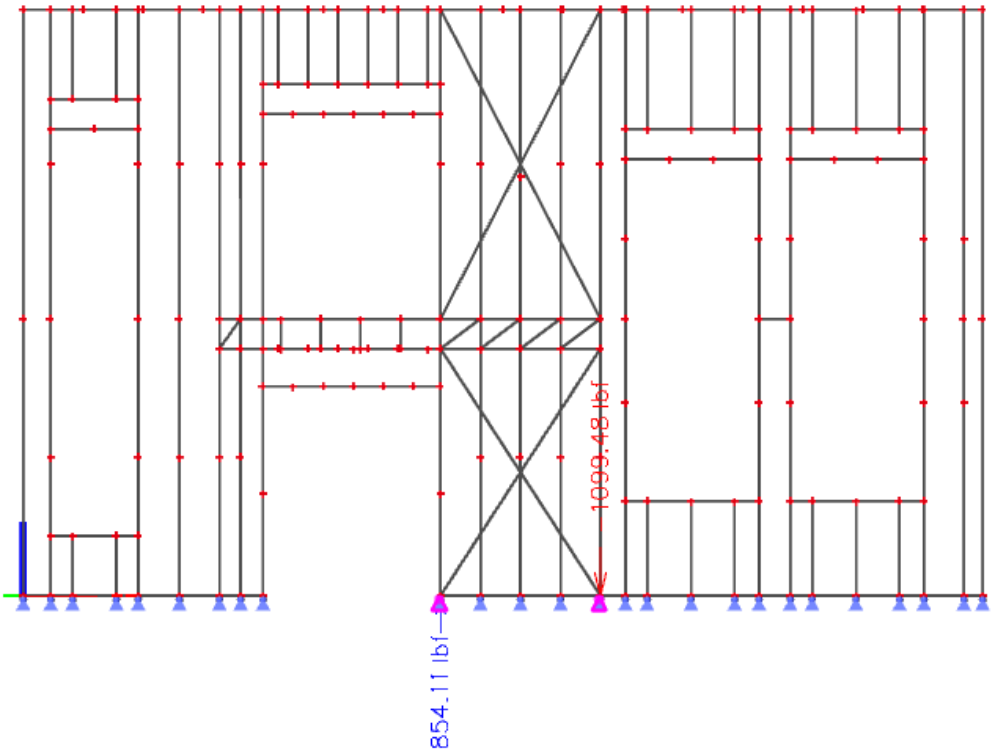
X-bracing Unity check





HOLDOWN DESIGN

Maximum support uplift reactions 1100 lb, from Combination LRFD-Ult (auto) 70 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-(-0.18)).



Accept holdown Simpson S/HDU4 series. Maximum tension load 6345 lb for steel 54 mil.

Model	H (in.)	Fasteners		Stud Member Thickness[2] mil (ga.)	ASD (lb.)		LRFD (lb.)		Nominal Tension Load[6] (lb.)
		Anchor Bolt Diameter[1] (in.)	Stud Fasteners[7]		Tension Load	Deflection at ASD Load[5]	Tension Load	Deflection at LRFD Load[5]	
S/HDU4	7⅞	⅝	(6) #14	2–33 (2–20)	2,320	0.093	3,705	0.149	5,685
				2–43 (2–18)	3,825	0.115	6,105	0.190	9,365
				2–54 (2–16)	3,970	0.093	6,345	0.156	9,730
				Steel fixture	4,470	0.063	7,165	0.103	12,120

HOLDOWN TO FLOOR BEAM BOLT CONNECTIONS CHECK

Tensile resistance check (AISC 360-16: J3-1), 5/8” bolts

$\phi R_n = \phi \cdot F_{nt} * A_b = 10.36$ kip

Where:

$A_b = 0.307$ in2 – gross bolt cross-sectional area A307 − 12 Table 3

$F_{nt} = 45.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$\phi = 0.75$ – capacity factor

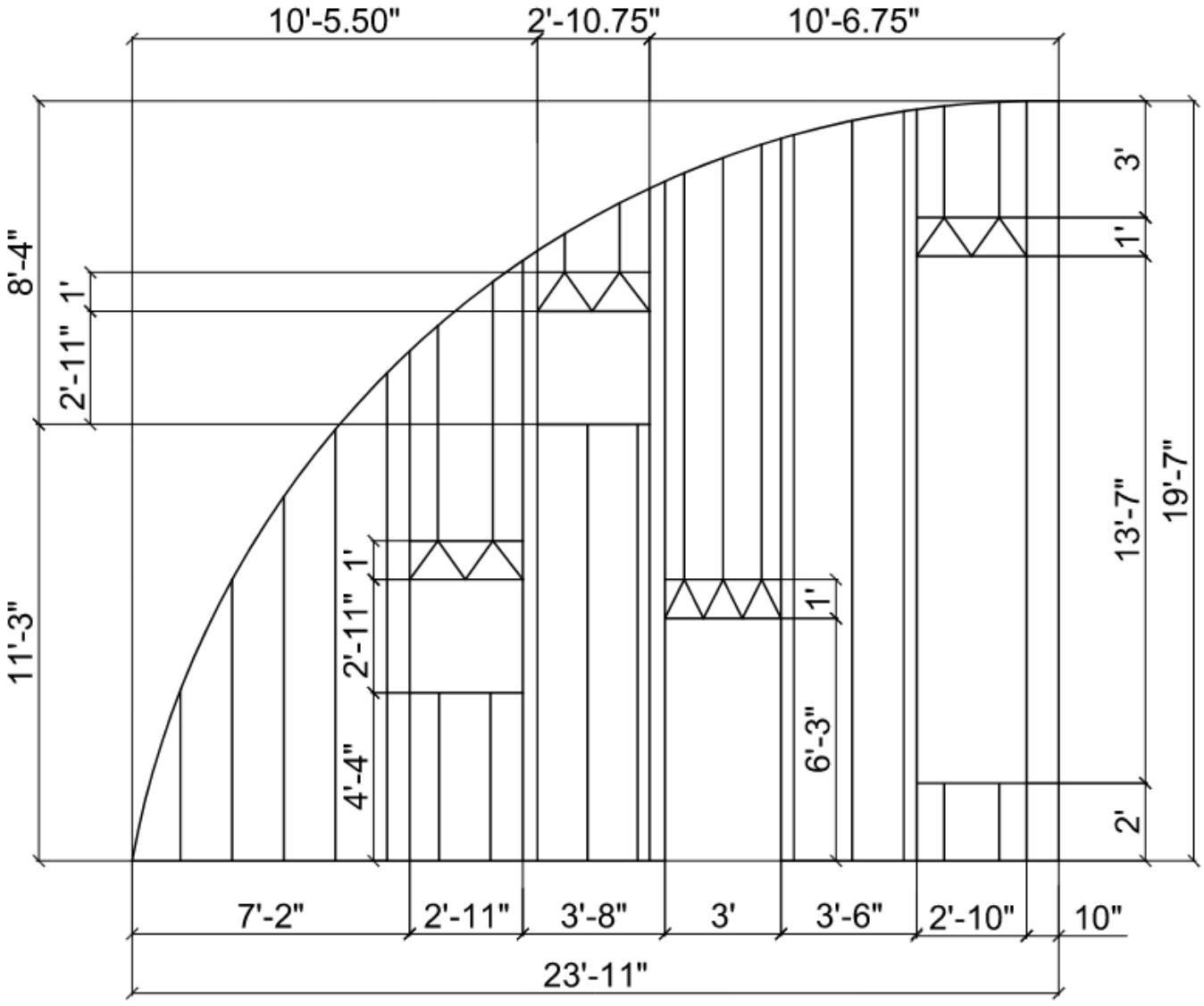
Maximum uplift support reactions - 1099lb = 1.1 kip.

1.1 kip < 10.36 kip

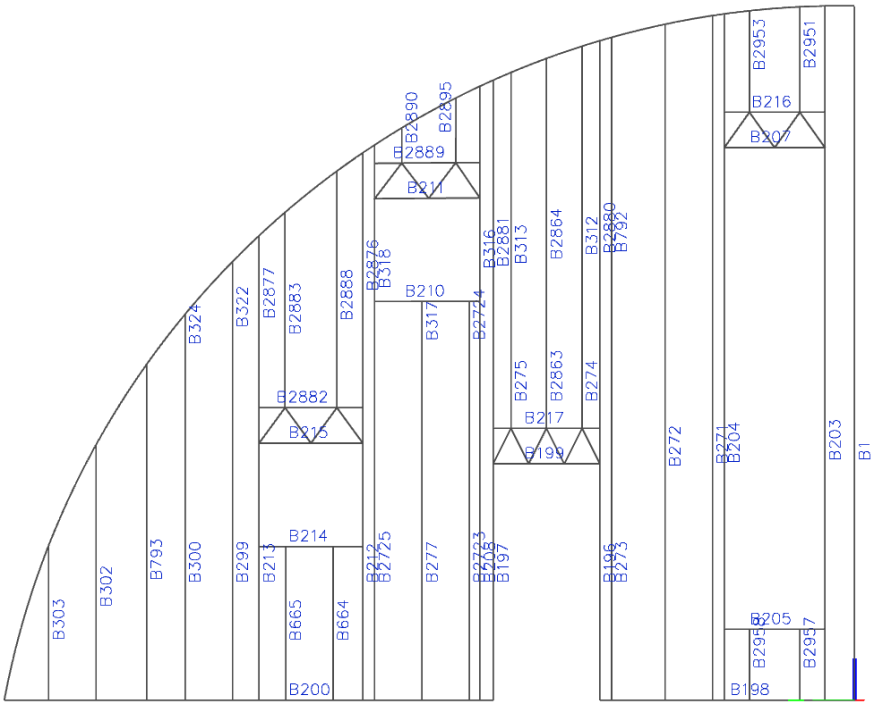
Connection is OK!

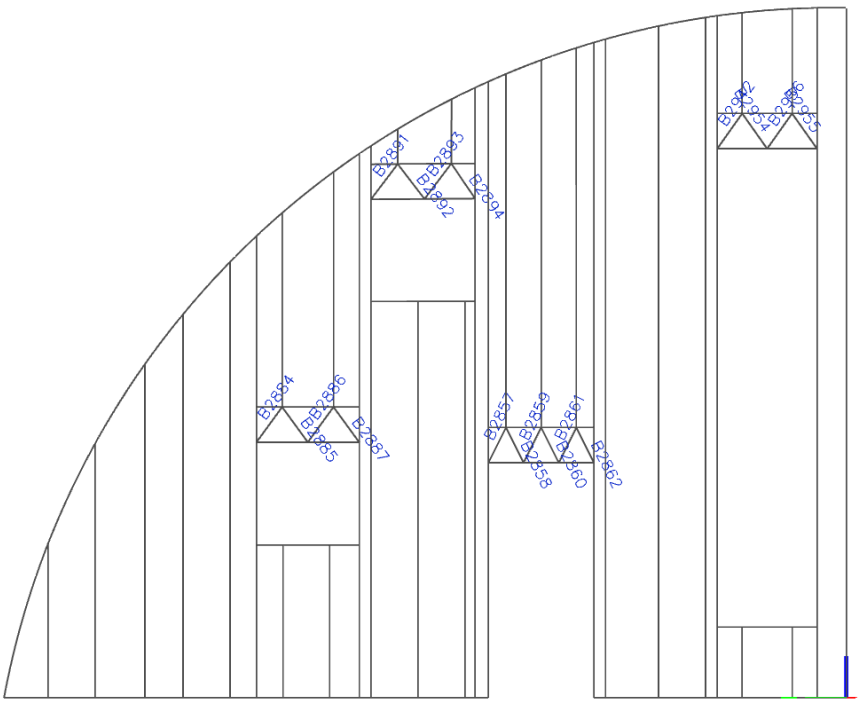
2.4.4 LEFT WALL

Wall stud spacing - 16”;
First story wall stud height - 9’-3”;
Second story wall stud height - 10’-4”.

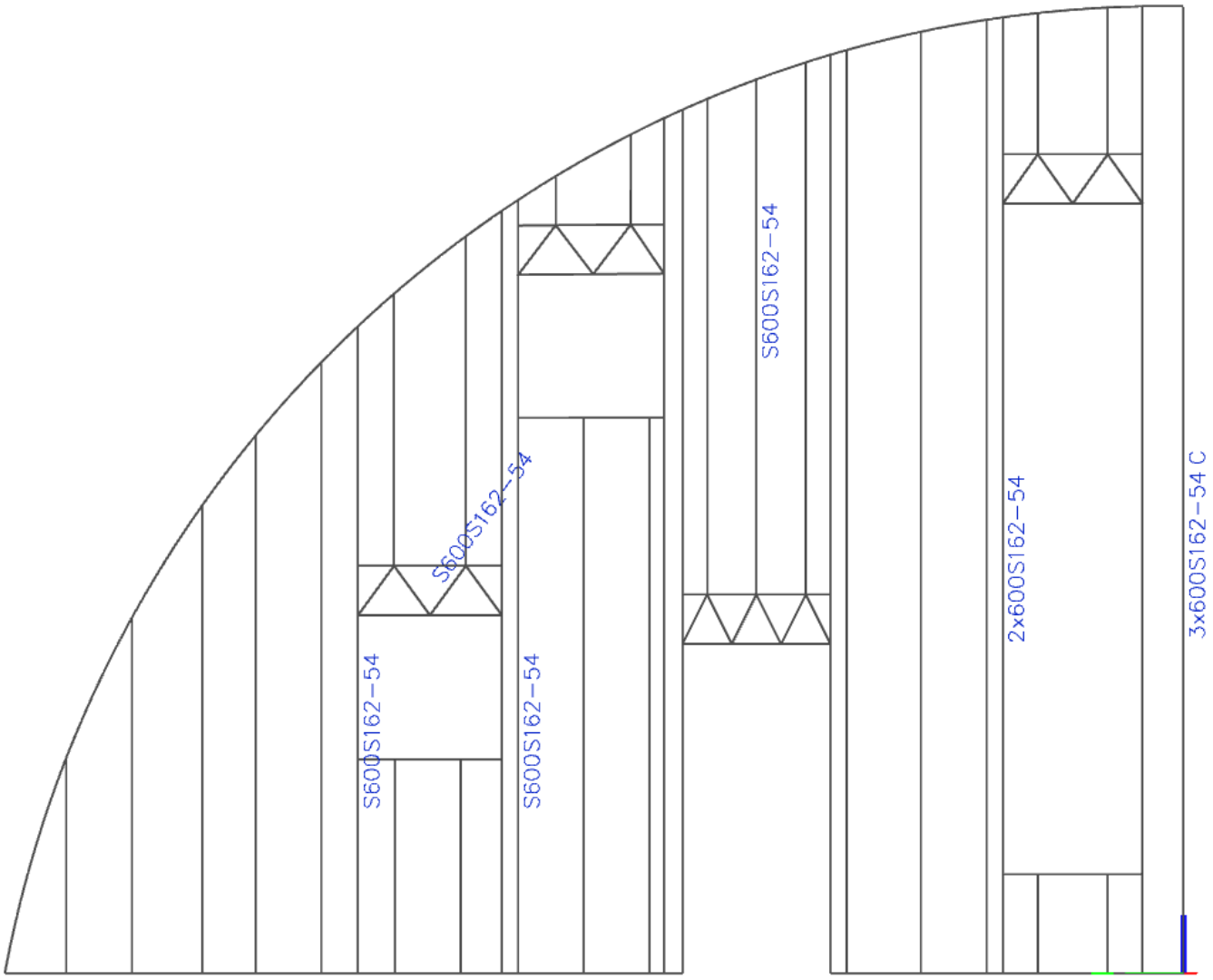


Numbers of members

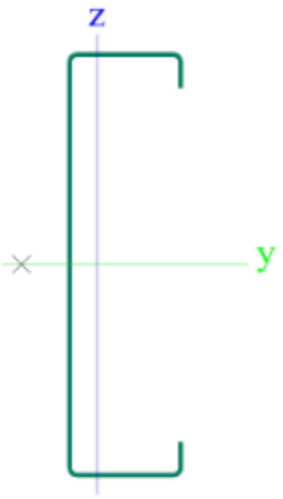




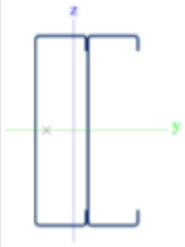
Cross-sections of members



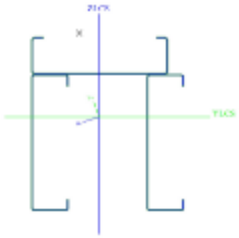
Cross-sections properties of 600S162-54

CS14		
Type	S600S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0,559	
A_y [inch^2], A_z [inch^2]	0,186	0,346
A_L [inch^2/inch], A_D [inch^2/inch]	1,98e+01	1,98e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0,413	3,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	2,886	0,181
i_y [inch], i_z [inch]	2,272	0,568
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0,953	0,149
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	1,140	0,217
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	5,70e+01	5,70e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,08e+01	1,08e+01
d_y [inch], d_z [inch]	-1,056	0,000
I_t [inch^4], I_w [inch^6]	0,001	1,337
β_y [inch], β_z [inch]	0,000	6,508
Picture		

Cross-sections properties of 2x600S162-54

CS11		
Type	2x600S162-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,112	
A_y [inch^2], A_z [inch^2]	0,346	0,676
A_L [inch^2/inch], A_D [inch^2/inch]	2,33e+01	3,81e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	2,948	3,968
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	5,718	1,095
i_y [inch], i_z [inch]	2,267	0,992
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	1,906	0,541
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	2,280	0,904
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,14e+02	1,14e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	4,52e+01	4,52e+01
d_y [inch], d_z [inch]	-0,860	0,000
I_t [inch^4], I_w [inch^6]	1,381	3,828
β_y [inch], β_z [inch]	0,000	3,169
Picture		

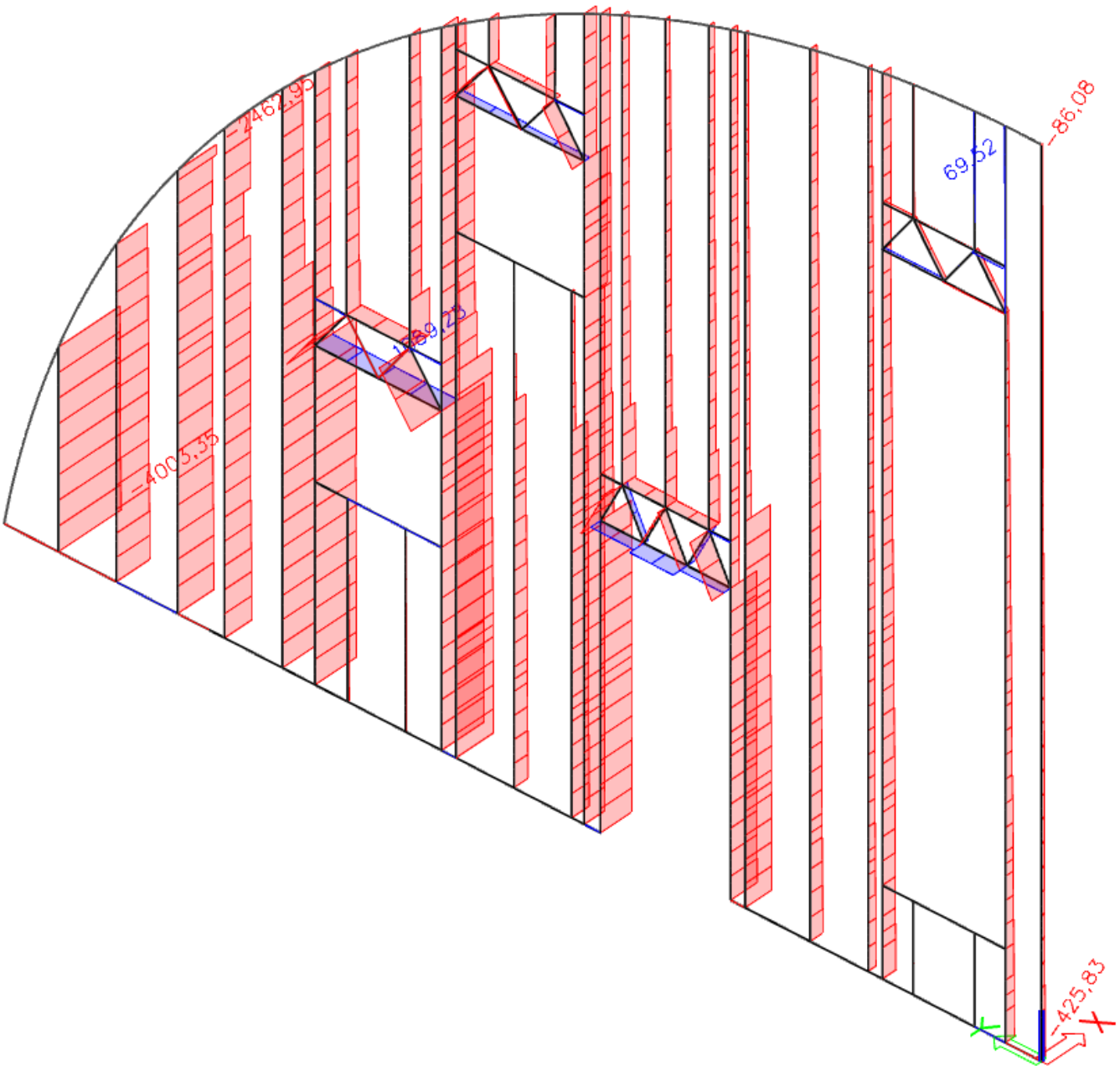
Cross-sections properties of 3x600S162-54 C

CS21		
Type	3x600S162-54 C	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,668	
A_y [inch^2], A_z [inch^2]	1,079	0,840
A_L [inch^2/inch], A_D [inch^2/inch]	5,53e+01	5,53e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	4,407	4,242
$I_{Y,LCS}$ [inch^4], $I_{Z,LCS}$ [inch^4]	10,218	10,372
$I_{YZ,LCS}$ [inch^4]	0,064	
α [deg]	110,01	
I_y [inch^4], I_z [inch^4]	10,396	10,195
i_y [inch], i_z [inch]	2,496	2,472
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	2,497	1,989
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	3,722	3,380
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,86e+02	1,86e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,69e+02	1,69e+02
d_y [inch], d_z [inch]	3,751	-0,439
I_t [inch^4], I_w [inch^6]	0,002	77,572
β_y [inch], β_z [inch]	1,226	-8,898
Picture		

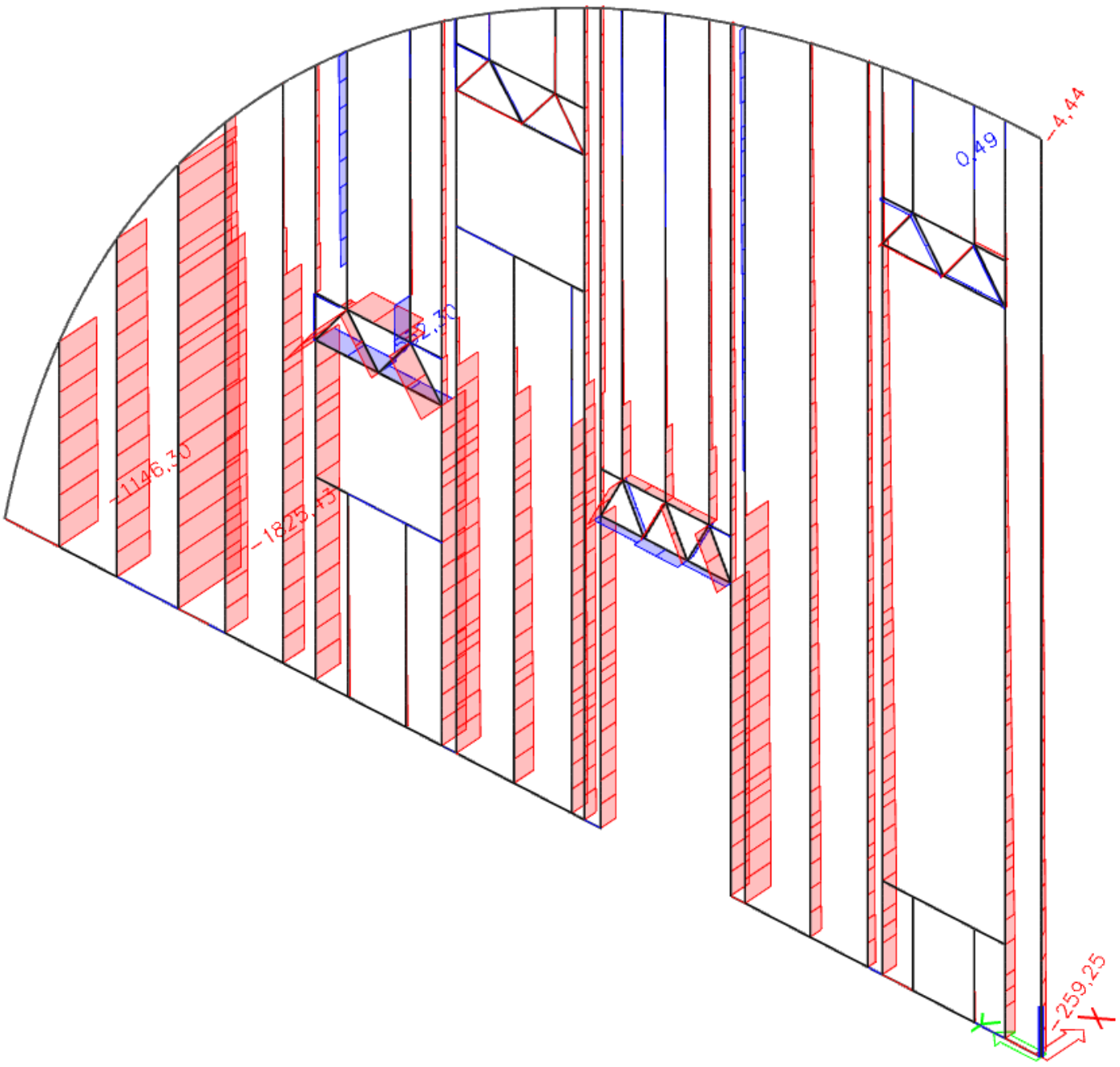
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Axial force diagram N,
LRFD-Ult (auto) 8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1.6*Lr), lbf.



Axial force diagram N,
LRFD-Ult (auto) 68 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+(-0.18)), lbf.



Shear force diagram Vz,
LRFD-Ult (auto) 68 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+(-0.18)), lbf.

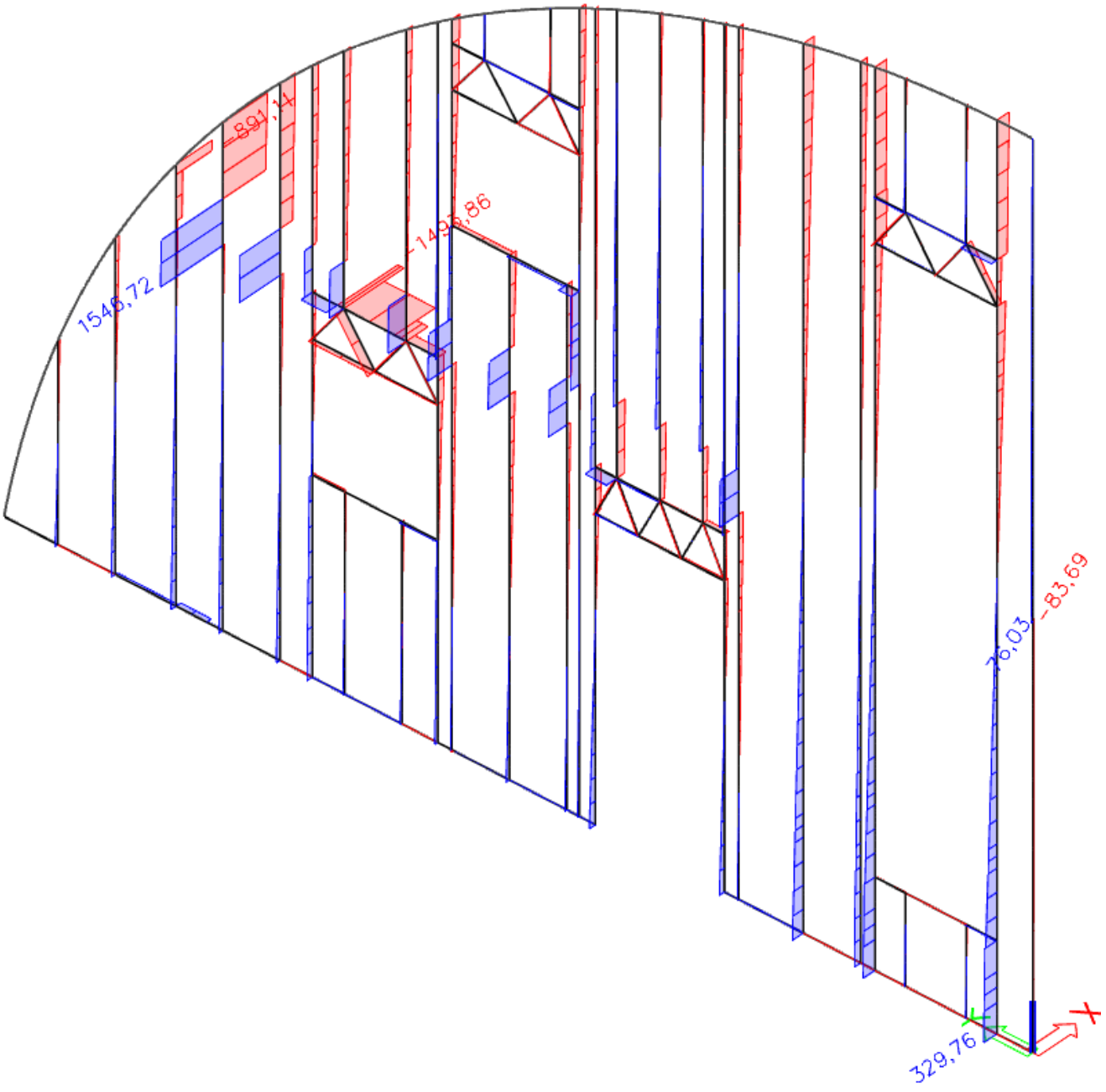
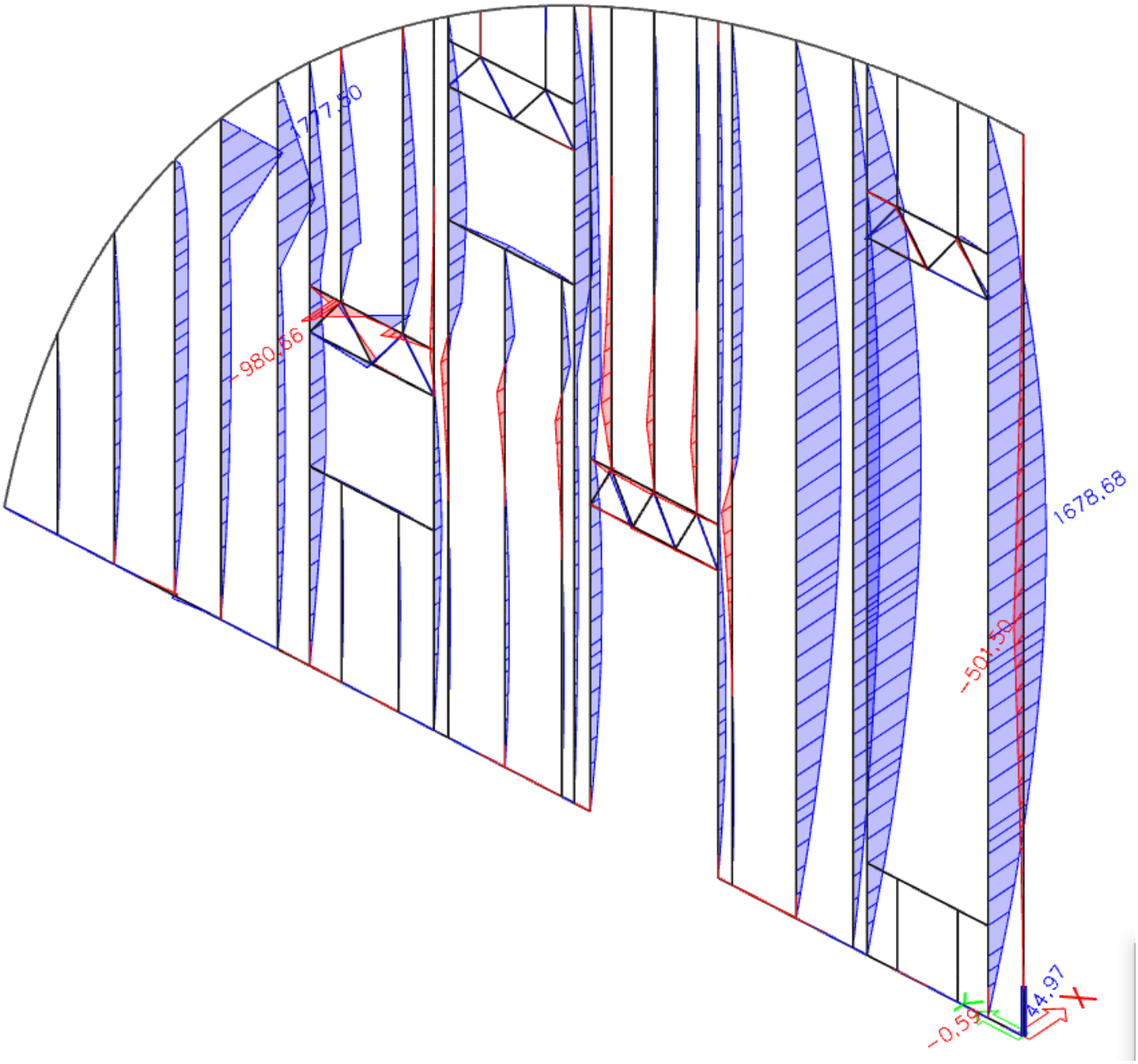
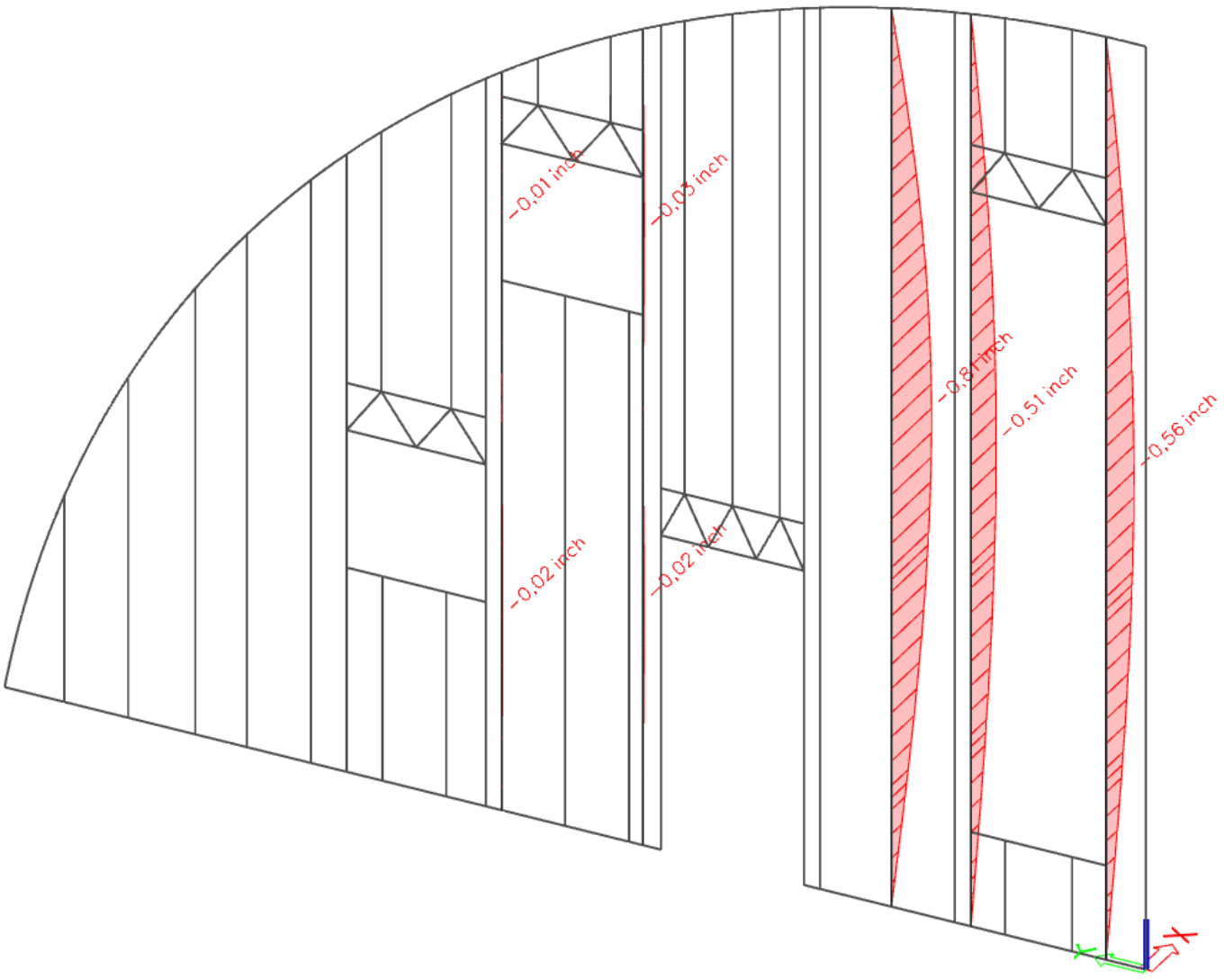


Diagram of bending moments My,
LRFD-Ult (auto) 68 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx+(-0.18)), lbf*ft.



Wall stud deflection by Wind Load Wx+ (0.18), inch.



The maximum horizontal deflection of 0.81”. According to TABLE R301.7 the code IRC 2018, maximum wall member deflection should not exceed - H/240. H = stud height.
19’-7”=235”, 235”/240=0.979”. 0.81"< 0.979”. <u>Deflection is OK!</u>

STEEL MEMBER B203 CHECK

AISI S100-16 LRFD Check

Member B203	2x600S162-54	A913 grade 50	LRFD-Ult (auto)	0.28

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 9.25 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-243.11	lbf
Vux	-30.08	lbf
Vuy	-10.06	lbf
Mut	6.78	lbfft
Mux	-1453.26	lbfft
Muy	-112.55	lbfft

Combined Bending and Torsional Loading
According to article H4 and formula (H4-1)

Table of values		
Critical fibre	37	
Sigma Mx	9.023	ksi
Sigma My	3.384	ksi
f bending	12.407	ksi
Tau t	-0.005	ksi
f torsion	-0.005	ksi
Composed Stress	12.407	ksi
R	1.00	-

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-
6	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-

7	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-

Table of values		
Sxe	1.906	$inch^3$
Mnxo	7941.74	lbfft
Resistance factor	0.90	
Unity check	0.20	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	3' 7.500"	ft
Sigma,ey	148.995	ksi
Kt	1.00	
Lt	3' 7.500"	ft
Sigma,t	2093.529	ksi
Cb	1.05	
Sfx	1.906	$inch^3$
Fcre	892.803	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	-4.257 -4.257	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	41.486 -3.446	0.08	8.707	297.289	0.374	1.000	- 1.568	0.509 1.060	-	- -	-
3	5.943	41.486 41.486	1.00	4.000	9.511	2.089	0.428	2.546 -	- -	-	- -	-
4	1.568	41.486 -3.446	0.08	8.707	297.289	0.374	1.000	- 1.568	0.509 1.060	-	- -	-
5	0.472	-4.257 -4.257	-	-	-	-	-	- -	- -	-	- -	-
6	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	-5.068 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	5.000	-5.068 -5.068	-	-	-	-	-	- -	- -	-	- -	-
9	1.568	-5.068 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.447	$inch^3$
Mnyo	1862.75	lbfft
Resistance factor	0.90	
Unity check	0.07	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	26.656	ksi
Kt	1.00	
Lt	3' 7.500"	ft
Sigma,t	2093.529	ksi
Cb	1.00	
Sfy	0.893	$inch^3$
Fcre	770.543	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 50.000	1.00	4.000	9.511	2.293	0.394	2.343 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
8	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.779	$inch^2$
Pno	38928.07	lbf
Resistance factor	0.85	
Unity check	0.01	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	19 5/8	3 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	19 5/8	3 5/8	ft
Slenderness	103.63	43.83	
Flexural Buckling stress Fcre	26.656	148.995	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	26.656	ksi
Sigma,ey	148.995	ksi
Kt	1.00	
Lt	3 5/8	ft
Sigma,t	2093.529	ksi
Sigma,TF	26.619	ksi
Torsional (-Flexural) buckling stress Fcre	26.619	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	22.779 22.779	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
2	1.568	22.779 22.779	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
3	5.943	22.779 22.779	1.00	4.000	9.511	1.548	0.554	3.294 -	- -	-	- -	-
4	1.568	22.779 22.779	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
5	0.472	22.779 22.779	1.00	4.000	6039.613	0.061	1.000	0.472 -	- -	-	- -	-
6	0.472	22.779 22.779	1.00	0.430	162.315	0.375	1.000	0.472 -	- -	-	- -	-
7	1.568	22.779 22.779	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
8	5.000	22.779 22.779	1.00	4.000	13.438	1.302	0.638	3.191 -	- -	-	- -	-
9	1.568	22.779 22.779	1.00	4.000	136.574	0.408	1.000	1.568 -	- -	-	- -	-
10	0.472	22.779 22.779	1.00	0.430	162.315	0.375	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	26.619	ksi
lambda, c	1.37	
Fn	22.779	ksi
Ae	0.883	inch^2
Pn	20115.05	lbf
Resistance factor	0.85	
Unity check	0.01	-

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (C5.2.1-3)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	0.219 0.219	1.00	4.000	6039.613	0.006	1.000	0.472 -	- -	-	- -	-
2	1.568	0.219 0.219	1.00	4.000	136.574	0.040	1.000	1.568 -	- -	-	- -	-
3	5.943	0.219 0.219	1.00	4.000	9.511	0.152	1.000	5.943 -	- -	-	- -	-
4	1.568	0.219 0.219	1.00	4.000	136.574	0.040	1.000	1.568 -	- -	-	- -	-
5	0.472	0.219 0.219	1.00	4.000	6039.613	0.006	1.000	0.472 -	- -	-	- -	-
6	0.472	0.219 0.219	1.00	0.430	162.315	0.037	1.000	0.472 -	- -	-	- -	-
7	1.568	0.219 0.219	1.00	4.000	136.574	0.040	1.000	1.568 -	- -	-	- -	-
8	5.000	0.219 0.219	1.00	4.000	13.438	0.128	1.000	5.000 -	- -	-	- -	-
9	1.568	0.219 0.219	1.00	4.000	136.574	0.040	1.000	1.568 -	- -	-	- -	-
10	0.472	0.219 0.219	1.00	0.430	162.315	0.037	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	7941.74	lbfft
Mny	1862.75	lbfft
Pn	20115.05	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.01+0.20+0.07 = 0.28 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B277 CHECK

AISI S100-16 LRFD Check

Member B277	S600S162-54	A913 grade 50	LRFD-Ult (auto)	0.93

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 8.25 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-775.78	lbf
Vux	-2.08	lbf
Vuy	-26.67	lbf
Mut	0.10	lbfft
Mux	-946.03	lbfft
Muy	-5.79	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	48.118 42.159	0.88	0.475	310.586	0.394	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	5.717	48.118 -46.914	0.97	23.357	60.020	0.895	0.842	- 4.816	1.212 2.408	-	- -	-
7	1.342	-48.795 -48.795	-	-	-	-	-	- -	- -	-	- -	-
9	0.359	-40.954 -46.914	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.925	inch^3
Mnxo	3854.26	lbfft
Resistance factor	0.90	
Unity check	0.27	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	15.973 13.969	0.87	0.476	310.987	0.227	1.000	0.359 -	- -	-	- -	-
3	1.342	16.605 16.605	1.00	3.387	157.959	0.324	1.000	1.342 -	0.671 0.671	53.50	0.000 0.000	0.359
5	5.717	15.973 -15.973	1.00	24.000	61.673	0.509	1.000	- 5.717	1.429 2.859	-	- -	-
7	1.342	-16.605 -16.605	-	-	-	-	-	- -	- -	-	- -	-
9	0.359	-13.969 -15.973	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Litb	9' 3.000"	ft
Sigma,ey	7.503	ksi
Kt	1.00	
Lt	9' 3.000"	ft
Sigma,t	10.251	ksi
Cb	1.27	
Sfx	0.962	inch^3
Fcre	16.605	ksi
Fc	16.605	ksi
Scx	0.953	inch^3

Table of values		
Mnx	1319.37	lbfft
Resistance factor	0.90	
Unity check	0.80	-

Distortional Buckling Strength
According to article F4 and formula F4.1-2.

Table of values		
Sfy	0.962	inch^3
My	4008.12	lbfft
L	1' 1.314"	ft
Beta	1.09	
k,phi,fe	311.83	lbf
k,phi,we	293.13	lbf
k,phi	0.00	lbf
k,phi,fg	0.008	inch^2
k,phi,wg	0.002	inch^2
Fd	67.547	ksi
Sf	0.962	inch^3
Mcrd	5414.70	lbfft
Lambda,d	0.86	
Mn	3467.39	lbfft
Resistance factor	0.90	
Unity check	0.30	-

Data		
Lm	9' 3.000"	ft
Lcr	1' 1.314"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	-0.052	inch
Ksi,web	2.00	

Number of compressed flanges: 1
Critical flange contains Initial shape parts: 2, 3, 1

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	1.342	19.276 -44.611	2.31	83.443	3891.418	0.070	1.000	- 1.342	0.253 0.671	-	- -	-
5	5.717	24.665 24.665	1.00	4.000	10.279	1.549	0.554	3.166 -	- -	-	- -	-
7	1.342	19.276 -44.611	2.31	83.443	3891.418	0.070	1.000	- 1.342	0.253 0.671	-	- -	-
9	0.359	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	0.141	$inch^3$
Mnyo	587.12	lbfft
Resistance factor	0.90	
Unity check	0.01	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.2-1).

Table of values		
Sigma,ex	31.457	ksi
Kt	1.00	
Lt	9' 3.000"	ft
Sigma,t	10.251	ksi
Cs	1.00	
CTF	0.91	
Sfy	0.437	$inch^3$
j	3.407	inch
Fcre	315.629	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...
Shear Strength
According to article G2.1 and formula (G2.1.1)
Shear force Vy

Element ID	Aw [inch^2]	Vn [lbf]
3	0.000	0.00
5	0.324	4441.21
7	0.000	0.00

Table of values		
Vn,y	4441.21	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnxo	3854.26	lbfft
Vny	4441.21	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.07+0.00) = 0.27

...::Axial Compression Strength::...
Nominal Axial Strength
According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	5.717	50.000 50.000	1.00	4.000	10.279	2.206	0.408	2.334 -	- -	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.349	inch^2
Pno	17465.16	lbf
Resistance factor	0.85	
Unity check	0.05	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	non-sway	
Unbraced Length L	8 1/4	9 1/4	ft
Effective Length factor K	2.19	0.63	
Effective Length	18 1/8	5 7/8	ft
Slenderness	95.40	122.22	
Flexural Buckling stress Fcre	31.457	19.166	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	31.457	ksi
Sigma,ey	19.166	ksi
Kt	1.00	
Lt	9 1/4	ft
Sigma,t	10.251	ksi
Sigma,TF	9.548	ksi
Torsional (-Flexural) buckling stress Fcre	9.548	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	8.373 8.373	1.00	0.430	281.003	0.173	1.000	0.359 -	- -	-	- -	-
3	1.342	8.373 8.373	1.00	4.000	186.542	0.212	1.000	1.342 -	0.671 0.671	75.34	- 0.000	0.359
5	5.717	8.373 8.373	1.00	4.000	10.279	0.903	0.838	4.790 -	- -	-	- -	-
7	1.342	8.373 8.373	1.00	4.000	186.542	0.212	1.000	1.342 -	0.671 0.671	75.34	- 0.000	0.359
9	0.359	8.373 8.373	1.00	0.430	281.003	0.173	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	9.548	ksi
lambda, c	2.29	
Fn	8.373	ksi
Ae	0.504	inch^2
Pn	4218.73	lbf
Resistance factor	0.85	
Unity check	0.22	-

Distortional Buckling Strength
According to article E4 and formula (E4.1-2).

Table of values		
Py	27953.53	lbf
L	1' 2.713"	ft
k,phi,fe	219.79	lbf
k,phi,we	160.55	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	$inch^2$
k,phi,wg	0.009	$inch^2$
Fd	24.162	ksi
Pcrd	13508.21	lbf
Lambda,d	1.44	
Pn	15149.05	lbf
Resistance factor	0.85	
Unity check	0.06	-

Data		
Lm	9' 3.000"	ft
Lcr	1' 2.713"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

Number of compressed flanges: 2
Critical flange contains Initial shape parts: 8, 7, 9

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (H1.2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	1.388 1.388	1.00	0.430	281.003	0.070	1.000	0.359 -	- -	-	- -	-
3	1.342	1.388 1.388	1.00	4.000	186.542	0.086	1.000	1.342 -	0.671 0.671	185.07	- 0.000	0.359
5	5.717	1.388 1.388	1.00	4.000	10.279	0.367	1.000	5.717 -	- -	-	- -	-
7	1.342	1.388 1.388	1.00	4.000	186.542	0.086	1.000	1.342 -	0.671 0.671	185.07	- 0.000	0.359
9	0.359	1.388 1.388	1.00	0.430	281.003	0.070	1.000	0.359 -	- -	-	- -	-

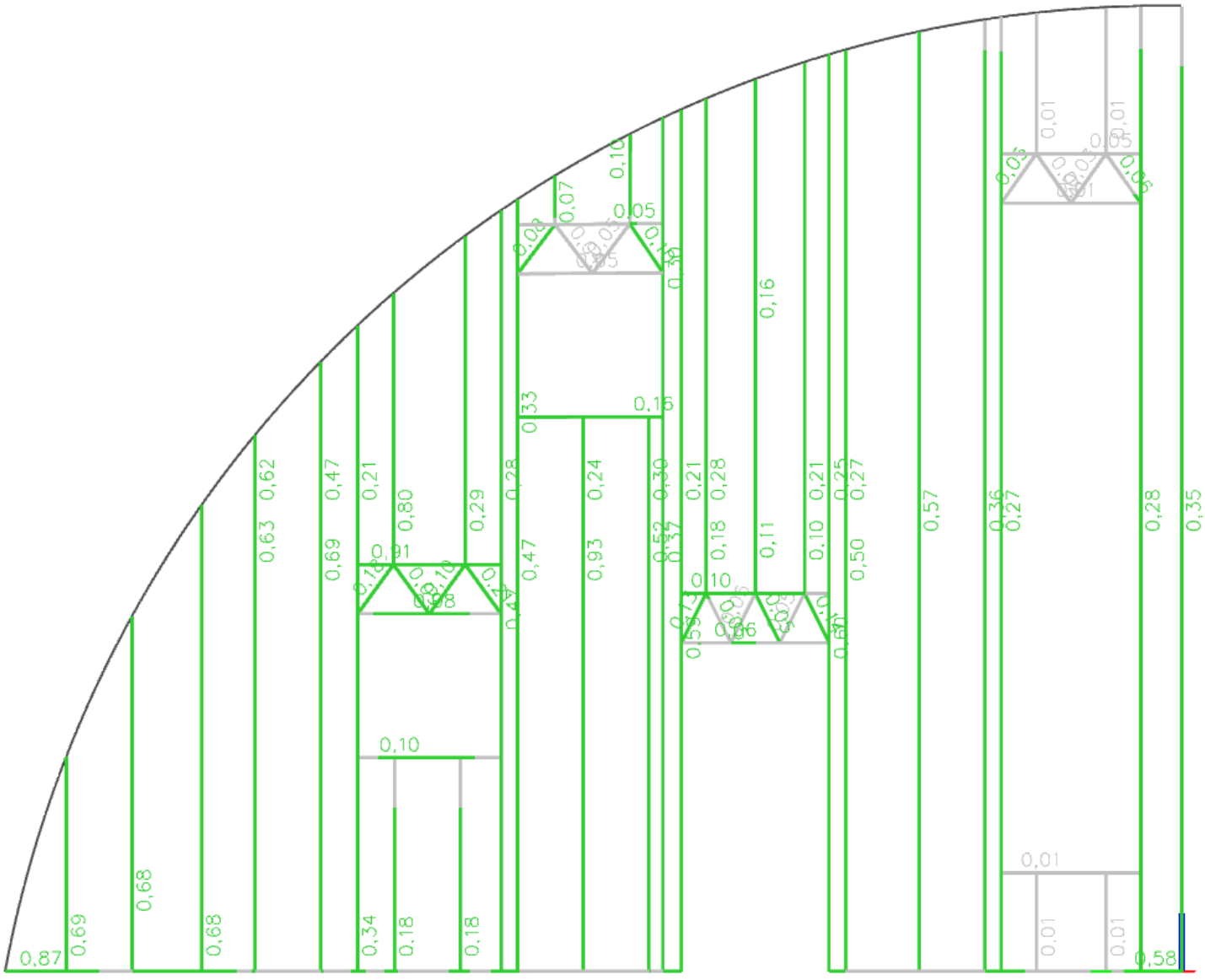
Table of values		
Mnx	1319.37	lbfft
Mny	587.12	lbfft
PEx	17586.43	lbf
PEy	10714.95	lbf
Alfa x	0.96	
Alfa y	0.93	

Table of values		
Cmx	0.85	
Cmy	0.85	
Pn	4218.73	lbf
Pno	17465.16	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.22+0.71+0.01 = 0.93 - (H1.2-1)
Unity check = 0.05+0.80+0.01 = 0.86 -

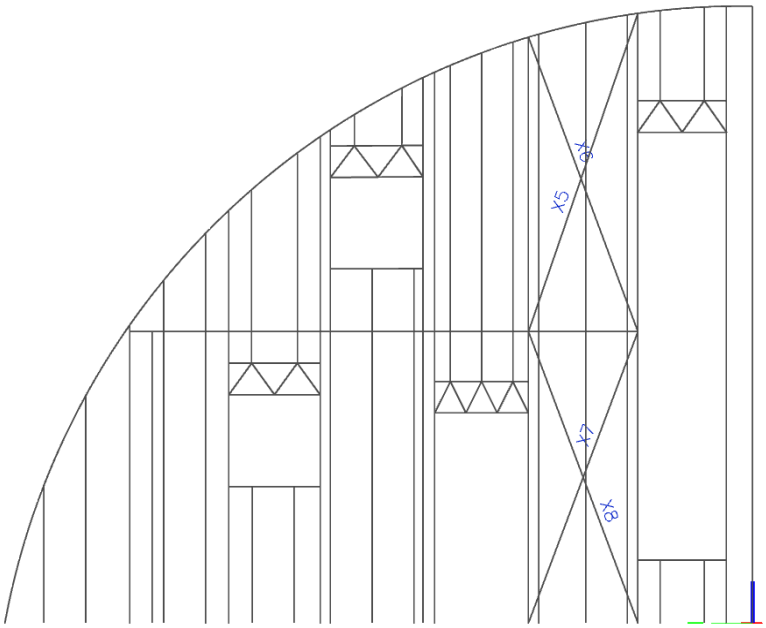
The member satisfies the check !

Unity check

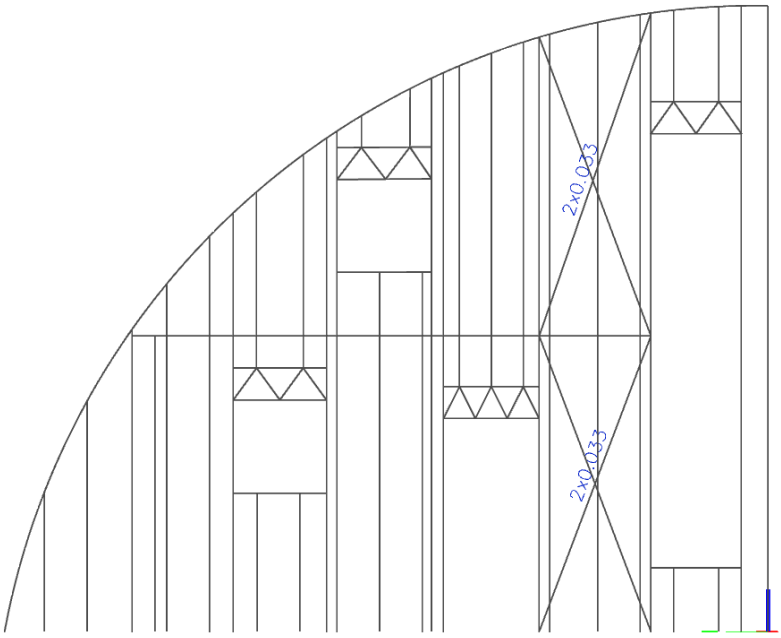

0,87
0,69
0,68
0,68
0,62
0,63
0,47
0,69
0,21
0,80
0,34
0,18
0,18
0,29
0,28
0,47
0,10
0,07
0,05
0,33
0,15
0,24
0,93
0,30
0,21
0,28
0,18
0,11
0,16
0,10
0,21
0,25
0,27
0,50
0,57
0,36
0,27
0,01
0,01
0,01
0,28
0,35
0,01
0,01
0,01
0,58

X-BRACING STRAP CHECK

Numbers of members



Cross-sections of members



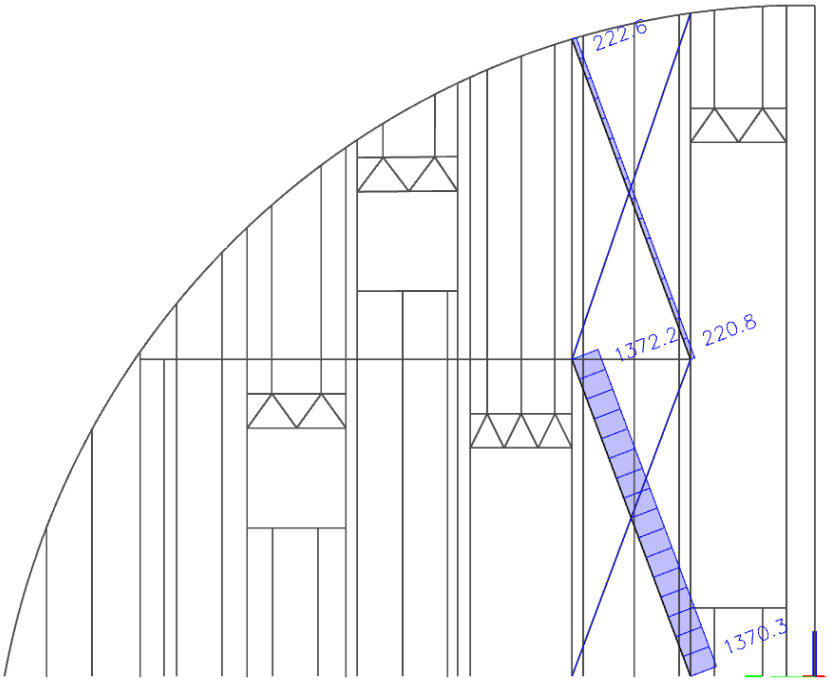
Cross-sections properties of X-BRACING STRAP 2”x0.033”

CS26		
Type	2x0.033	
Shape type	Thin-walled	
Item material	A653 grade 33	
Fabrication	cold formed	
Colour		
A [inch^2]	0.066	
A_y [inch^2], A_z [inch^2]	0.055	0.055
A_L [inch^2/inch], A_D [inch^2/inch]	4.07e+00	4.07e+00
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0.000	1.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.022	0.000
i_y [inch], i_z [inch]	0.577	0.010
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0.022	0.000
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	0.033	0.001
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.09e+00	1.09e+00
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.80e-02	1.80e-02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.000
β_y [inch], β_z [inch]	0.000	0.000
Picture		

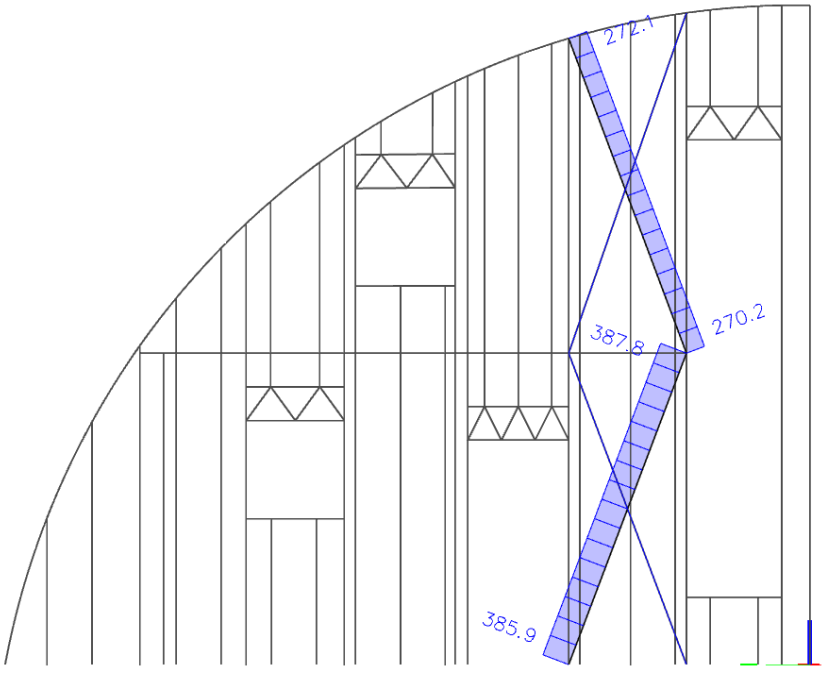
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

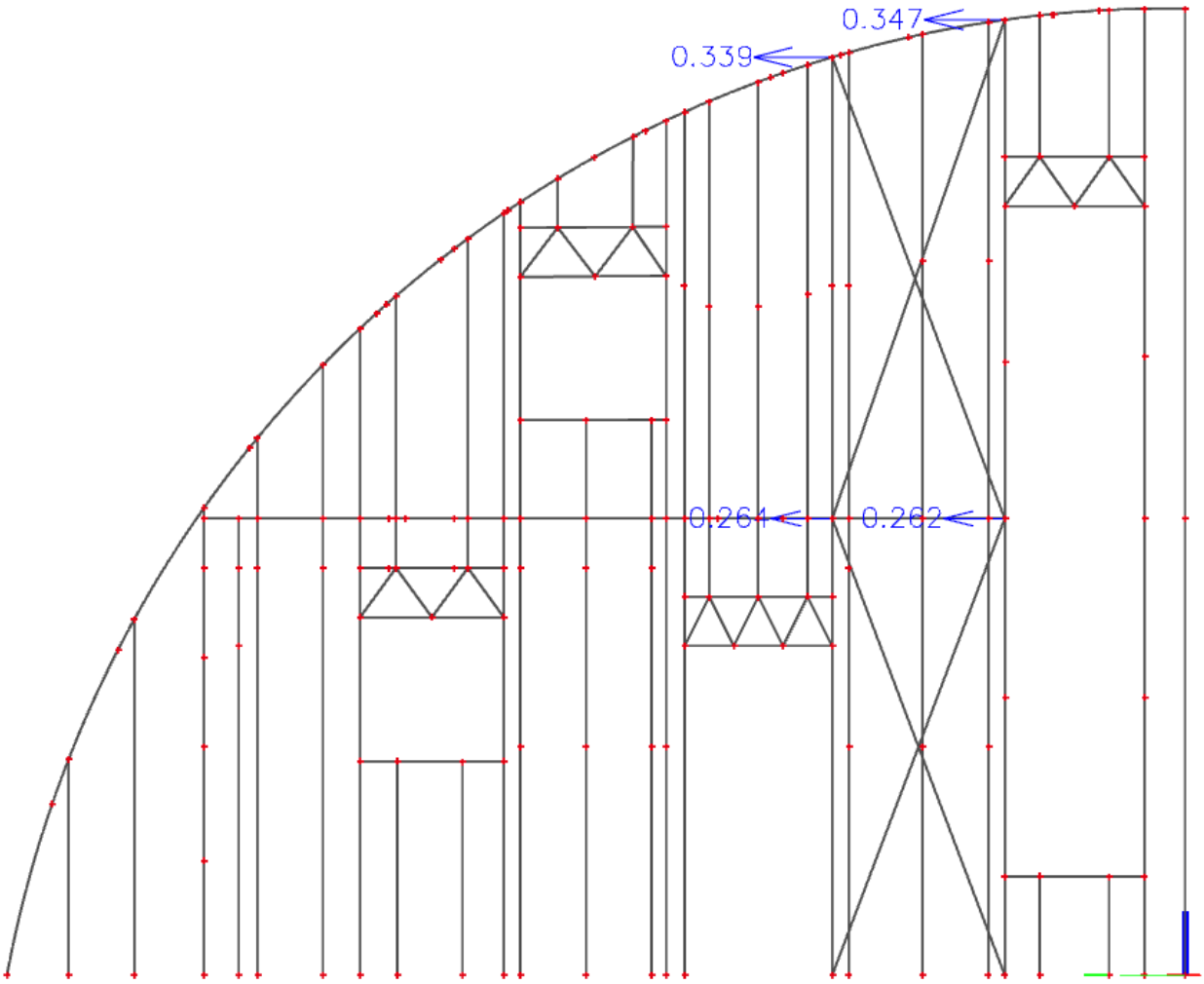
Maximum axial force N,
LRFD-Ult (auto) 71 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy+(0.18)), lbf.



LRFD-Ult (auto) 73 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy-(0.18)), lbf.



Wall Wind load deflection
Value: Uy - Wy+(-0.18), (inch).



The maximum deflection is 0.347" according to Table R301.7 the code IRC-2019 - the deflection limits H/360. L = 19'-4"= 19'*12"+4=232" 232/360 = 0.644"

0.347" < 0.644"

Deflection is OK!

STEEL MEMBER X8 CHECK

AISI S100-16 LRFD Check

Member X8	2x0.033	A653 grade 33	NC_LRFD-Ult (au	0.70

Material data		
Yield stress Fy	33.00	ksi
Tensile stress Fu	45.00	ksi
fabrication	cold formed	

Warning: Part 1 exceeds dimensional limit w/t <= 60! (art. B1.1(3))

The critical check is on position 9.89 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

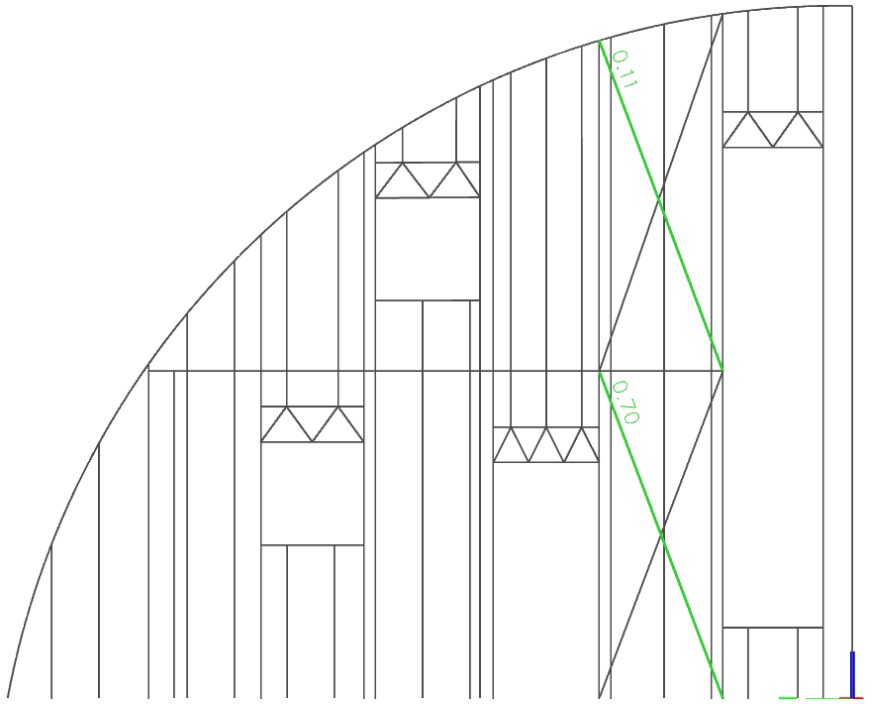
Internal forces		
Pu	1372.19	lbf
Vux	-0.00	lbf
Vuy	-0.00	lbf
Mut	-0.00	lbfft
Mux	0.00	lbfft
Muy	0.00	lbfft

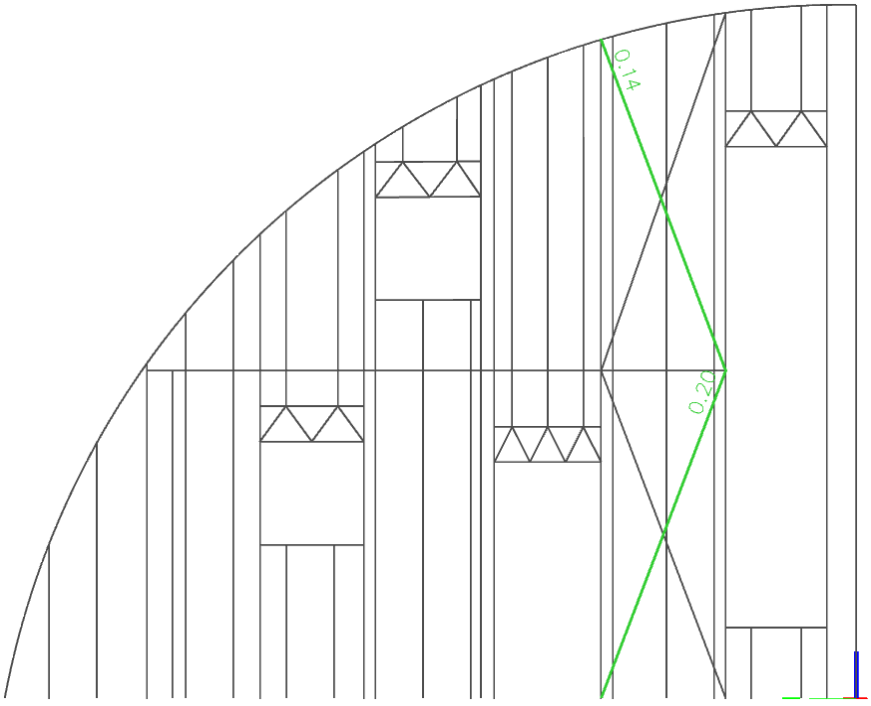
Nominal Tensile Strength
According to article D2 and formula (D2-1).

Table of values		
Tn	2178.0	lbf
Resistance factor	0.90	
Unity check	0.70	-

The member satisfies the check !

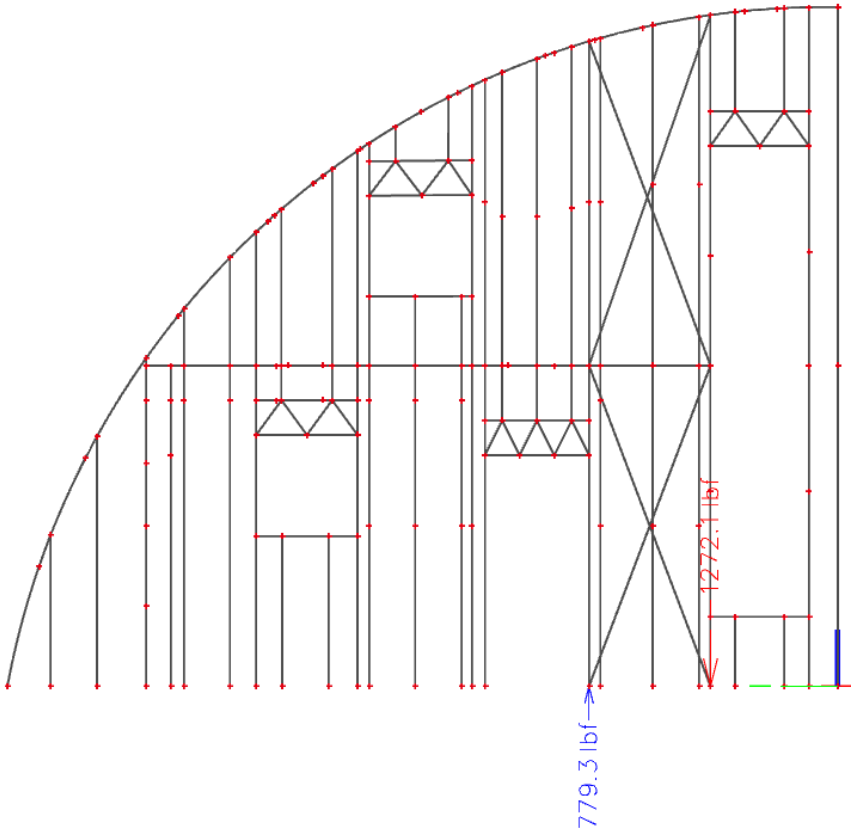
X-bracing Unity check





HOLDOWN DESIGN

Maximum support uplift reactions 1272 lb, from Combination LRFD-Ult (auto) 71 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy+(0.18)).



Accept holdown Simpson S/HDU4 series. Maximum tension load 6345 lb for steel 54 mil.

Model	H (in.)	Fasteners		Stud Member Thickness[2] mil (ga.)	ASD (lb.)		LRFD (lb.)		Nominal Tension Load[6] (lb.)
		Anchor Bolt Diameter[1] (in.)	Stud Fasteners[7]		Tension Load	Deflection at ASD Load[5]	Tension Load	Deflection at LRFD Load[5]	
S/HDU4	7⅞	⅝	(6) #14	2–33 (2–20)	2,320	0.093	3,705	0.149	5,685
				2–43 (2–18)	3,825	0.115	6,105	0.190	9,365
				2–54 (2–16)	3,970	0.093	6,345	0.156	9,730
				Steel fixture	4,470	0.063	7,165	0.103	12,120

HOLDOWN TO FLOOR BEAM BOLT CONNECTIONS CHECK

Tensile resistance check (AISC 360-16: J3-1), 5/8” bolts

$\phi R_n = \phi \cdot F_{nt} * A_b = 10.36$ kip

Where:

$A_b = 0.307$ in2 – gross bolt cross-sectional area A307 − 12 Table 3

$F_{nt} = 45.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$\phi = 0.75$ – capacity factor

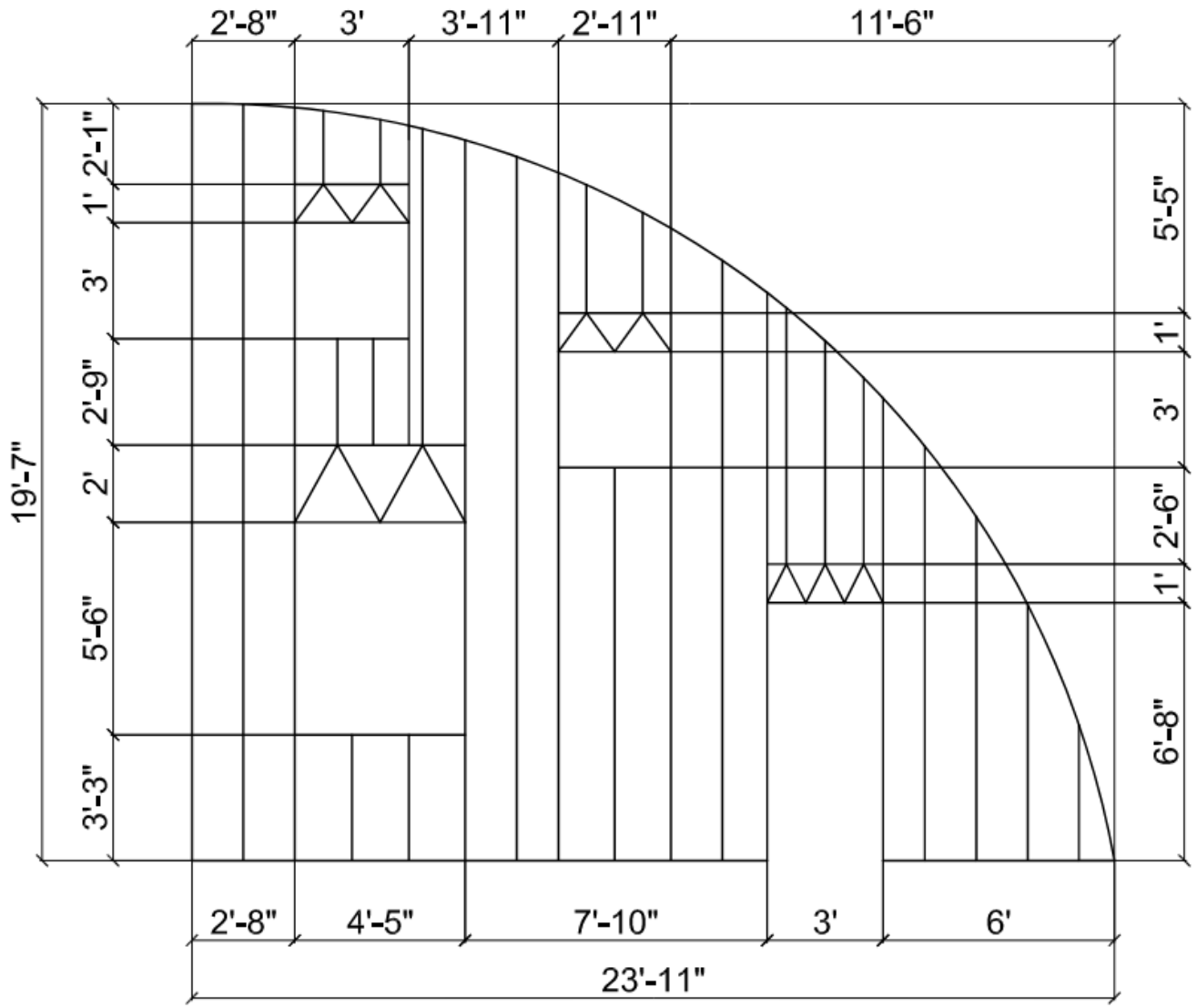
Maximum uplift support reactions - 1299lb = 1.3 kip.

1.3 kip < 10.36 kip

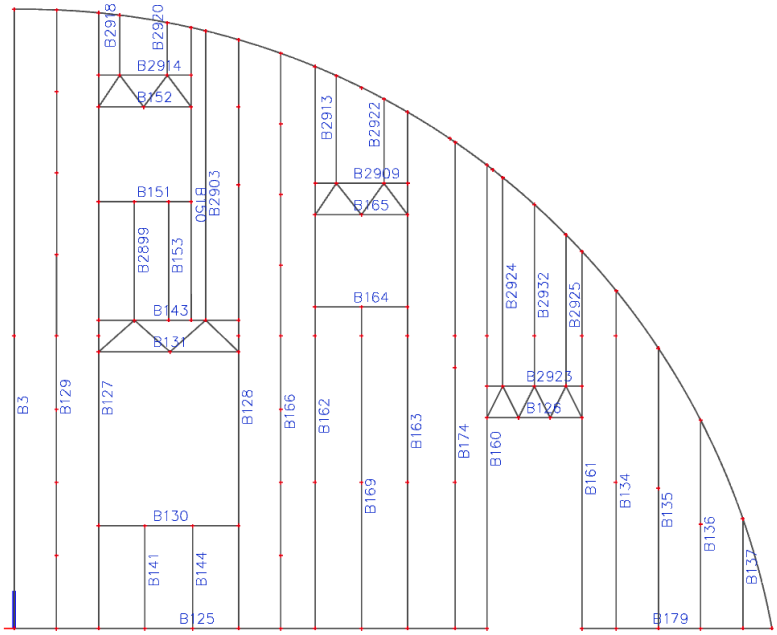
Connection is OK!

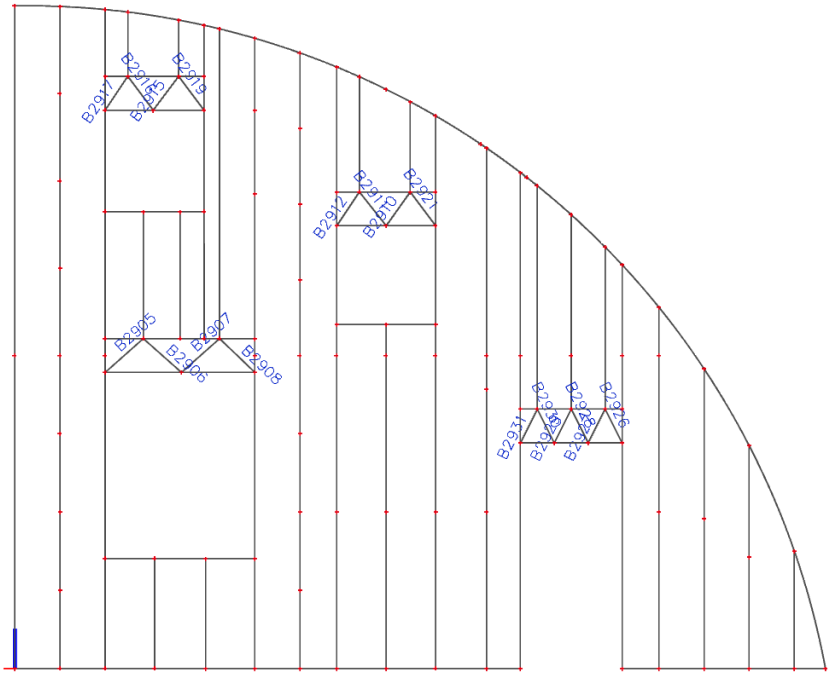
2.4.5 RIGHT WALL

Wall stud spacing - 16”;
First story wall stud height - 9’-3”;
Second story wall stud height - 10’-4”.

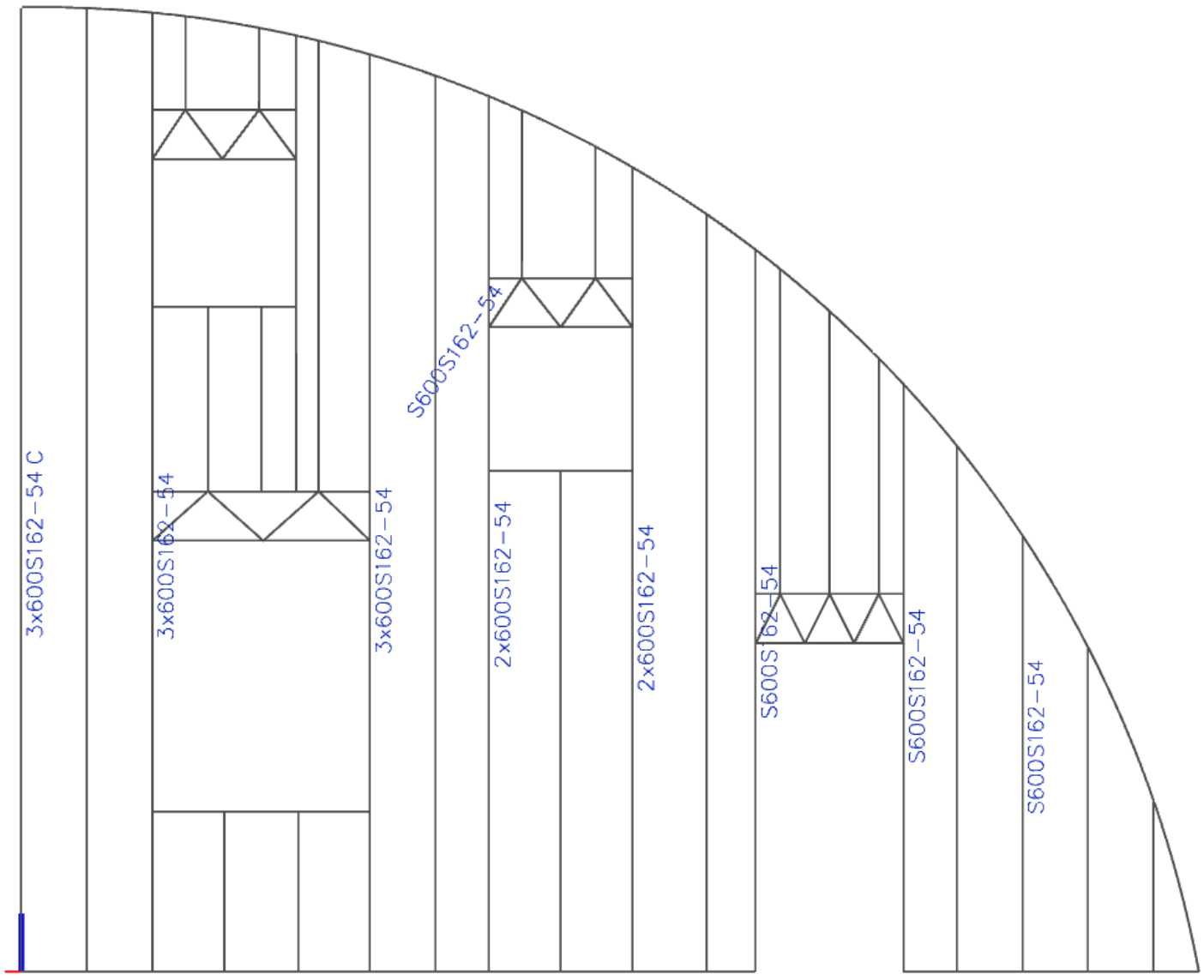


Numbers of members

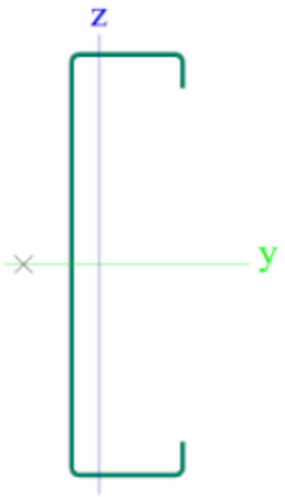




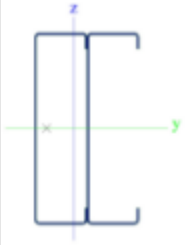
Cross-sections of members



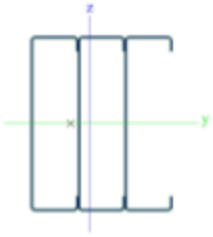
Cross-sections properties of 600S162-54

CS14		
Type	S600S162-54	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	0,559	
A_y [inch^2], A_z [inch^2]	0,186	0,346
A_L [inch^2/inch], A_D [inch^2/inch]	1,98e+01	1,98e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0,413	3,000
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	2,886	0,181
i_y [inch], i_z [inch]	2,272	0,568
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0,953	0,149
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	1,140	0,217
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	5,70e+01	5,70e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,08e+01	1,08e+01
d_y [inch], d_z [inch]	-1,056	0,000
I_t [inch^4], I_w [inch^6]	0,001	1,337
β_y [inch], β_z [inch]	0,000	6,508
Picture		

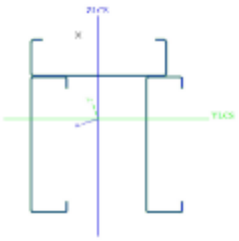
Cross-sections properties of 2x600S162-54

CS11		
Type	2x600S162-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,112	
A_y [inch^2], A_z [inch^2]	0,346	0,676
A_L [inch^2/inch], A_D [inch^2/inch]	2,33e+01	3,81e+01
$c_{y.ucs}$ [inch], $c_{z.ucs}$ [inch]	2,948	3,968
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	5,718	1,095
i_y [inch], i_z [inch]	2,267	0,992
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	1,906	0,541
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	2,280	0,904
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1,14e+02	1,14e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	4,52e+01	4,52e+01
d_y [inch], d_z [inch]	-0,860	0,000
I_t [inch^4], I_w [inch^6]	1,381	3,828
β_y [inch], β_z [inch]	0,000	3,169
Picture		

Cross-sections properties of 3x600S162-54

CS17		
Type	3x600S162-54	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,668	
A_y [inch^2], A_z [inch^2]	0,507	1,014
A_L [inch^2/inch], A_D [inch^2/inch]	2,69e+01	5,64e+01
$c_{y.ucs}$ [inch], $c_{z.ucs}$ [inch]	3,760	3,968
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	8,577	3,479
i_y [inch], i_z [inch]	2,267	1,444
$W_{el.y}$ [inch^3], $W_{el.z}$ [inch^3]	2,859	1,226
$W_{pl.y}$ [inch^3], $W_{pl.z}$ [inch^3]	3,420	2,024
$M_{pl.y.+}$ [kipinch], $M_{pl.y.-}$ [kipinch]	1,71e+02	1,71e+02
$M_{pl.z.+}$ [kipinch], $M_{pl.z.-}$ [kipinch]	1,01e+02	1,01e+02
d_y [inch], d_z [inch]	-0,676	0,000
I_t [inch^4], I_w [inch^6]	4,201	4,273
β_y [inch], β_z [inch]	0,000	2,036
Picture		

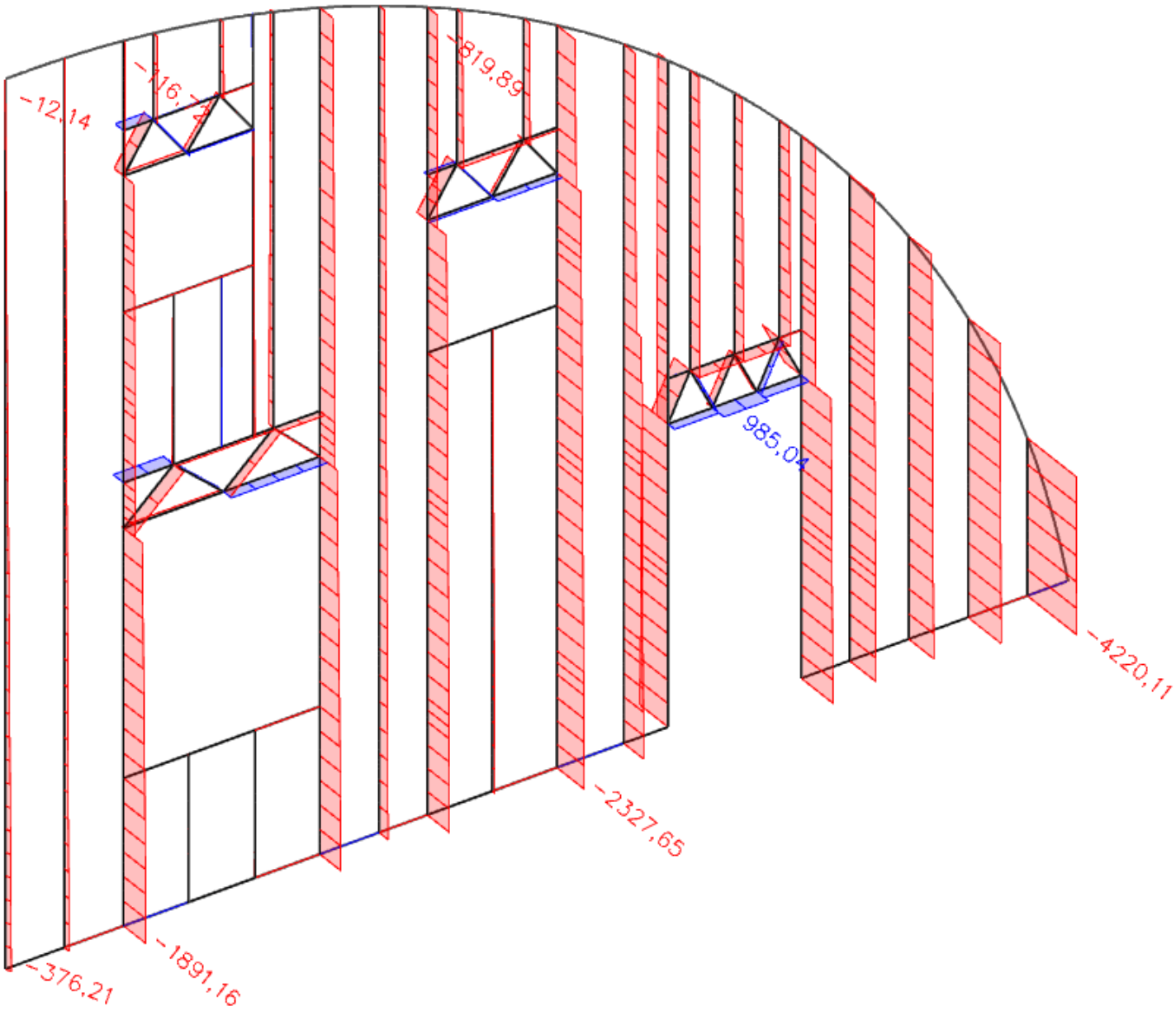
Cross-sections properties of 3x600S162-54 C

CS21		
Type	3x600S162-54 C	
Shape type	Thin-walled	
Item material	A913 grade 50	
Fabrication	cold formed	
Colour		
A [inch^2]	1,668	
A_y [inch^2], A_z [inch^2]	1,079	0,840
A_L [inch^2/inch], A_D [inch^2/inch]	5,53e+01	5,53e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	4,407	4,242
$I_{Y,LCS}$ [inch^4], $I_{Z,LCS}$ [inch^4]	10,218	10,372
$I_{YZ,LCS}$ [inch^4]	0,064	
α [deg]	110,01	
I_y [inch^4], I_z [inch^4]	10,396	10,195
i_y [inch], i_z [inch]	2,496	2,472
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	2,497	1,989
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	3,722	3,380
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,86e+02	1,86e+02
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,69e+02	1,69e+02
d_y [inch], d_z [inch]	3,751	-0,439
I_t [inch^4], I_w [inch^6]	0,002	77,572
β_y [inch], β_z [inch]	1,226	-8,898
Picture		

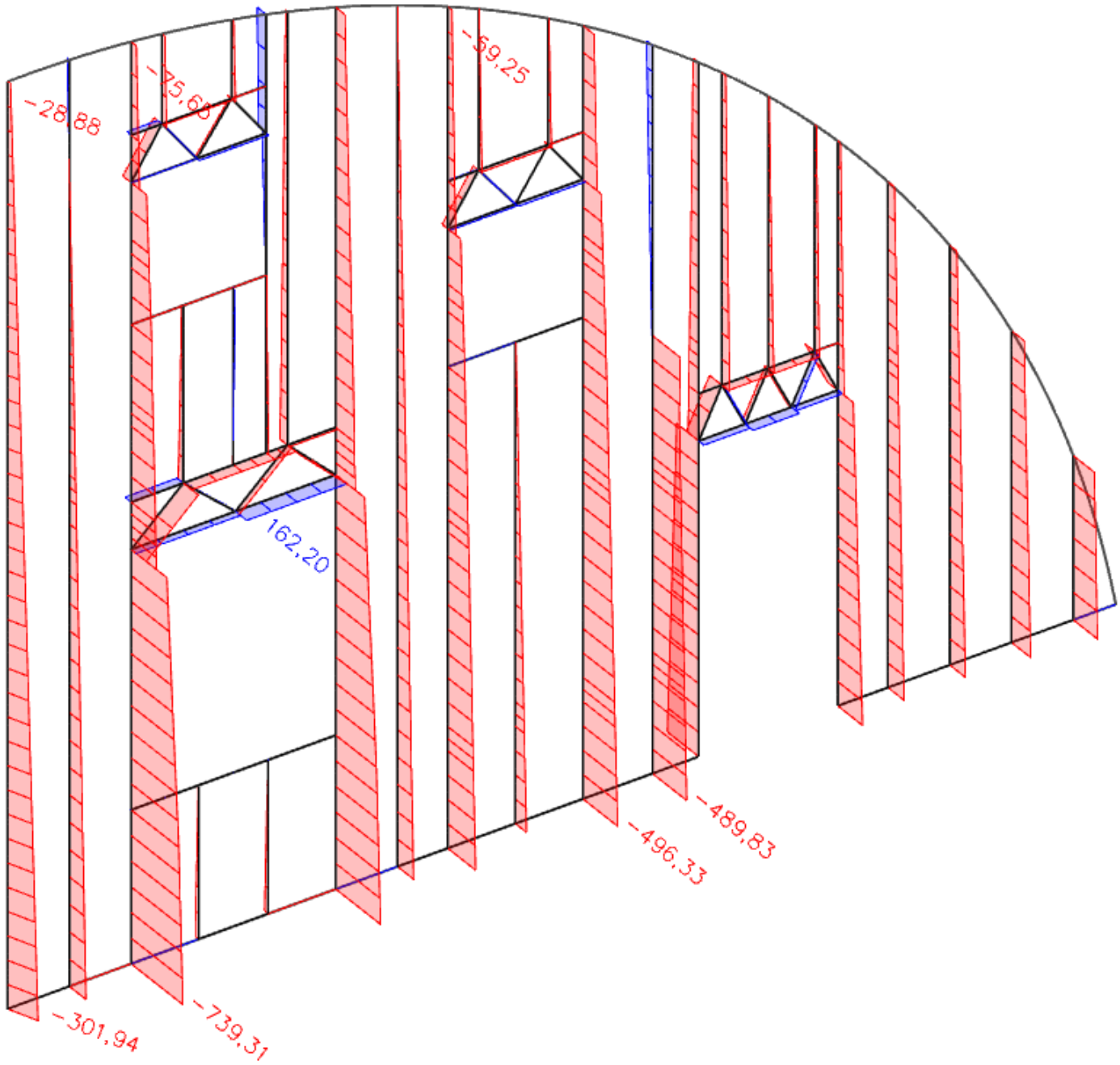
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Axial force diagram N,
LRFD-Ult (auto) 8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 0.5*L + 1.6*Lr), lbf.



Axial force diagram N,
LRFD-Ult (auto) 70 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-(-0.18)), lbf.



Shear force diagram Vz,
LRFD-Ult (auto) 70 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-(-0.18)), lbf.

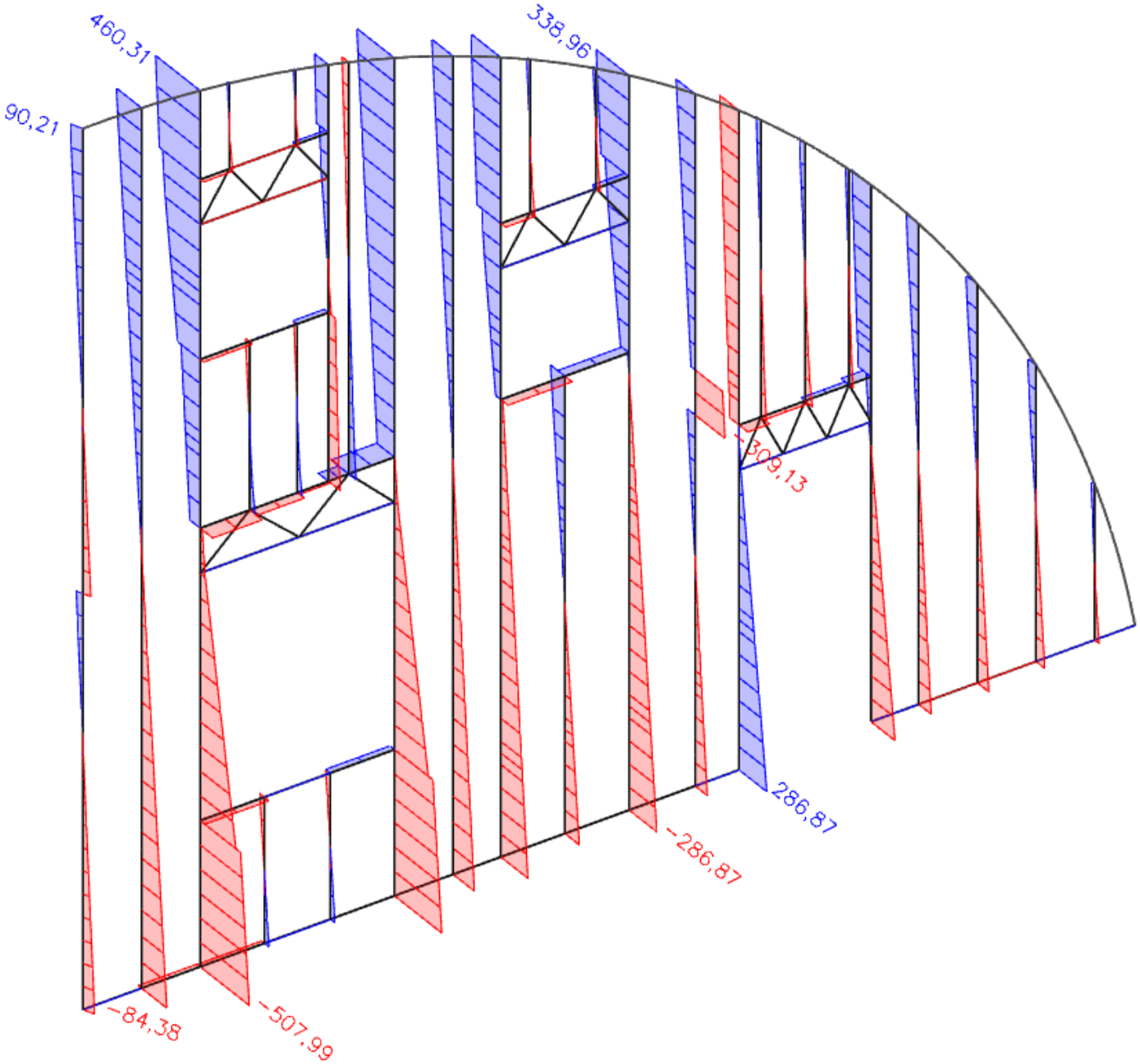
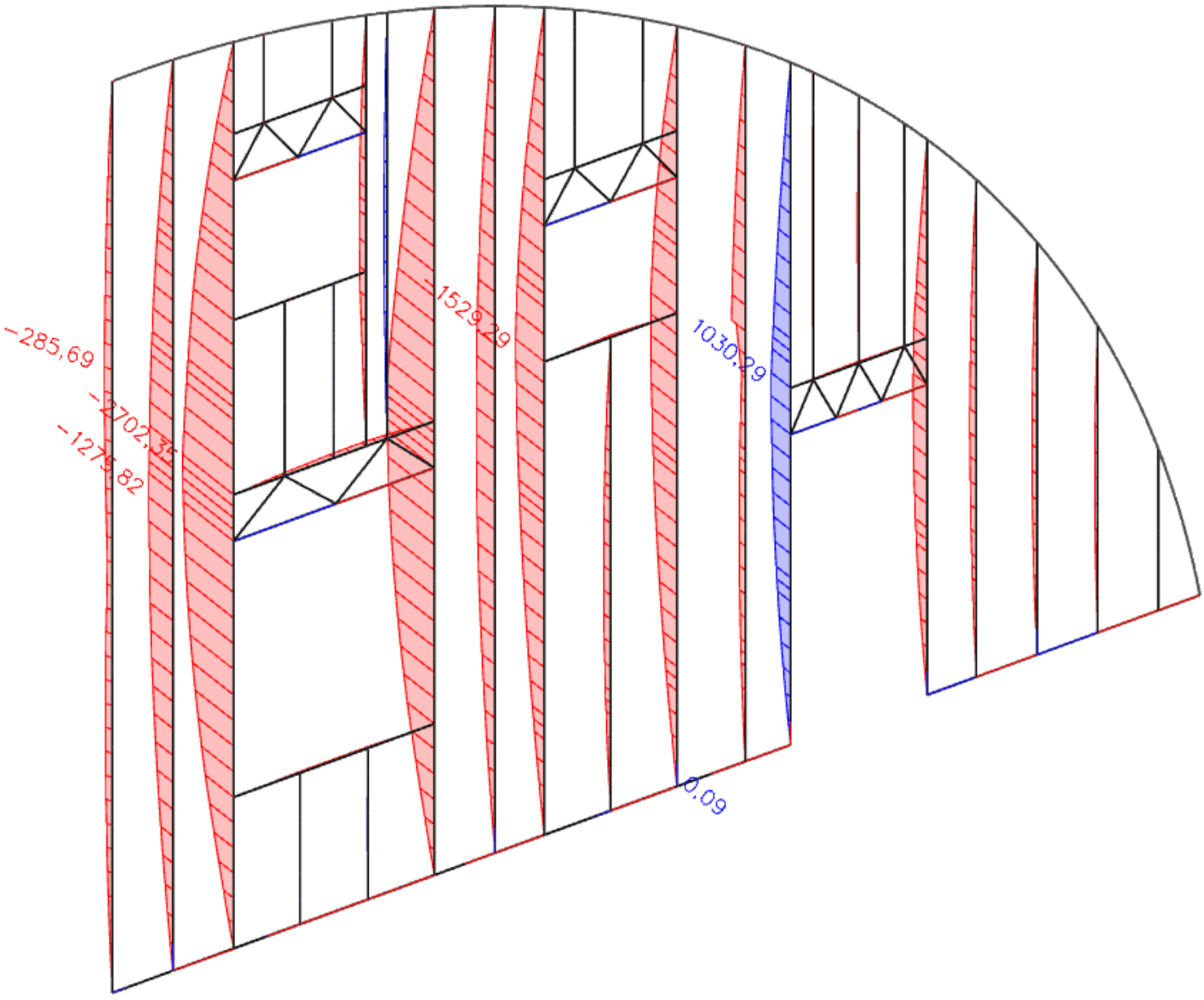
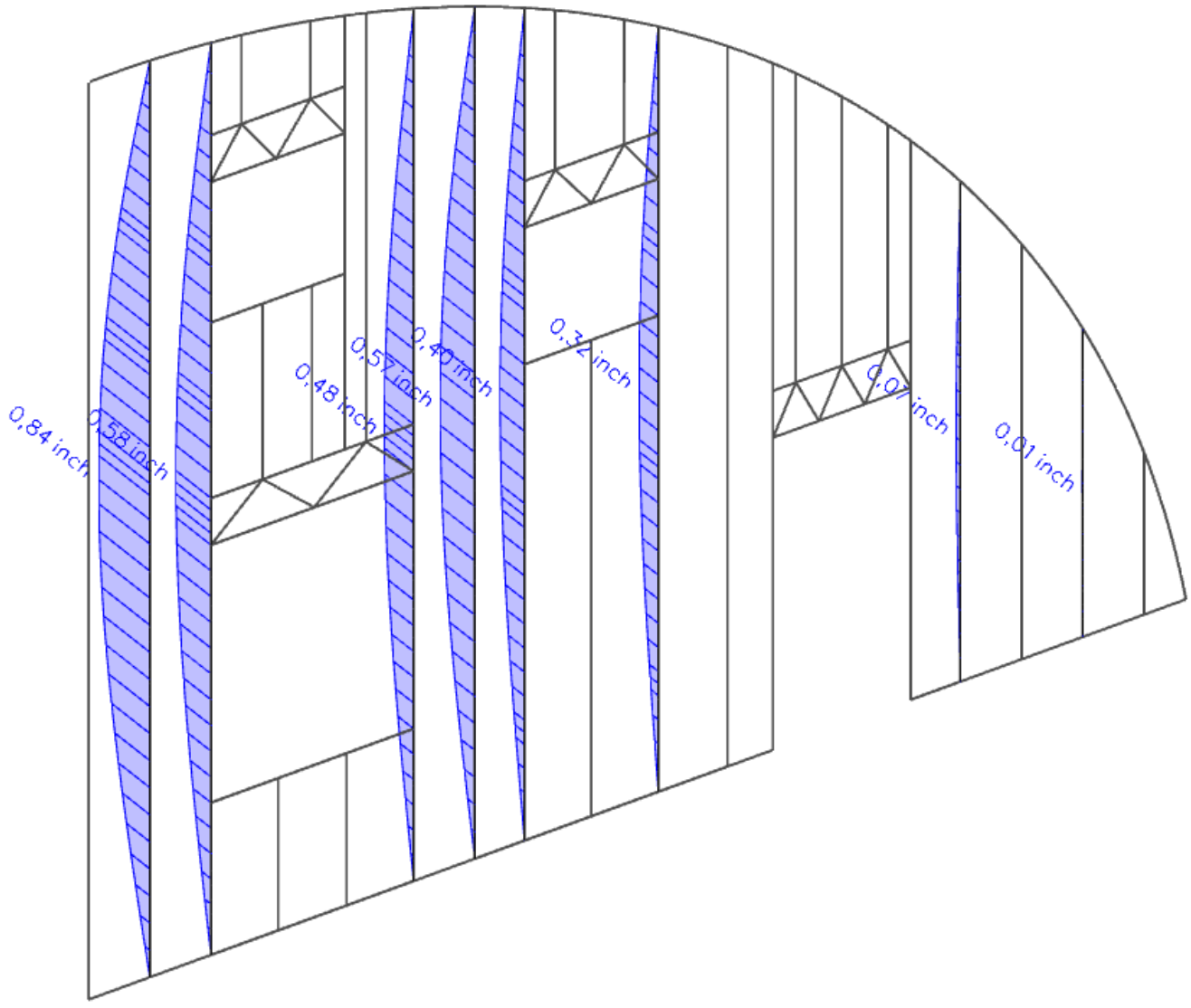


Diagram of bending moments My,
LRFD-Ult (auto) 70 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wx-(-0.18)), lbf*ft.



Wall stud deflection by Wind Load Wx- (-0.18), inch.



The maximum horizontal deflection of 0.84”. According to TABLE R301.7 the code IRC 2018, maximum wall member deflection should not exceed - H/240. H = stud height.

19’-7”=235”, 235”/240=0.979”. 0.84"< 0.979”. Deflection is OK!

STEEL MEMBER B128 CHECK

AISI S100-16 LRFD Check

Member B128	3x600S162-54	A913 grade 50	LRFD-Ult (auto)	0.33

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 8.75 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-274.19	lbf
Vux	-91.98	lbf
Vuy	29.29	lbf
Mut	23.00	lbfft
Mux	2165.93	lbfft
Muy	-465.09	lbfft

Combined Bending and Torsional Loading
According to article H4 and formula (H4-1)

Table of values		
Critical fibre	77	
Sigma Mx	8.965	ksi
Sigma My	5.958	ksi
f bending	14.923	ksi
Tau t	-0.004	ksi
f torsion	-0.004	ksi
Composed Stress	14.923	ksi
R	1.00	-

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
4	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
5	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
6	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-

7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
9	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
10	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
11	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-
12	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
13	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
14	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
15	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	2.859	$inch^3$
Mnxo	11912.61	lbfft
Resistance factor	0.90	
Unity check	0.20	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	5' 6.000"	ft
Sigma,ey	137.037	ksi
Kt	1.00	
Lt	5' 6.000"	ft
Sigma,t	3678.151	ksi
Cb	1.12	
Sfx	2.859	$inch^3$
Fcre	1290.683	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	14.313 14.313	1.00	4.000	6039.613	0.049	1.000	0.472 -	- -	-	- -	-
2	1.568	46.188 14.878	0.32	5.979	204.134	0.476	1.000	- 1.568	0.586 0.983	-	- -	-
3	5.943	46.188 46.188	1.00	4.000	9.511	2.204	0.408	2.428 -	- -	-	- -	-
4	1.568	46.188 14.878	0.32	5.979	204.134	0.476	1.000	- 1.568	0.586 0.983	-	- -	-
5	0.472	14.313 14.313	1.00	4.000	6039.613	0.049	1.000	0.472 -	- -	-	- -	-

6	0.472	-18.126 -18.126	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	13.748 -17.561	1.28	32.175	1098.573	0.112	1.000	- 1.568	0.367 0.784	-	- -	-
8	5.000	13.748 13.748	1.00	4.000	13.438	1.011	0.774	3.868 -	- -	-	- -	-
9	1.568	13.748 -17.561	1.28	32.175	1098.573	0.112	1.000	- 1.568	0.367 0.784	-	- -	-
10	0.472	-18.126 -18.126	-	-	-	-	-	- -	- -	-	- -	-
11	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
12	1.568	-18.691 -50.000	-	-	-	-	-	- -	- -	-	- -	-
13	5.000	-18.691 -18.691	-	-	-	-	-	- -	- -	-	- -	-
14	1.568	-18.691 -50.000	-	-	-	-	-	- -	- -	-	- -	-
15	0.472	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sye	1.034	$inch^3$
Mnyo	4308.89	lbfft
Resistance factor	0.90	
Unity check	0.12	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	29.469	ksi
Kt	1.00	
Lt	5' 6.000"	ft
Sigma,t	3678.151	ksi
Cb	1.00	
Sfy	1.707	$inch^3$
Fcre	892.142	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...

Shear Strength

According to article G2.1 and formula (G2.1.1)

Shear force Vx

Element ID	Aw [inch^2]	Vn [lbf]
1	0.000	0.00
2	0.089	2663.14
3	0.000	0.00
4	0.089	2663.14
5	0.000	0.00
6	0.000	0.00
7	0.089	2663.14
8	0.000	0.00
9	0.089	2663.14
10	0.000	0.00
11	0.000	0.00
12	0.089	2663.14
13	0.000	0.00
14	0.089	2663.14
15	0.000	0.00

Table of values		
Vn,x	15978.86	lbf
Resistance factor	0.95	
Unity check	0.01	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnyo	4308.89	lbfft
Vnx	15978.86	lbf
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.01+0.00) = 0.12

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 50.000	1.00	4.000	9.511	2.293	0.394	2.343 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-

5	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
11	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
12	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
13	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
14	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
15	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	1.194	$inch^2$
Pno	59677.33	lbf
Resistance factor	0.85	
Unity check	0.01	-

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	18 5/8	5 1/2	ft
Effective Length factor K	1.00	1.00	
Effective Length	18 5/8	5 1/2	ft
Slenderness	98.57	45.71	
Flexural Buckling stress Fcre	29.469	137.037	ksi

Torsional (-Flexural) Buckling Strength

According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	29.469	ksi
Sigma,ey	137.037	ksi
Kt	1.00	
Lt	5 1/2	ft
Sigma,t	3678.151	ksi
Sigma,TF	29.455	ksi
Torsional (-Flexural) buckling stress Fcre	29.455	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	24.570 24.570	1.00	4.000	6039.613	0.064	1.000	0.472 -	- -	-	- -	-
2	1.568	24.570 24.570	1.00	4.000	136.574	0.424	1.000	1.568 -	- -	-	- -	-
3	5.943	24.570 24.570	1.00	4.000	9.511	1.607	0.537	3.192 -	- -	-	- -	-
4	1.568	24.570 24.570	1.00	4.000	136.574	0.424	1.000	1.568 -	- -	-	- -	-
5	0.472	24.570 24.570	1.00	4.000	6039.613	0.064	1.000	0.472 -	- -	-	- -	-
6	0.472	24.570 24.570	1.00	4.000	6039.613	0.064	1.000	0.472 -	- -	-	- -	-
7	1.568	24.570 24.570	1.00	4.000	136.574	0.424	1.000	1.568 -	- -	-	- -	-
8	5.000	24.570 24.570	1.00	4.000	13.438	1.352	0.619	3.096 -	- -	-	- -	-
9	1.568	24.570 24.570	1.00	4.000	136.574	0.424	1.000	1.568 -	- -	-	- -	-
10	0.472	24.570 24.570	1.00	4.000	6039.613	0.064	1.000	0.472 -	- -	-	- -	-
11	0.472	24.570 24.570	1.00	0.430	162.315	0.389	1.000	0.472 -	- -	-	- -	-
12	1.568	24.570 24.570	1.00	4.000	136.574	0.424	1.000	1.568 -	- -	-	- -	-
13	5.000	24.570 24.570	1.00	4.000	13.438	1.352	0.619	3.096 -	- -	-	- -	-
14	1.568	24.570 24.570	1.00	4.000	136.574	0.424	1.000	1.568 -	- -	-	- -	-
15	0.472	24.570 24.570	1.00	0.430	162.315	0.389	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	29.455	ksi
lambda, c	1.30	
Fn	24.570	ksi
Ae	1.332	inch²
Pn	32728.97	lbf
Resistance factor	0.85	
Unity check	0.01	-

Combined Compressive Axial Load and Bending

According to article H1.2 and formulas (C5.2.1-3)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.472	0.164 0.164	1.00	4.000	6039.613	0.005	1.000	0.472 -	- -	-	- -	-
2	1.568	0.164 0.164	1.00	4.000	136.574	0.035	1.000	1.568 -	- -	-	- -	-
3	5.943	0.164 0.164	1.00	4.000	9.511	0.131	1.000	5.943 -	- -	-	- -	-
4	1.568	0.164 0.164	1.00	4.000	136.574	0.035	1.000	1.568 -	- -	-	- -	-
5	0.472	0.164	1.00	4.000	6039.613	0.005	1.000	0.472	-	-	-	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
		0.164						-	-		-	
6	0.472	0.164 0.164	1.00	4.000	6039.613	0.005	1.000	0.472 -	- -	-	- -	-
7	1.568	0.164 0.164	1.00	4.000	136.574	0.035	1.000	1.568 -	- -	-	- -	-
8	5.000	0.164 0.164	1.00	4.000	13.438	0.111	1.000	5.000 -	- -	-	- -	-
9	1.568	0.164 0.164	1.00	4.000	136.574	0.035	1.000	1.568 -	- -	-	- -	-
10	0.472	0.164 0.164	1.00	4.000	6039.613	0.005	1.000	0.472 -	- -	-	- -	-
11	0.472	0.164 0.164	1.00	0.430	162.315	0.032	1.000	0.472 -	- -	-	- -	-
12	1.568	0.164 0.164	1.00	4.000	136.574	0.035	1.000	1.568 -	- -	-	- -	-
13	5.000	0.164 0.164	1.00	4.000	13.438	0.111	1.000	5.000 -	- -	-	- -	-
14	1.568	0.164 0.164	1.00	4.000	136.574	0.035	1.000	1.568 -	- -	-	- -	-
15	0.472	0.164 0.164	1.00	0.430	162.315	0.032	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	11912.61	lbfft
Mny	4308.89	lbfft
Pn	32728.97	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.01+0.20+0.12 = 0.33 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B161 CHECK

AISI S100-16 LRFD Check

Member B161	S600S162-54	A913 grade 50	LRFD-Ult (auto)	0.62

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 6.67 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-2285.61	lbf
Vux	3.12	lbf
Vuy	12.24	lbf
Mut	0.00	lbfft
Mux	-372.03	lbfft
Muy	20.82	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	48.118 42.159	0.88	0.475	310.586	0.394	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
5	5.717	48.118 -46.914	0.97	23.357	60.020	0.895	0.842	- 4.816	1.212 2.408	-	- -	-
7	1.342	-48.795 -48.795	-	-	-	-	-	- -	- -	-	- -	-
9	0.359	-40.954 -46.914	-	-	-	-	-	- -	- -	-	- -	-

Table of values		
Sxe	0.925	inch^3
Mnxo	3854.26	lbfft
Resistance factor	0.90	
Unity check	0.11	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	6' 8.000"	ft
Sigma,ey	315991.360	ksi
Kt	1.00	
Lt	6' 8.000"	ft
Sigma,t	18.041	ksi
Cb	1.25	
Sfx	0.962	$inch^3$
Fcre	4454.981	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
Distortional Buckling Strength
According to article F4 and formula F4.1-2.

Table of values		
Sfy	0.962	$inch^3$
My	4008.12	lbfft
L	-0.541"	ft
Beta	1.30	
k,phi,fe	87949800.67	lbf
k,phi,we	524430.69	lbf
k,phi	0.00	lbf
k,phi,fg	4.774	$inch^2$
k,phi,wg	1.511	$inch^2$
Fd	18300.723	ksi

Table of values		
Sf	0.962	$inch^3$
Mcrd	1467030.15	lbfft
Lambda,d	0.05	
Cyd	3.00	
Mp	4750.35	lbfft
cyt	3.00	
Myc	4008.12	lbfft
Myt3	0.00	lbfft
Mn	4667.88	lbfft
Resistance factor	0.90	
Unity check	0.09	-

Data		
Lm	-0.541"	ft
Lcr	1' 1.314"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	-0.052	inch
Ksi,web	2.00	

Number of compressed flanges: 1
Critical flange contains Initial shape parts: 2, 3, 1
...::Flexural Strength about Y-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.359 -	- -	-	- -	-
3	1.342	45.216 -11.497	0.25	10.455	487.570	0.305	1.000	- 1.342	0.412 0.671	-	- -	-
5	5.717	-16.281 -16.281	-	-	-	-	-	- -	- -	-	- -	-
7	1.342	45.216 -11.497	0.25	10.455	487.570	0.305	1.000	- 1.342	0.412 0.671	-	- -	-
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.359 -	- -	-	- -	-

Table of values		
Sye	0.149	inch^3
Mnyo	620.93	lbfft
Resistance factor	0.90	
Unity check	0.04	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.2-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.359	48.253 48.253	1.00	0.430	281.003	0.414	1.000	0.359 -	- -	-	- -	-
3	1.342	43.636 -11.095	0.25	10.455	487.570	0.299	1.000	- 1.342	0.412 0.671	-	- -	-
5	5.717	-15.712 -15.712	-	-	-	-	-	- -	- -	-	- -	-
7	1.342	43.636 -11.095	0.25	10.455	487.570	0.299	1.000	- 1.342	0.412 0.671	-	- -	-
9	0.359	48.253 48.253	1.00	0.430	281.003	0.414	1.000	0.359 -	- -	-	- -	-

Table of values		
Sigma,ex	72.096	ksi
Kt	1.00	
Lt	6' 8.000"	ft
Sigma,t	18.041	ksi
Cs	-1.00	
CTF	0.60	
Sfy	0.149	inch^3
j	3.407	inch
Fcre	105.658	ksi

Table of values		
Fc	48.253	ksi
Scy	0.149	$inch^3$
Mny	599.23	lbfft
Resistance factor	0.90	
Unity check	0.04	-

Distortional Buckling Strength
According to article F4 and formula F4.1-2.

Table of values		
Sfy	0.149	$inch^3$
My	620.87	lbfft
L	1' 1.314"	ft
Beta	1.00	
k,phi,fe	311.83	lbf
k,phi,we	293.13	lbf
k,phi	0.00	lbf
k,phi,fg	0.008	$inch^2$
k,phi,wg	0.011	$inch^2$
Fd	31.356	ksi
Sf	0.149	$inch^3$
Mcrd	389.36	lbfft
Lambda,d	1.26	
Mn	406.02	lbfft
Resistance factor	0.90	
Unity check	0.06	-

Data		
Lm	1614' 4.442"	ft
Lcr	1' 1.314"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch
Ksi,web	0.00	

Number of compressed flanges: 2
Critical flange contains Initial shape parts: 8,

...::Axial Compression Strength::...
Nominal Axial Strength
According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-
3	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222

5	5.717	50.000 50.000	1.00	4.000	10.279	2.206	0.408	2.334 -	- -	-	- -	-
7	1.342	50.000 50.000	1.00	2.882	134.415	0.610	1.000	1.342 -	0.415 0.927	30.83	0.000 0.000	0.222
9	0.359	50.000 50.000	1.00	0.430	281.003	0.422	1.000	0.222 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.349	$inch^2$
Pno	17465.16	lbf
Resistance factor	0.85	
Unity check	0.15	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	12	6 3/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	12	6 3/4	ft
Slenderness	63.02	140.78	
Flexural Buckling stress Fcre	72.096	14.445	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	72.096	ksi
Sigma,ey	14.445	ksi
Kt	1.00	
Lt	6 3/4	ft
Sigma,t	18.041	ksi
Sigma,TF	14.445	ksi
Torsional (-Flexural) buckling stress Fcre	14.445	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.359	12.668 12.668	1.00	0.430	281.003	0.212	1.000	0.359 -	- -	-	- -	-
3	1.342	12.668 12.668	1.00	3.387	157.959	0.283	1.000	1.342 -	0.671 0.671	61.25	0.000 0.000	0.359
5	5.717	12.668 12.668	1.00	4.000	10.279	1.110	0.722	4.129 -	- -	-	- -	-
7	1.342	12.668 12.668	1.00	3.387	157.959	0.283	1.000	1.342 -	0.671 0.671	61.25	0.000 0.000	0.359
9	0.359	12.668 12.668	1.00	0.430	281.003	0.212	1.000	0.359 -	- -	-	- -	-

Table of values		
Fe	14.445	ksi
lambda, c	1.86	
Fn	12.668	ksi
Ae	0.466	$inch^2$
Pn	5908.71	lbf
Resistance factor	0.85	
Unity check	0.46	-

Distortional Buckling Strength
According to article E4 and formula (E4.1-2).

Table of values		
Py	27953.53	lbf
L	1' 2.713"	ft
k,phi,fe	219.79	lbf
k,phi,we	160.55	lbf
k,phi	0.00	lbf
k,phi,fg	0.006	$inch^2$
k,phi,wg	0.009	$inch^2$
Fd	24.162	ksi
Pcrd	13508.21	lbf
Lambda,d	1.44	
Pn	15149.05	lbf
Resistance factor	0.85	
Unity check	0.18	-

Data		
Lm	1614' 4.442"	ft
Lcr	1' 2.713"	ft
h0	6.000	inch
Ixf	0.002	$inch^4$
Iyf	0.024	$inch^4$
Ixyf	-0.004	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.550	inch
hxf	-1.004	inch
Af	0.106	$inch^2$
y0f	0.052	inch

Number of compressed flanges: 2
Critical flange contains Initial shape parts: 8, 7, 9

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (H1.2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.359	4.088 4.088	1.00	0.430	281.003	0.121	1.000	0.359 -	- -	-	- -	-
3	1.342	4.088 4.088	1.00	4.000	186.542	0.148	1.000	1.342 -	0.671 0.671	107.82	- 0.000	0.359
5	5.717	4.088 4.088	1.00	4.000	10.279	0.631	1.000	5.717 -	- -	-	- -	-
7	1.342	4.088 4.088	1.00	4.000	186.542	0.148	1.000	1.342 -	0.671 0.671	107.82	- 0.000	0.359
9	0.359	4.088 4.088	1.00	0.430	281.003	0.121	1.000	0.359 -	- -	-	- -	-

Table of values		
Mnx	3854.26	lbfft
Mny	406.02	lbfft
PEx	40306.61	lbf
PEy	8075.85	lbf
Alfa x	0.94	
Alfa y	0.72	
Cmx	0.85	
Cmy	0.85	
Pn	5908.71	lbf
Pno	17465.16	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.46+0.10+0.07 = 0.62 - (H1.2-1)
Unity check = 0.15+0.11+0.06 = 0.32 -

The member satisfies the check !

STEEL MEMBER B162 CHECK

AISI S100-16 LRFD Check

Member B162	2x600S162-54	A913 grade 50	LRFD-Ult (auto)	0.26

Material data		
Yield stress Fy	50.00	ksi
Tensile stress Fu	65.00	ksi
fabrication	cold formed	

The critical check is on position 10.17 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-739.41	lbf
Vux	10.76	lbf
Vuy	52.92	lbf
Mut	-8.14	lbfft
Mux	-1529.29	lbfft
Muy	18.14	lbfft

Combined Bending and Torsional Loading
According to article H4 and formula (H4-1)

Table of values		
Critical fibre	46	
Sigma Mx	-9.628	ksi
Sigma My	-0.374	ksi
f bending	-10.002	ksi
Tau t	0.004	ksi
f torsion	0.004	ksi
Composed Stress	10.002	ksi
R	1.00	-

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
2	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
3	5.943	50.000 -50.000	1.00	24.000	57.064	0.936	0.817	- 4.857	1.214 2.429	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-

5	0.472	50.000 42.063	0.84	4.325	6531.026	0.087	1.000	- 0.472	0.219 0.253	-	- -	-
6	0.472	-42.063 -50.000	-	-	-	-	-	- -	- -	-	- -	-
7	1.568	-50.000 -50.000	-	-	-	-	-	- -	- -	-	- -	-
8	5.000	42.063 -42.063	1.00	24.000	80.629	0.722	0.963	- 4.814	1.203 2.407	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 42.063	0.84	0.489	184.700	0.520	1.000	0.472 -	- -	-	- -	-

Table of values		
Sxe	1.906	$inch^3$
Mnxo	7941.74	lbfft
Resistance factor	0.90	
Unity check	0.21	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	2' 11.000"	ft
Sigma,ey	230.151	ksi
Kt	1.00	
Lt	2' 11.000"	ft
Sigma,t	2134.839	ksi
Cb	1.07	
Sfx	1.906	$inch^3$
Fcre	1147.927	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Flexural Strength about Y-axis::...

Nominal Flexural Strength

According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	10.164 10.164	1.00	4.000	6039.613	0.041	1.000	0.472 -	- -	-	- -	-
2	1.568	9.458 -29.672	3.14	153.900	5254.683	0.042	1.000	- 1.568	0.256 0.784	-	- -	-
3	5.943	-29.672 -29.672	-	-	-	-	-	- -	- -	-	- -	-
4	1.568	9.458 -29.672	3.14	153.900	5254.683	0.042	1.000	- 1.568	0.256 0.784	-	- -	-
5	0.472	10.164 10.164	1.00	4.000	6039.613	0.041	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 10.870	0.22	6.524	222.745	0.474	1.000	- 1.568	0.564 1.005	-	- -	-

8	5.000	10.870 10.870	1.00	4.000	13.438	0.899	0.840	4.199 -	- -	-	- -	-
9	1.568	50.000 10.870	0.22	6.524	222.745	0.474	1.000	- 1.568	0.564 1.005	-	- -	-
10	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Sye	0.539	$inch^3$
Mnyo	2244.50	lbfft
Resistance factor	0.90	
Unity check	0.01	-

Lateral-Torsional Buckling Strength

According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	32.352	ksi
Kt	1.00	
Lt	2' 11.000"	ft
Sigma,t	2134.839	ksi
Cb	1.00	
Sfy	0.541	$inch^3$
Fcre	1415.273	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
2	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
3	5.943	50.000 50.000	1.00	4.000	9.511	2.293	0.394	2.343 -	- -	-	- -	-
4	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
5	0.472	50.000 50.000	1.00	4.000	6039.613	0.091	1.000	0.472 -	- -	-	- -	-
6	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-
7	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
8	5.000	50.000 50.000	1.00	4.000	13.438	1.929	0.459	2.296 -	- -	-	- -	-
9	1.568	50.000 50.000	1.00	4.000	136.574	0.605	1.000	1.568 -	- -	-	- -	-
10	0.472	50.000 50.000	1.00	0.430	162.315	0.555	1.000	0.472 -	- -	-	- -	-

Table of values		
Fn	50.000	ksi
Ae	0.779	$inch^2$
Pno	38928.07	lbf
Resistance factor	0.85	
Unity check	0.02	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	17 7/8	3	ft
Effective Length factor K	1.00	1.00	
Effective Length	17 7/8	3	ft
Slenderness	94.07	35.27	
Flexural Buckling stress Fcre	32.352	230.151	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	32.352	ksi
Sigma,ey	230.151	ksi
Kt	1.00	
Lt	3	ft
Sigma,t	2134.839	ksi
Sigma,TF	32.298	ksi
Torsional (-Flexural) buckling stress Fcre	32.298	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [$inch^4$]	ds [inch]
1	0.472	26.156 26.156	1.00	4.000	6039.613	0.066	1.000	0.472 -	- -	-	- -	-
2	1.568	26.156 26.156	1.00	4.000	136.574	0.438	1.000	1.568 -	- -	-	- -	-
3	5.943	26.156 26.156	1.00	4.000	9.511	1.658	0.523	3.108 -	- -	-	- -	-
4	1.568	26.156 26.156	1.00	4.000	136.574	0.438	1.000	1.568 -	- -	-	- -	-
5	0.472	26.156 26.156	1.00	4.000	6039.613	0.066	1.000	0.472 -	- -	-	- -	-
6	0.472	26.156 26.156	1.00	0.430	162.315	0.401	1.000	0.472 -	- -	-	- -	-
7	1.568	26.156 26.156	1.00	4.000	136.574	0.438	1.000	1.568 -	- -	-	- -	-
8	5.000	26.156 26.156	1.00	4.000	13.438	1.395	0.604	3.019 -	- -	-	- -	-
9	1.568	26.156 26.156	1.00	4.000	136.574	0.438	1.000	1.568 -	- -	-	- -	-
10	0.472	26.156 26.156	1.00	0.430	162.315	0.401	1.000	0.472 -	- -	-	- -	-

Table of values		
Fe	32.298	ksi
lambda, c	1.24	
Fn	26.156	ksi
Ae	0.863	inch^2
Pn	22565.84	lbf
Resistance factor	0.85	
Unity check	0.04	-

Combined Compressive Axial Load and Bending

According to article H1.2 and formulas (C5.2.1-3)

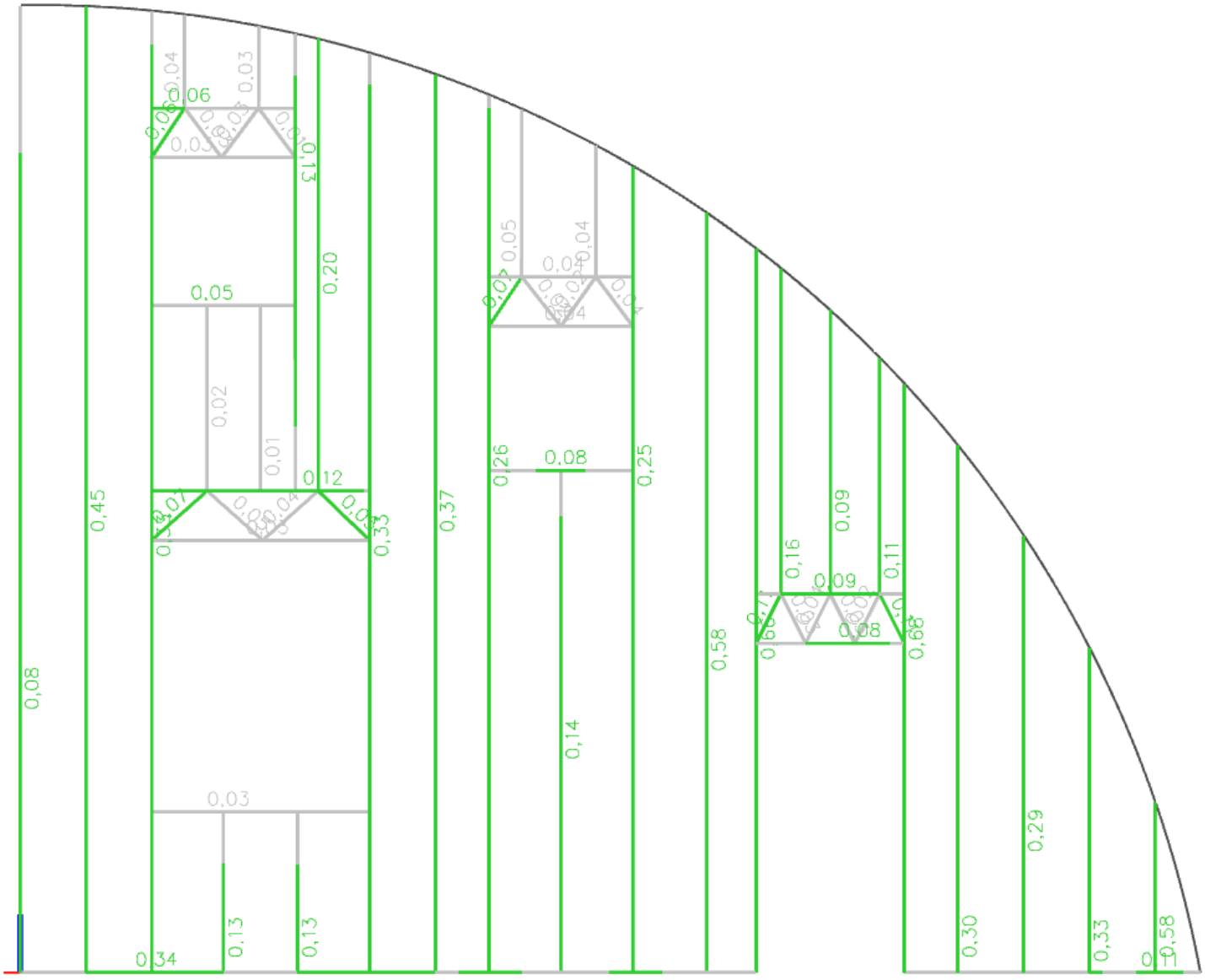
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.472	0.665 0.665	1.00	4.000	6039.613	0.010	1.000	0.472 -	- -	-	- -	-
2	1.568	0.665 0.665	1.00	4.000	136.574	0.070	1.000	1.568 -	- -	-	- -	-
3	5.943	0.665 0.665	1.00	4.000	9.511	0.264	1.000	5.943 -	- -	-	- -	-
4	1.568	0.665 0.665	1.00	4.000	136.574	0.070	1.000	1.568 -	- -	-	- -	-
5	0.472	0.665 0.665	1.00	4.000	6039.613	0.010	1.000	0.472 -	- -	-	- -	-
6	0.472	0.665 0.665	1.00	0.430	162.315	0.064	1.000	0.472 -	- -	-	- -	-
7	1.568	0.665 0.665	1.00	4.000	136.574	0.070	1.000	1.568 -	- -	-	- -	-
8	5.000	0.665 0.665	1.00	4.000	13.438	0.222	1.000	5.000 -	- -	-	- -	-
9	1.568	0.665 0.665	1.00	4.000	136.574	0.070	1.000	1.568 -	- -	-	- -	-
10	0.472	0.665 0.665	1.00	0.430	162.315	0.064	1.000	0.472 -	- -	-	- -	-

Table of values		
Mnx	7941.74	lbfft
Mny	2244.50	lbfft
Pn	22565.84	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.04+0.21+0.01 = 0.26 - (C5.2.1-3)

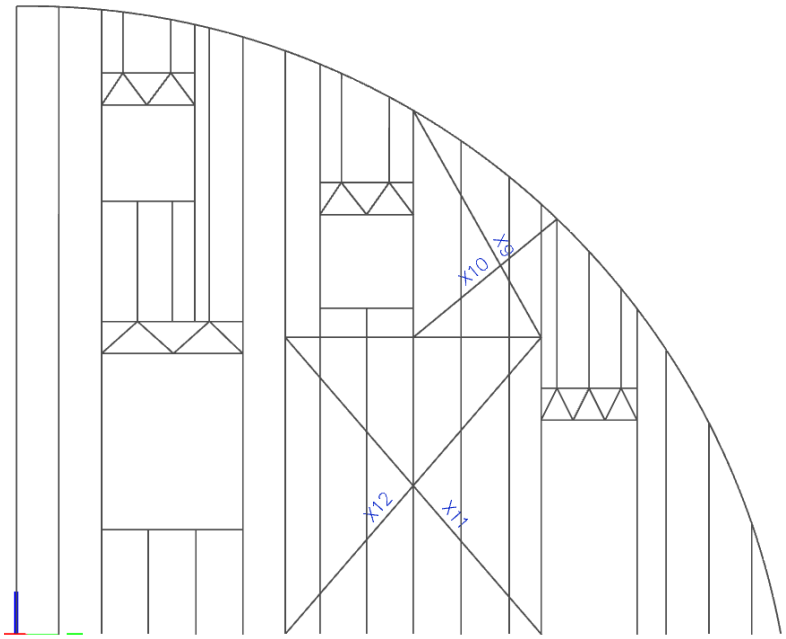
The member satisfies the check !

Unity Check

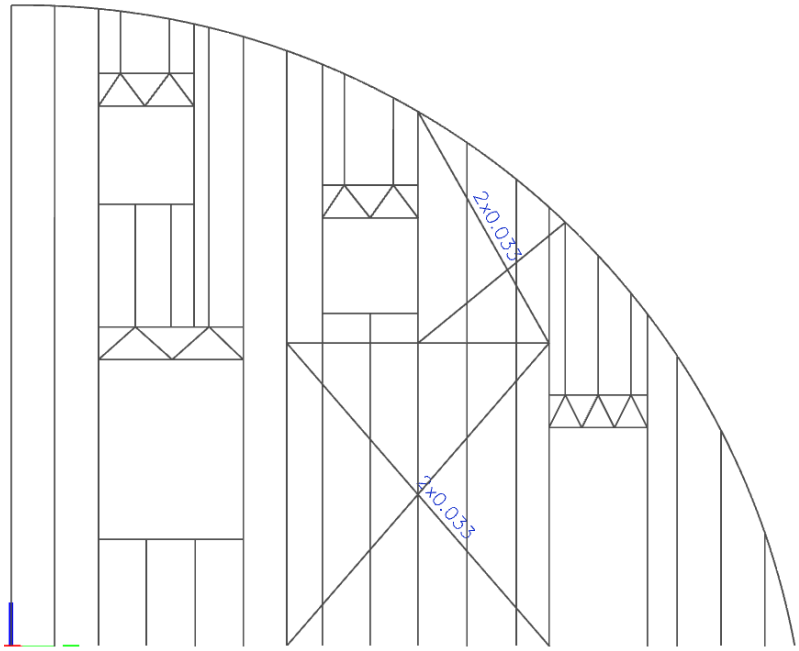

0,08
0,45
0,34
0,13
0,13
0,03
0,33
0,37
0,26
0,08
0,25
0,14
0,58
0,16
0,09
0,09
0,11
0,08
0,68
0,30
0,29
0,33
0,58
0,05
0,20
0,12
0,06
0,13

X-BRACING STRAP CHECK

Numbers of members



Cross-sections of members



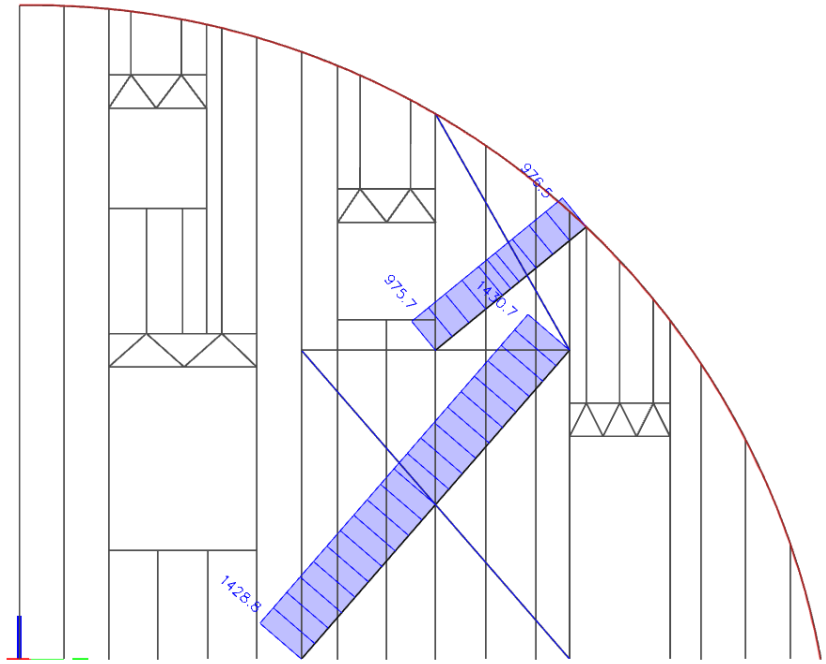
Cross-sections properties of X-BRACING STRAP 2”x0.033”

CS26		
Type	2x0.033	
Shape type	Thin-walled	
Item material	A653 grade 33	
Fabrication	cold formed	
Colour		
A [inch^2]	0.066	
A_y [inch^2], A_z [inch^2]	0.055	0.055
A_L [inch^2/inch], A_D [inch^2/inch]	4.07e+00	4.07e+00
$c_{y,UCS}$ [inch], $c_{z,UCS}$ [inch]	0.000	1.000
α [deg]	0.00	
I_y [inch^4], I_z [inch^4]	0.022	0.000
i_y [inch], i_z [inch]	0.577	0.010
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0.022	0.000
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	0.033	0.001
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1.09e+00	1.09e+00
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1.80e-02	1.80e-02
d_y [inch], d_z [inch]	0.000	0.000
I_t [inch^4], I_w [inch^6]	0.000	0.000
β_y [inch], β_z [inch]	0.000	0.000
Picture		

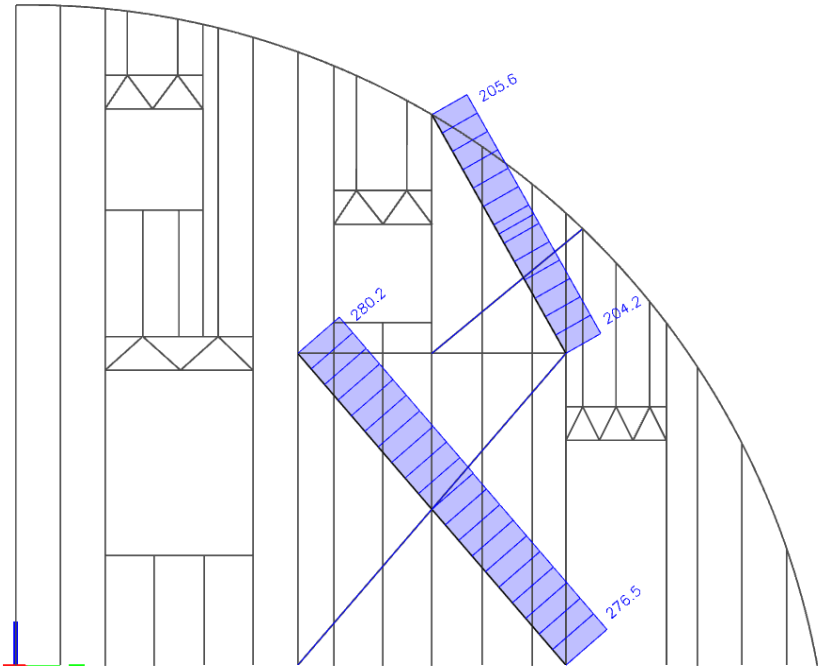
Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

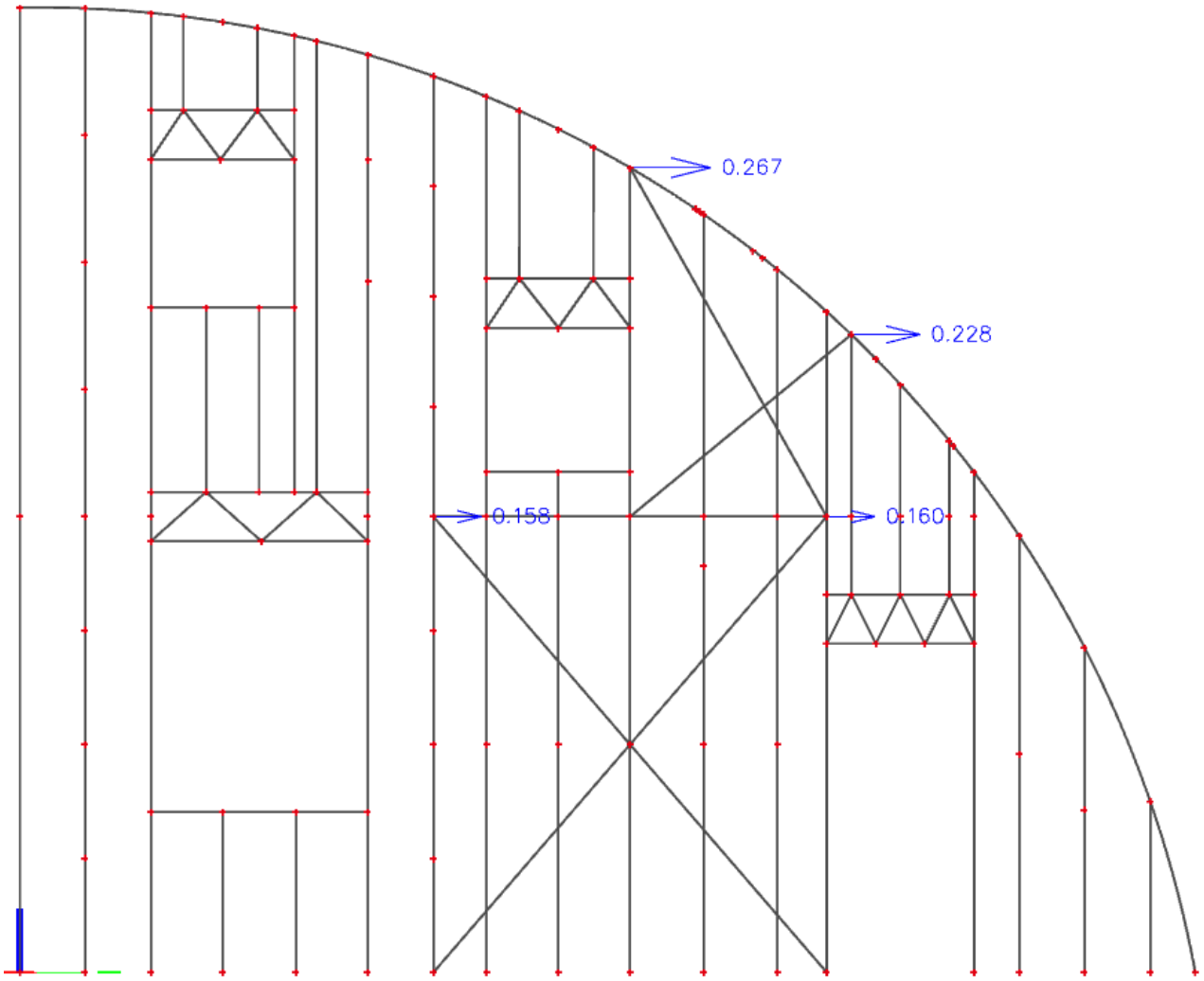
Maximum axial force N,
LRFD-Ult (auto) 71 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy+(0.18)), lbf.



LRFD-Ult (auto) 73 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy-(0.18)), lbf.



Wall Wind load deflection
Value: Uy - Wy+(-0.18), (inch).



The maximum deflection is 0.267" according to Table R301.7 the code IRC-2019 - the deflection limits H/360. L = 19'-4"= 19'*12"+4=232" 232/360 = 0.644"

0.267" < 0.644"

Deflection is OK!

STEEL MEMBER X8 CHECK

AISI S100-16 LRFD Check

Member X12	2x0.033	A653 grade 33	NC_LRFD-Ult (au	0.73

Material data		
Yield stress Fy	33.00	ksi
Tensile stress Fu	45.00	ksi
fabrication	cold formed	

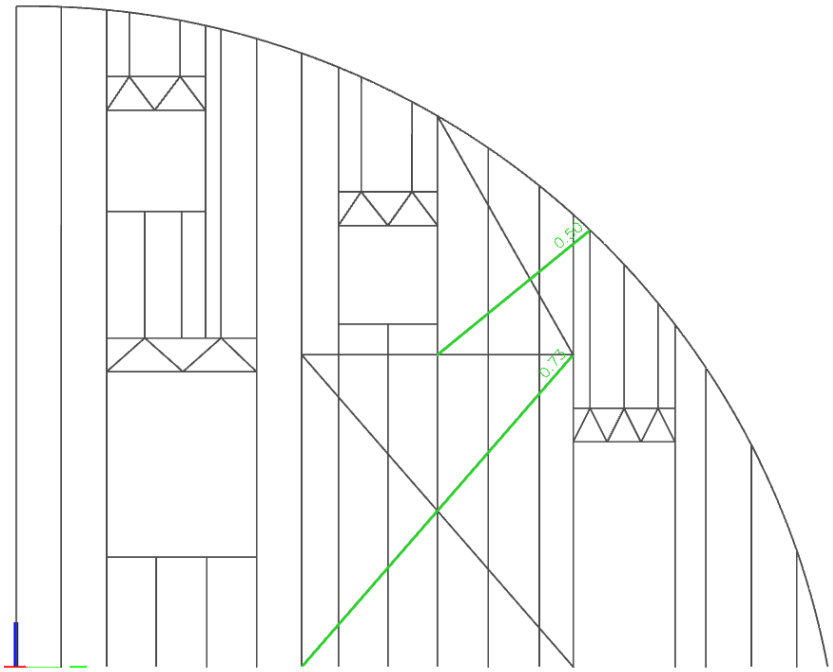
Warning: Part 1 exceeds dimensional limit w/t <= 60! (art. B1.1(3))

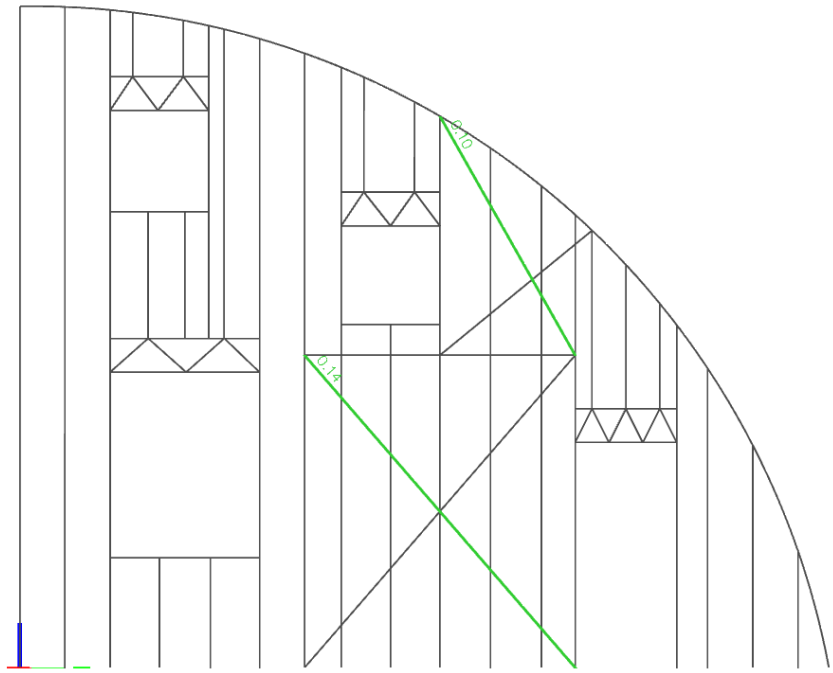
The critical check is on position 12.23 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	1430.70	lbf
Vux	-0.00	lbf
Vuy	-0.00	lbf
Mut	-0.00	lbfft
Mux	0.00	lbfft
Muy	0.00	lbfft

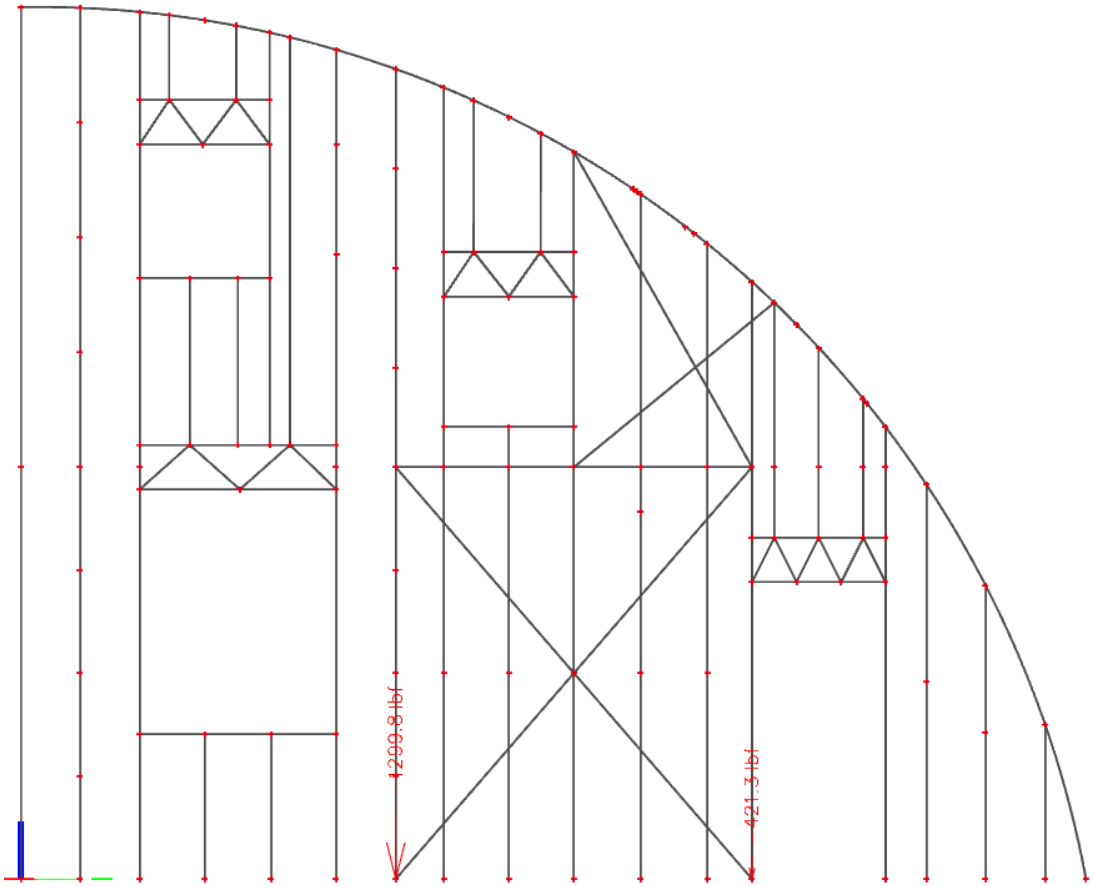
X-bracing Unity check





HOLDOWN DESIGN

Maximum support uplift reactions 1299 lb, from Combination LRFD-Ult (auto) 71 (0.9*DL1 + 0.9*DL2 + 0.9*DL3 + 0.9*DL4 + Wy+(0.18)).



Accept holdown Simpson S/HDU4 series. Maximum tension load 6345 lb for steel 54 mil.

Model	H (in.)	Fasteners		Stud Member Thickness[2] mil (ga.)	ASD (lb.)		LRFD (lb.)		Nominal Tension Load[6] (lb.)
		Anchor Bolt Diameter[1] (in.)	Stud Fasteners[7]		Tension Load	Deflection at ASD Load[5]	Tension Load	Deflection at LRFD Load[5]	
S/HDU4	7⅞	⅝	(6) #14	2–33 (2–20)	2,320	0.093	3,705	0.149	5,685
				2–43 (2–18)	3,825	0.115	6,105	0.190	9,365
				2–54 (2–16)	3,970	0.093	6,345	0.156	9,730
				Steel fixture	4,470	0.063	7,165	0.103	12,120

HOLDOWN TO FLOOR BEAM BOLT CONNECTIONS CHECK

Tensile resistance check (AISC 360-16: J3-1), 5/8” bolts

$\phi R_n = \phi \cdot F_{nt} * A_b = 10.36$ kip

Where:

$A_b = 0.307$ in2 – gross bolt cross-sectional area A307 − 12 Table 3

$F_{nt} = 45.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$\phi = 0.75$ – capacity factor

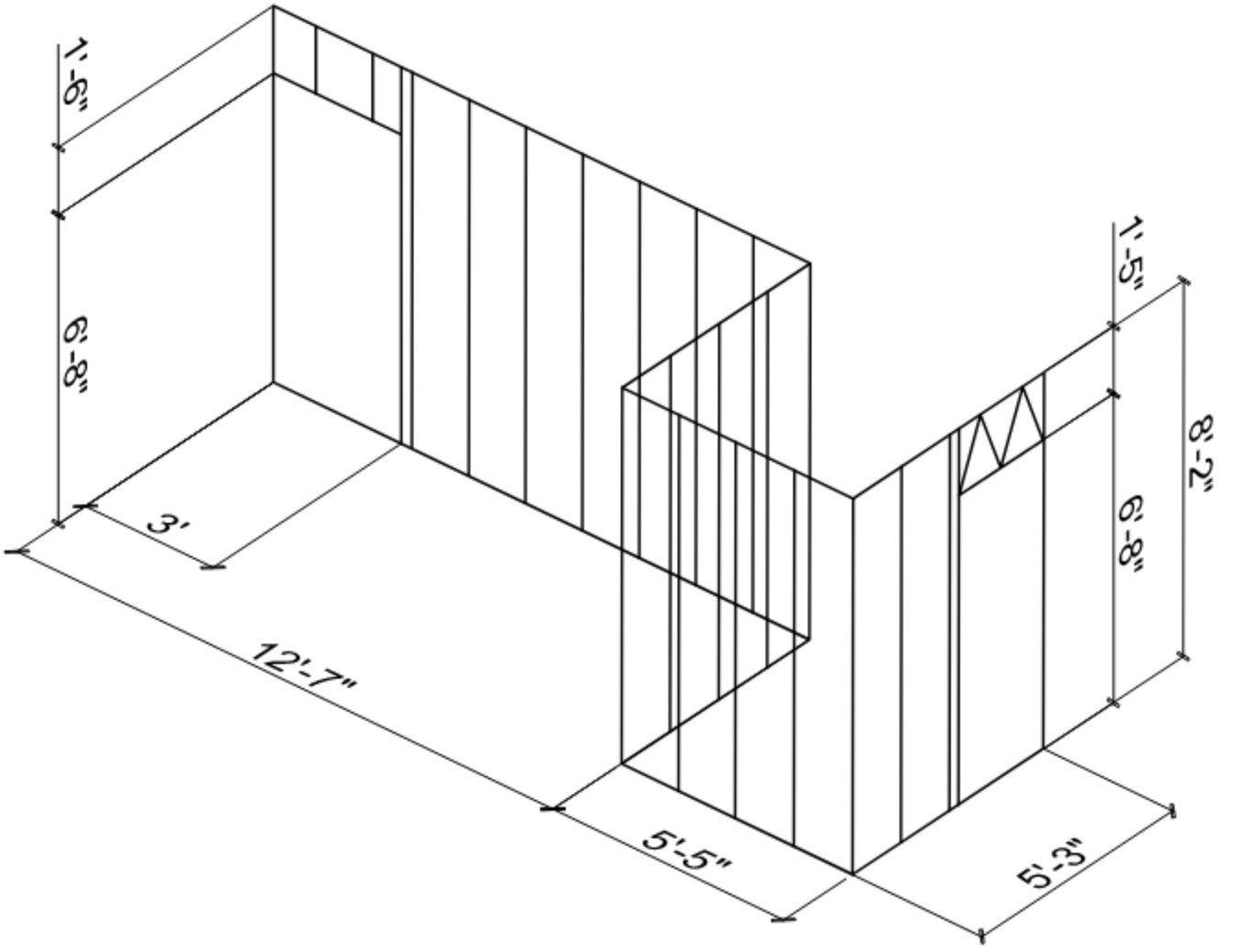
Maximum uplift support reactions - 1299lb = 1.3 kip.

1.3 kip < 10.36 kip

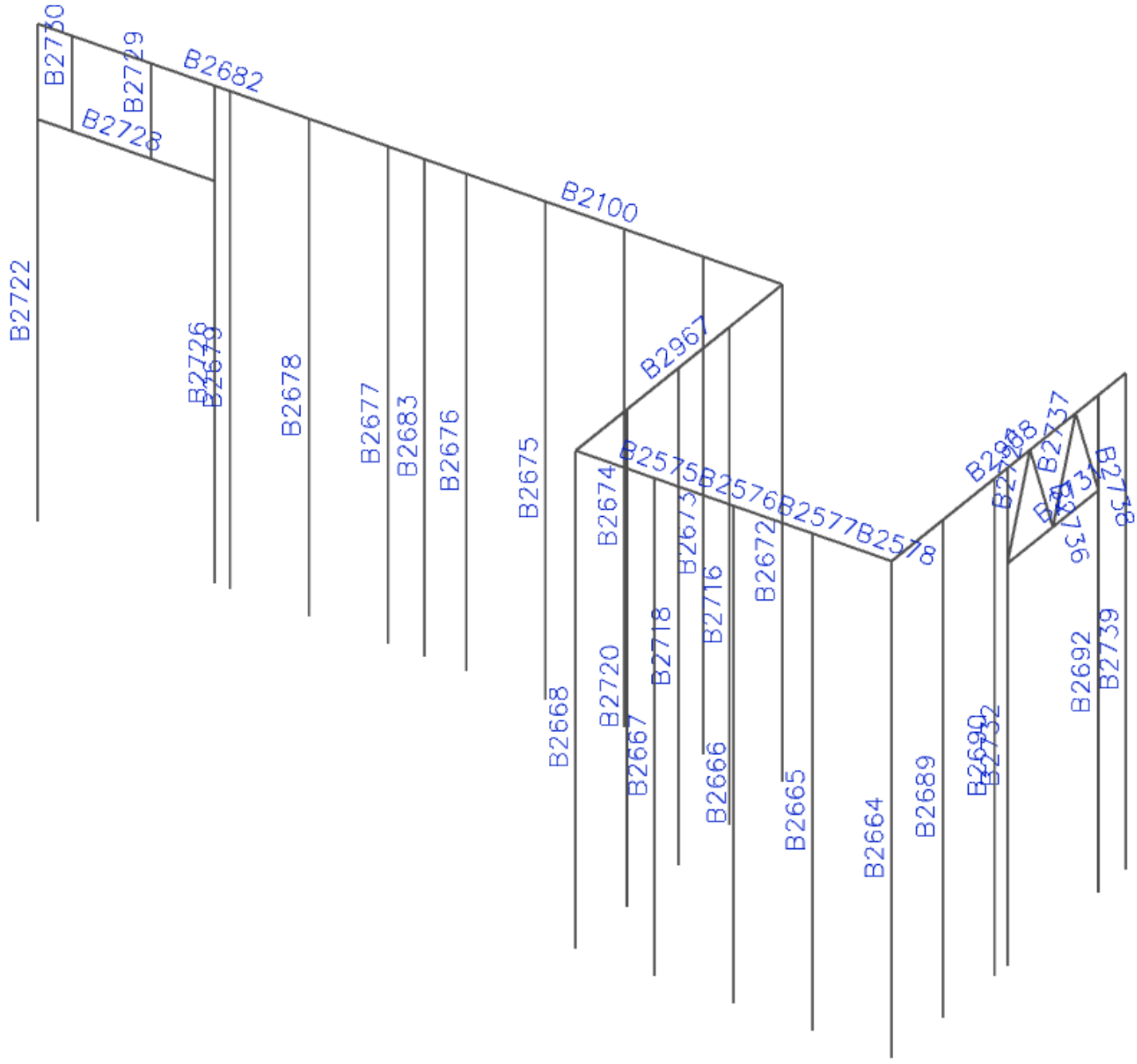
Connection is OK!.

2.4.6 INTERIOR WALL

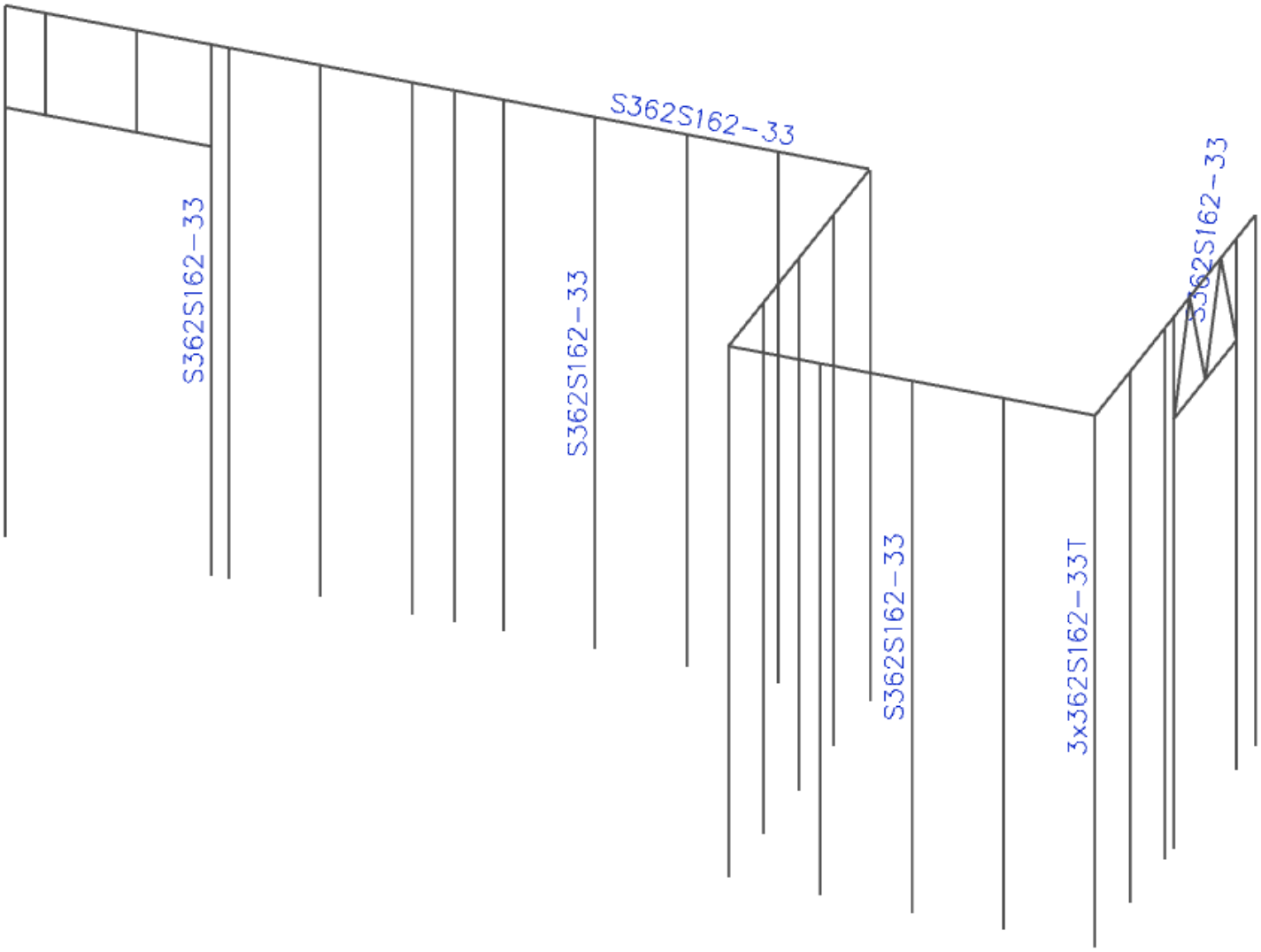
Wall stud spacing - 16";



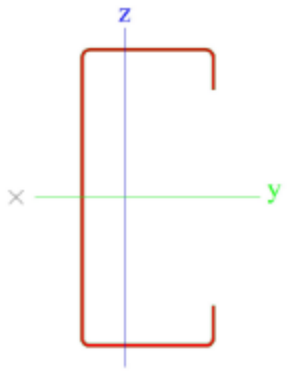
Numbers of members



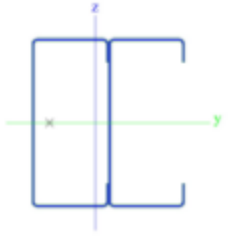
Cross-sections of members



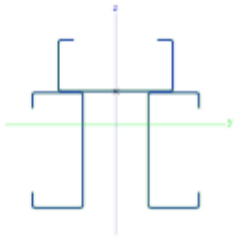
Cross-sections properties of 362S162-33

CS3		
Type	S362S162-33	
Formcode	114 - Cold formed C section	
Shape type	Thin-walled	
Item material	A653 grade 33	
Fabrication	cold formed	
Colour		
A [inch^2]	0,263	
A_y [inch^2], A_z [inch^2]	0,112	0,137
A_L [inch^2/inch], A_D [inch^2/inch]	1,52e+01	1,52e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	0,537	1,812
α [deg]	0,00	
I_y [inch^4], I_z [inch^4]	0,555	0,099
i_y [inch], i_z [inch]	1,452	0,614
$W_{el,y}$ [inch^3], $W_{el,z}$ [inch^3]	0,304	0,091
$W_{pl,y}$ [inch^3], $W_{pl,z}$ [inch^3]	0,351	0,135
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	1,16e+01	1,16e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	4,45e+00	4,45e+00
d_y [inch], d_z [inch]	-1,314	0,000
I_t [inch^4], I_w [inch^6]	0,000	0,297
β_y [inch], β_z [inch]	0,000	4,162
Picture		

Cross-sections properties of 2x362S162-33

CS24		
Type	2x362S162-33	
Shape type	Thin-walled	
Item material	A653 grade 33	
Fabrication	cold formed	
Colour		
A [inch²]	0,679	
A_y [inch²], A_z [inch²]	0,284	0,346
A_L [inch²/inch], A_D [inch²/inch]	1,87e+01	2,88e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	2,666	3,104
α [deg]	0,00	
I_y [inch⁴], I_z [inch⁴]	1,419	0,702
i_y [inch], i_z [inch]	1,445	1,017
$W_{el,y}$ [inch³], $W_{el,z}$ [inch³]	0,783	0,369
$W_{pl,y}$ [inch³], $W_{pl,z}$ [inch³]	0,906	0,552
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	2,99e+01	2,99e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	1,82e+01	1,82e+01
d_y [inch], d_z [inch]	-0,978	0,000
I_t [inch⁴], I_w [inch⁶]	0,589	0,855
β_y [inch], β_z [inch]	0,000	2,511
Picture		

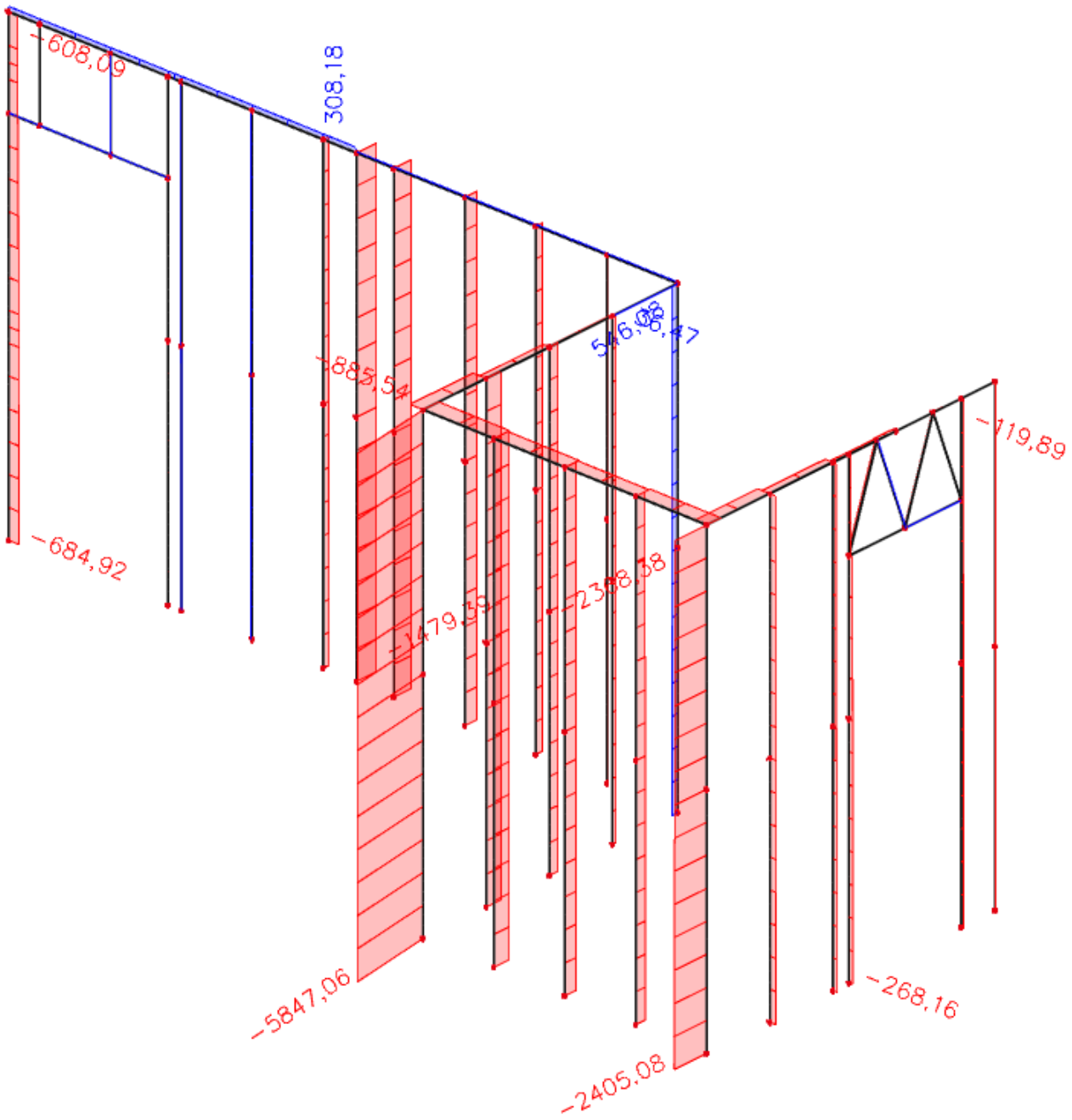
Cross-sections properties of 3x362S162-33 T

CS22		
Type	3x362S162-33T	
Shape type	Thin-walled	
Item material	A653 grade 33	
Fabrication	cold formed	
Colour		
A [inch²]	1,019	
A_y [inch²], A_z [inch²]	0,243	0,459
A_L [inch²/inch], A_D [inch²/inch]	4,31e+01	4,31e+01
$c_{Y,UCS}$ [inch], $c_{Z,UCS}$ [inch]	3,129	3,888
α [deg]	0,00	
I_y [inch⁴], I_z [inch⁴]	2,796	2,568
i_y [inch], i_z [inch]	1,657	1,588
$W_{el,y}$ [inch³], $W_{el,z}$ [inch³]	1,053	0,978
$W_{pl,y}$ [inch³], $W_{pl,z}$ [inch³]	1,399	1,497
$M_{pl,y,+}$ [kipinch], $M_{pl,y,-}$ [kipinch]	4,62e+01	4,62e+01
$M_{pl,z,+}$ [kipinch], $M_{pl,z,-}$ [kipinch]	4,94e+01	4,94e+01
d_y [inch], d_z [inch]	0,000	1,039
I_t [inch⁴], I_w [inch⁶]	0,001	3,073
β_y [inch], β_z [inch]	-2,761	0,000
Picture		

Explanations of symbols	
Formcode	s - Thickness r - Inner radius b - Flange width h - Height c - Lip
A	Area
A_y	Shear Area in principal y-direction
A_z	Shear Area in principal z-direction
A_L	Circumference per unit length
A_D	Drying surface per unit length
$c_{Y.UCS}$	Centroid coordinate in Y-direction of Input axis system
$c_{Z.UCS}$	Centroid coordinate in Z-direction of Input axis system
$I_{Y.LCS}$	Second moment of area about the YLCS axis
$I_{Z.LCS}$	Second moment of area about the ZLCS axis
$I_{YZ.LCS}$	Product moment of area in the LCS system
α	Rotation angle of the principal axis system
I_y	Second moment of area about the principal y-axis
I_z	Second moment of area about the principal z-axis
i_y	Radius of gyration about the principal y-axis

Explanations of symbols	
i_z	Radius of gyration about the principal z-axis
$W_{el.y}$	Elastic section modulus about the principal y-axis
$W_{el.z}$	Elastic section modulus about the principal z-axis
$W_{pl.y}$	Plastic section modulus about the principal y-axis
$W_{pl.z}$	Plastic section modulus about the principal z-axis
$M_{pl.y.+}$	Plastic moment about the principal y-axis for a positive My moment
$M_{pl.y.-}$	Plastic moment about the principal y-axis for a negative My moment
$M_{pl.z.+}$	Plastic moment about the principal z-axis for a positive Mz moment
$M_{pl.z.-}$	Plastic moment about the principal z-axis for a negative Mz moment
d_y	Shear center coordinate in principal y-direction measured from the centroid
d_z	Shear center coordinate in principal z-direction measured from the centroid
I_t	Torsional constant
I_w	Warping constant
β_y	Mono-symmetry constant about the principal y-axis
β_z	Mono-symmetry constant about the principal z-axis

Axial force diagram N,
LRFD-Ult (auto) 4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lbf.



Shear force diagram Vy,
LRFD-Ult (auto) 4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lbf.

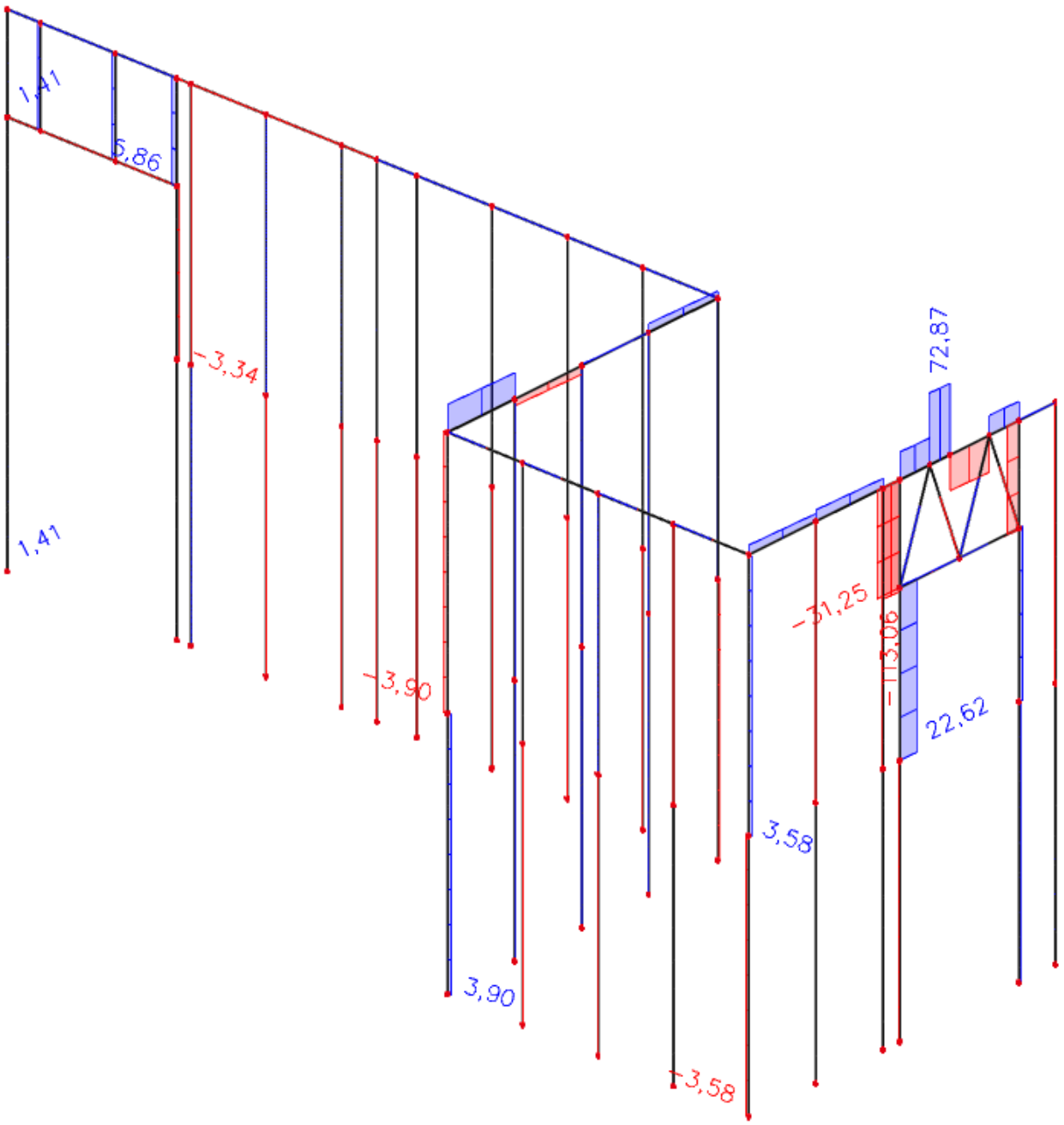
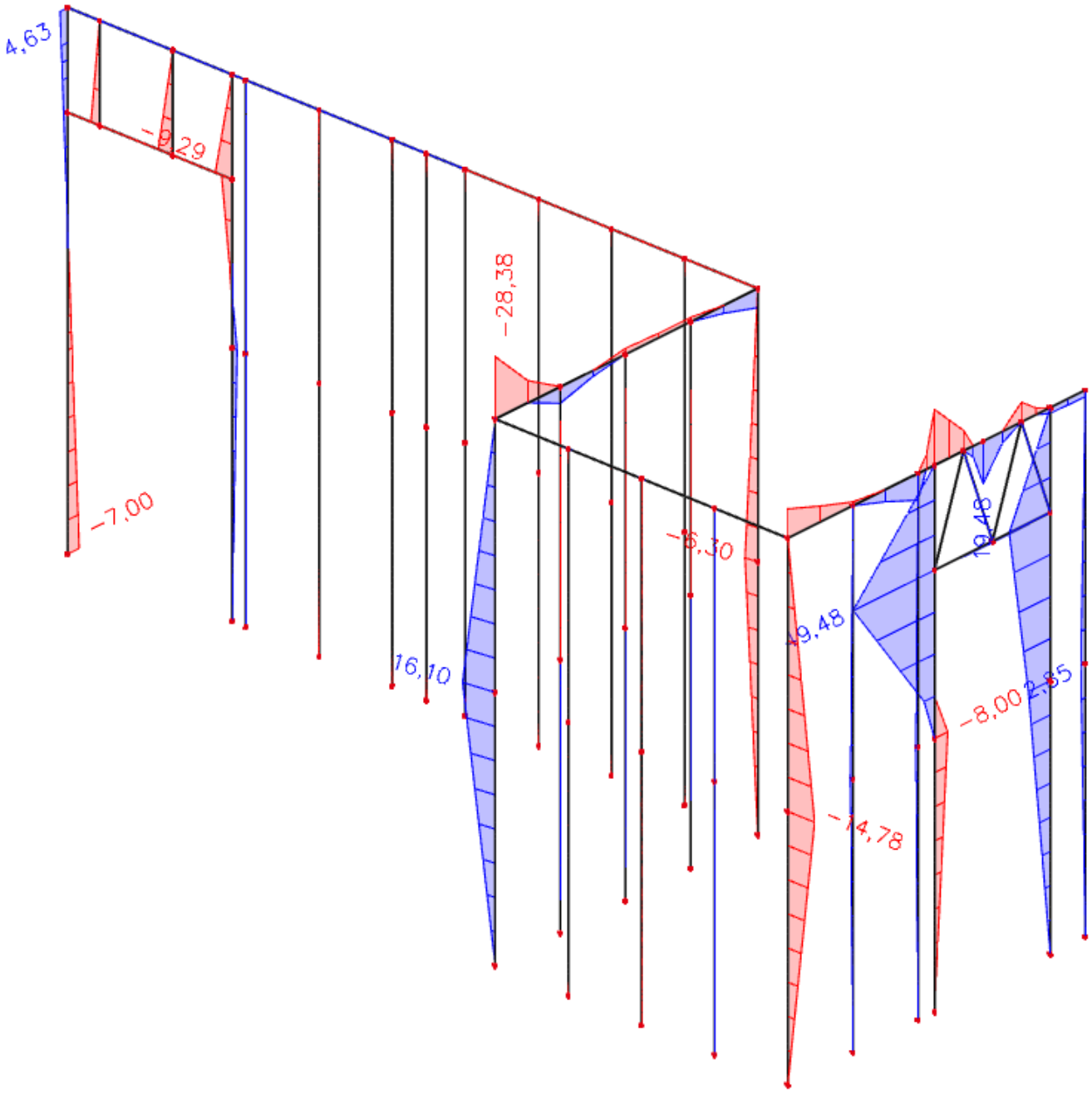


Diagram of bending moments My,
LRFD-Ult (auto) 4 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lbf*ft.



STEEL MEMBER B2664 CHECK

AISI S100-16 LRFD Check

Member B2664	3x362S162-33T	A653 grade 33	LRFD-Ult (auto)	0.36

Material data		
Yield stress Fy	33.00	ksi
Tensile stress Fu	45.00	ksi
fabrication	cold formed	

The critical check is on position 4.12 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-1561.42	lbf
Vux	-2.82	lbf
Vuy	153.26	lbf
Mut	0.00	lbfft
Mux	632.18	lbfft
Muy	-11.62	lbfft

...::Flexural Strength about X-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
2	1.580	33.000 13.169	0.40	5.636	120.405	0.524	1.000	- 1.580	0.607 0.972	-	- -	-
3	2.045	13.169 13.169	1.00	4.000	51.000	0.508	1.000	- 2.045	1.023 1.023	-	- -	-
4	1.580	33.000 13.169	0.40	5.636	120.405	0.524	1.000	- 1.580	0.607 0.972	-	- -	-
5	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
6	0.477	12.603 6.610	0.52	0.669	156.406	0.284	1.000	0.477 -	- -	-	- -	-
7	0.812	12.603 12.603	1.00	4.000	323.113	0.197	1.000	0.812 -	- -	-	- -	-
8	3.580	12.603 -32.331	2.57	101.775	423.485	0.173	1.000	- 3.580	0.643 1.790	-	- -	-
9	1.580	-32.331 -32.331	-	-	-	-	-	- -	- -	-	- -	-
10	0.477	-26.339 -32.331	-	-	-	-	-	- -	- -	-	- -	-

11	0.477	-26.339 -32.331	-	-	-	-	-	- -	- -	-	- -	-
12	1.580	-32.331 -32.331	-	-	-	-	-	- -	- -	-	- -	-
13	3.580	12.603 -32.331	2.57	101.775	423.485	0.173	1.000	- 3.580	0.643 1.790	-	- -	-
14	0.813	12.603 12.603	1.00	4.000	323.113	0.197	1.000	- 0.813	0.406 0.406	-	- -	-
15	0.477	12.603 6.610	0.52	0.669	156.406	0.284	1.000	0.477 -	- -	-	- -	-
16	0.767	12.886 12.886	1.00	4.000	1448.830	0.094	1.000	0.767 -	- -	-	- -	-
21	0.767	12.886 12.886	1.00	4.000	1448.830	0.094	1.000	- 0.767	0.384 0.384	-	- -	-

Table of values		
Sxe	1.054	$inch^3$
Mnxo	2899.57	lbfft
Resistance factor	0.90	
Unity check	0.24	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Lltb	4' 1.500"	ft
Sigma,ey	294.497	ksi
Kt	1.00	
Lt	4' 1.500"	ft
Sigma,t	56.933	ksi
Cb	1.67	
Sfx	1.053	$inch^3$
Fcre	525.815	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.
...::Flexural Strength about Y-axis::...

...::Shear Strength::...
Shear Strength
According to article G2.1 and formula (G2.1.1)
Shear force Vy

Element ID	Aw [$inch^2$]	Vn [lbf]
1	0.000	0.00
2	0.071	1410.82
3	0.000	0.00
4	0.071	1410.82
5	0.000	0.00
6	0.022	426.35
7	0.000	0.00
8	0.161	2759.99
9	0.000	0.00
10	0.022	426.35

11	0.022	426.35
12	0.000	0.00
13	0.161	2759.99
14	0.000	0.00
15	0.022	426.35
16	0.000	0.00
21	0.000	0.00

Table of values		
Vn,y	10047.04	lbf
Resistance factor	0.95	
Unity check	0.02	-

Combined Bending and Shear

According to article H2 and formula (H2-1)

Table of values		
Mnxo	2899.57	lbfft
Vny	10047.04	lbf
Resistance factor shear	0.95	
Resistance factor bending x	0.90	

Unity check (Mx, Vy) = sqrt(0.06+0.00) = 0.24

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
2	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
3	2.045	33.000 33.000	1.00	4.000	51.000	0.804	0.903	1.847 -	- -	-	- -	-
4	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
5	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
6	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
7	0.812	33.000 33.000	1.00	4.000	323.113	0.320	1.000	0.812 -	- -	-	- -	-
8	3.580	33.000 33.000	1.00	4.000	16.644	1.408	0.599	2.145 -	- -	-	- -	-
9	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
10	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
11	0.477	33.000	1.00	0.430	100.590	0.573	1.000	0.477	-	-	-	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
		33.000						-	-		-	
12	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
13	3.580	33.000 33.000	1.00	4.000	16.644	1.408	0.599	2.145 -	- -	-	- -	-
14	0.813	33.000 33.000	1.00	4.000	323.113	0.320	1.000	0.813 -	- -	-	- -	-
15	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
16	0.767	33.000 33.000	1.00	4.000	1448.830	0.151	1.000	0.767 -	- -	-	- -	-
21	0.767	33.000 33.000	1.00	4.000	1448.830	0.151	1.000	0.767 -	- -	-	- -	-

Table of values		
Fn	33.000	ksi
Ae	0.903	inch^2
Pno	29813.71	lbf
Resistance factor	0.85	
Unity check	0.06	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	non-sway	
Unbraced Length L	4 1/8	4 1/8	ft
Effective Length factor K	4.01	0.94	
Effective Length	16 5/8	4	ft
Slenderness	119.68	29.44	
Flexural Buckling stress Fcre	19.987	330.418	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	19.987	ksi
Sigma,ey	330.418	ksi
Kt	1.00	
Lt	4 1/8	ft
Sigma,t	56.933	ksi
Sigma,TF	19.987	ksi
Torsional (-Flexural) buckling stress Fcre	19.987	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.477	16.535 16.535	1.00	0.430	100.590	0.405	1.000	0.477 -	- -	-	- -	-
2	1.580	16.535 16.535	1.00	4.000	85.456	0.440	1.000	1.580 -	- -	-	- -	-
3	2.045	16.535 16.535	1.00	4.000	51.000	0.569	1.000	2.045 -	- -	-	- -	-
4	1.580	16.535 16.535	1.00	4.000	85.456	0.440	1.000	1.580 -	- -	-	- -	-
5	0.477	16.535 16.535	1.00	0.430	100.590	0.405	1.000	0.477 -	- -	-	- -	-
6	0.477	16.535 16.535	1.00	0.430	100.590	0.405	1.000	0.477 -	- -	-	- -	-
7	0.812	16.535 16.535	1.00	4.000	323.113	0.226	1.000	0.812 -	- -	-	- -	-
8	3.580	16.535 16.535	1.00	4.000	16.644	0.997	0.782	2.799 -	- -	-	- -	-
9	1.580	16.535 16.535	1.00	4.000	85.456	0.440	1.000	1.580 -	- -	-	- -	-
10	0.477	16.535 16.535	1.00	0.430	100.590	0.405	1.000	0.477 -	- -	-	- -	-
11	0.477	16.535 16.535	1.00	0.430	100.590	0.405	1.000	0.477 -	- -	-	- -	-
12	1.580	16.535 16.535	1.00	4.000	85.456	0.440	1.000	1.580 -	- -	-	- -	-
13	3.580	16.535 16.535	1.00	4.000	16.644	0.997	0.782	2.799 -	- -	-	- -	-
14	0.813	16.535 16.535	1.00	4.000	323.113	0.226	1.000	0.813 -	- -	-	- -	-
15	0.477	16.535	1.00	0.430	100.590	0.405	1.000	0.477	-	-	-	-

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
		16.535						-	-		-	
16	0.767	16.535 16.535	1.00	4.000	1448.830	0.107	1.000	0.767 -	- -	-	- -	-
21	0.767	16.535 16.535	1.00	4.000	1448.830	0.107	1.000	0.767 -	- -	-	- -	-

Table of values		
Fe	19.987	ksi
lambda, c	1.28	
Fn	16.535	ksi
Ae	0.971	$inch^2$
Pn	16060.88	lbf
Resistance factor	0.85	
Unity check	0.11	-

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (C5.2.1-3)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.477	1.532 1.532	1.00	0.430	100.590	0.123	1.000	0.477 -	- -	-	- -	-
2	1.580	1.532 1.532	1.00	4.000	85.456	0.134	1.000	1.580 -	- -	-	- -	-
3	2.045	1.532 1.532	1.00	4.000	51.000	0.173	1.000	2.045 -	- -	-	- -	-
4	1.580	1.532 1.532	1.00	4.000	85.456	0.134	1.000	1.580 -	- -	-	- -	-
5	0.477	1.532 1.532	1.00	0.430	100.590	0.123	1.000	0.477 -	- -	-	- -	-
6	0.477	1.532 1.532	1.00	0.430	100.590	0.123	1.000	0.477 -	- -	-	- -	-
7	0.812	1.532 1.532	1.00	4.000	323.113	0.069	1.000	0.812 -	- -	-	- -	-
8	3.580	1.532 1.532	1.00	4.000	16.644	0.303	1.000	3.580 -	- -	-	- -	-
9	1.580	1.532 1.532	1.00	4.000	85.456	0.134	1.000	1.580 -	- -	-	- -	-
10	0.477	1.532 1.532	1.00	0.430	100.590	0.123	1.000	0.477 -	- -	-	- -	-
11	0.477	1.532 1.532	1.00	0.430	100.590	0.123	1.000	0.477 -	- -	-	- -	-
12	1.580	1.532 1.532	1.00	4.000	85.456	0.134	1.000	1.580 -	- -	-	- -	-
13	3.580	1.532 1.532	1.00	4.000	16.644	0.303	1.000	3.580 -	- -	-	- -	-
14	0.813	1.532 1.532	1.00	4.000	323.113	0.069	1.000	0.813 -	- -	-	- -	-
15	0.477	1.532 1.532	1.00	0.430	100.590	0.123	1.000	0.477 -	- -	-	- -	-
16	0.767	1.532 1.532	1.00	4.000	1448.830	0.033	1.000	0.767 -	- -	-	- -	-
21	0.767	1.532 1.532	1.00	4.000	1448.830	0.033	1.000	0.767 -	- -	-	- -	-

Table of values		
Mnx	2899.57	lbfft
Mny	2654.66	lbfft
Pn	16060.88	lbf
Resistance factor compression	0.85	
Resistance factor bending x	0.90	
Resistance factor bending y	0.90	

Unity check = 0.11+0.24+0.00 = 0.36 - (C5.2.1-3)

The member satisfies the check !

STEEL MEMBER B2720 CHECK

AISI S100-16 LRFD Check

Member B2720	S362S162-33	A653 grade 33	LRFD-Ult (auto)	0.56

Material data		
Yield stress Fy	33.00	ksi
Tensile stress Fu	45.00	ksi
fabrication	cold formed	

The critical check is on position 0.00 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-1099.37	lbf
Vux	0.00	lbf
Vuy	0.01	lbf
Mut	0.00	lbfft
Mux	-0.00	lbfft
Muy	-0.00	lbfft

...::Axial Compression Strength::...
Nominal Axial Strength
According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.389	33.000 33.000	1.00	0.430	89.189	0.608	1.000	0.360 -	- -	-	- -	-
3	1.403	33.000 33.000	1.00	3.391	54.075	0.781	0.920	1.290 -	0.597 0.693	37.95	0.000 0.000	0.360
5	3.403	33.000 33.000	1.00	4.000	10.841	1.745	0.501	1.705 -	- -	-	- -	-
7	1.403	33.000 33.000	1.00	3.391	54.075	0.781	0.920	1.290 -	0.597 0.693	37.95	0.000 0.000	0.360
9	0.389	33.000 33.000	1.00	0.430	89.189	0.608	1.000	0.360 -	- -	-	- -	-

Table of values		
Fn	33.000	ksi
Ae	0.194	inch^2
Pno	6386.52	lbf
Resistance factor	0.85	
Unity check	0.20	-

Buckling check
According to article E2 and formula (E2-1)
Flexural Buckling Strength
According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	4 1/8	8 1/4	ft
Effective Length factor K	1.00	1.00	
Effective Length	4 1/8	8 1/4	ft
Slenderness	34.09	161.17	
Flexural Buckling stress Fcre	246.291	11.021	ksi

Torsional (-Flexural) Buckling Strength
According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	246.291	ksi
Sigma,ey	11.021	ksi
Kt	1.00	
Lt	4 1/8	ft
Sigma,t	32.371	ksi
Sigma,TF	11.021	ksi
Torsional (-Flexural) buckling stress Fcre	11.021	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.389	9.666 9.666	1.00	0.430	89.189	0.329	1.000	0.389 -	- -	-	- -	-
3	1.403	9.666 9.666	1.00	3.468	55.299	0.418	1.000	1.403 -	0.702 0.702	70.12	0.000 0.000	0.389
5	3.403	9.666 9.666	1.00	4.000	10.841	0.944	0.812	2.764 -	- -	-	- -	-
7	1.403	9.666 9.666	1.00	3.468	55.299	0.418	1.000	1.403 -	0.702 0.702	70.12	0.000 0.000	0.389
9	0.389	9.666 9.666	1.00	0.430	89.189	0.329	1.000	0.389 -	- -	-	- -	-

Table of values		
Fe	11.021	ksi
lambda, c	1.73	
Fn	9.666	ksi
Ae	0.240	inch^2
Pn	2319.72	lbf
Resistance factor	0.85	
Unity check	0.56	-

Distortional Buckling Strength
According to article E4 and formula (E4.1-2).

Table of values		
Py	8683.60	lbf
L	1' 5.014"	ft
k,phi,fe	71.10	lbf
k,phi,we	60.71	lbf
k,phi	0.00	lbf
k,phi,fg	0.003	$inch^2$
k,phi,wg	0.001	$inch^2$
Fd	32.519	ksi
Pcrd	8556.96	lbf
Lambda,d	1.01	
Pn	6474.43	lbf
Resistance factor	0.85	
Unity check	0.20	-

Data		
Lm	4' 1.500"	ft
Lcr	1' 5.014"	ft
h0	3.625	inch
Ixf	0.001	$inch^4$
Iyf	0.016	$inch^4$
Ixyf	0.002	$inch^4$
Cwf	0.000	$inch^6$
Jf	0.000	$inch^4$
x0f	0.566	inch
hxf	-1.012	inch
Af	0.067	$inch^2$
y0f	-0.055	inch

Number of compressed flanges: 2
Critical flange contains Initial shape parts: 2, 3, 1

The member satisfies the check !

STEEL MEMBER B2732 CHECK

AISI S100-16 LRFD Check

Member B2732	2x362S162-33	A653 grade 33	LRFD-Ult (auto)	0.80

Material data		
Yield stress Fy	33.00	ksi
Tensile stress Fu	45.00	ksi
fabrication	cold formed	

The critical check is on position 6.67 ft

Axis definition :
- local x- axis in this code check is referring to the local y axis in Scia Engineer
- local y- axis in this code check is referring to the local z axis in Scia Engineer

Internal forces		
Pu	-599.99	lbf
Vux	307.48	lbf
Vuy	-0.00	lbf
Mut	0.00	lbfft
Mux	-0.00	lbfft
Muy	693.77	lbfft

...::Flexural Strength about Y-axis::...
Nominal Flexural Strength
According to article F3.1 and formula (F3.1-1).

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.477	4.748 4.748	1.00	4.000	3742.875	0.036	1.000	0.477 -	- -	-	- -	-
2	1.580	4.350 -23.505	5.40	541.916	11577.480	0.019	1.000	- 1.580	0.188 0.790	-	- -	-
3	3.580	-23.505 -23.505	-	-	-	-	-	- -	- -	-	- -	-
4	1.580	4.350 -23.505	5.40	541.916	11577.480	0.019	1.000	- 1.580	0.188 0.790	-	- -	-
5	0.477	4.748 4.748	1.00	4.000	3742.875	0.036	1.000	0.477 -	- -	-	- -	-
6	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
7	1.580	33.000 5.145	0.16	6.891	147.218	0.473	1.000	- 1.580	0.556 1.024	-	- -	-
8	2.625	5.145 5.145	1.00	4.000	30.956	0.408	1.000	2.625 -	- -	-	- -	-
9	1.580	33.000 5.145	0.16	6.891	147.218	0.473	1.000	- 1.580	0.556 1.024	-	- -	-
10	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-

Table of values		
Sye	0.371	$inch^3$
Mnyo	1019.18	lbfft
Resistance factor	0.90	
Unity check	0.76	-

Lateral-Torsional Buckling Strength
According to article F2.1 and formula (F2.1-1),(F2.1.1-1).

Table of values		
Sigma,ex	61.022	ksi
Kt	1.00	
Lt	2' 6.500"	ft
Sigma,t	2466.802	ksi
Cb	1.00	
Sfy	0.369	$inch^3$
Fcre	1440.690	ksi

Note: Lateral-Torsional buckling is not governing since Fe is greater than or equal to 2.78 Fy.

...::Shear Strength::...
Shear Strength
According to article G2.1 and formula (G2.1.1)
Shear force Vx

Element ID	Aw [$inch^2$]	Vn [lbf]
1	0.000	0.00
2	0.071	1410.82
3	0.000	0.00
4	0.071	1410.82
5	0.000	0.00
6	0.000	0.00
7	0.071	1410.82
8	0.000	0.00
9	0.071	1410.82
10	0.000	0.00

Table of values		
Vn,x	5643.28	lbf
Resistance factor	0.95	
Unity check	0.06	-

Combined Bending and Shear
According to article H2 and formula (H2-1)

Table of values		
Mnyo	1019.18	lbfft
Vnx	5643.28	lbf
Resistance factor shear	0.95	
Resistance factor bending y	0.90	

Unity check (My, Vx) = sqrt(0.57+0.00) = 0.76

...::Axial Compression Strength::...

Nominal Axial Strength

According to article E2 and formula (E2-1)

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch^4]	ds [inch]
1	0.477	33.000 33.000	1.00	4.000	3742.875	0.094	1.000	0.477 -	- -	-	- -	-
2	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
3	3.580	33.000 33.000	1.00	4.000	16.644	1.408	0.599	2.145 -	- -	-	- -	-
4	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
5	0.477	33.000 33.000	1.00	4.000	3742.875	0.094	1.000	0.477 -	- -	-	- -	-
6	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-
7	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
8	2.625	33.000 33.000	1.00	4.000	30.956	1.032	0.762	2.001 -	- -	-	- -	-
9	1.580	33.000 33.000	1.00	4.000	85.456	0.621	1.000	1.580 -	- -	-	- -	-
10	0.477	33.000 33.000	1.00	0.430	100.590	0.573	1.000	0.477 -	- -	-	- -	-

Table of values		
Fn	33.000	ksi
Ae	0.602	inch^2
Pno	19856.83	lbf
Resistance factor	0.85	
Unity check	0.04	-

Buckling check

According to article E2 and formula (E2-1)

Flexural Buckling Strength

According to article E2.1 and formula (E2.1-1)

Buckling parameters	xx	yy	
Sway type	sway	sway	
Unbraced Length L	8 1/4	2 5/8	ft
Effective Length factor K	1.00	1.00	
Effective Length	8 1/4	2 5/8	ft
Slenderness	68.50	30.00	
Flexural Buckling stress Fcre	61.022	318.078	ksi

Torsional (-Flexural) Buckling Strength

According to article E2.2, E2.3, E2.4

Table of values		
Sigma,ex	61.022	ksi
Sigma,ey	318.078	ksi
Kt	1.00	
Lt	2 5/8	ft
Sigma,t	2466.802	ksi
Sigma,TF	60.663	ksi
Torsional (-Flexural) buckling stress Fcre	60.663	ksi

Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch⁴]	ds [inch]
1	0.477	26.280 26.280	1.00	4.000	3742.875	0.084	1.000	0.477 -	- -	-	- -	-
2	1.580	26.280 26.280	1.00	4.000	85.456	0.555	1.000	1.580 -	- -	-	- -	-
3	3.580	26.280 26.280	1.00	4.000	16.644	1.257	0.656	2.350 -	- -	-	- -	-
4	1.580	26.280 26.280	1.00	4.000	85.456	0.555	1.000	1.580 -	- -	-	- -	-
5	0.477	26.280 26.280	1.00	4.000	3742.875	0.084	1.000	0.477 -	- -	-	- -	-
6	0.477	26.280 26.280	1.00	0.430	100.590	0.511	1.000	0.477 -	- -	-	- -	-
7	1.580	26.280 26.280	1.00	4.000	85.456	0.555	1.000	1.580 -	- -	-	- -	-
8	2.625	26.280 26.280	1.00	4.000	30.956	0.921	0.826	2.169 -	- -	-	- -	-
9	1.580	26.280 26.280	1.00	4.000	85.456	0.555	1.000	1.580 -	- -	-	- -	-
10	0.477	26.280 26.280	1.00	0.430	100.590	0.511	1.000	0.477 -	- -	-	- -	-

Table of values		
Fe	60.663	ksi
lambda, c	0.74	
Fn	26.280	ksi
Ae	0.619	$inch^2$
Pn	16255.60	lbf
Resistance factor	0.85	
Unity check	0.04	-

Combined Compressive Axial Load and Bending
According to article H1.2 and formulas (C5.2.1-3)

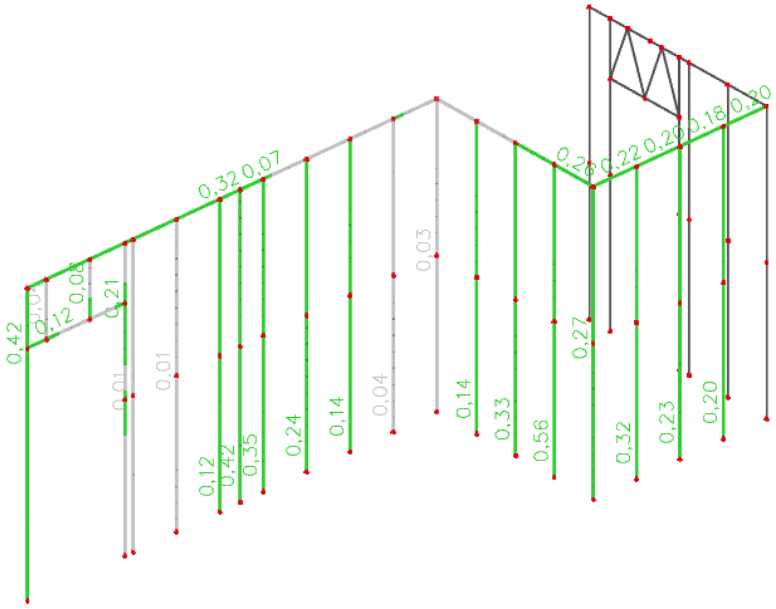
Id	w [inch]	f1 f2 [ksi]	psi [-]	k [-]	Fcr [ksi]	lambda [-]	rho [-]	b be [inch]	b1 b2 [inch]	S [-]	Ia Is [inch ^{4}]	ds [inch]
1	0.477	0.883 0.883	1.00	4.000	3742.875	0.015	1.000	0.477 -	- -	-	- -	-
2	1.580	0.883 0.883	1.00	4.000	85.456	0.102	1.000	1.580 -	- -	-	- -	-
3	3.580	0.883 0.883	1.00	4.000	16.644	0.230	1.000	3.580 -	- -	-	- -	-
4	1.580	0.883 0.883	1.00	4.000	85.456	0.102	1.000	1.580 -	- -	-	- -	-
5	0.477	0.883 0.883	1.00	4.000	3742.875	0.015	1.000	0.477 -	- -	-	- -	-
6	0.477	0.883 0.883	1.00	0.430	100.590	0.094	1.000	0.477 -	- -	-	- -	-
7	1.580	0.883 0.883	1.00	4.000	85.456	0.102	1.000	1.580 -	- -	-	- -	-
8	2.625	0.883 0.883	1.00	4.000	30.956	0.169	1.000	2.625 -	- -	-	- -	-
9	1.580	0.883 0.883	1.00	4.000	85.456	0.102	1.000	1.580 -	- -	-	- -	-
10	0.477	0.883 0.883	1.00	0.430	100.590	0.094	1.000	0.477 -	- -	-	- -	-

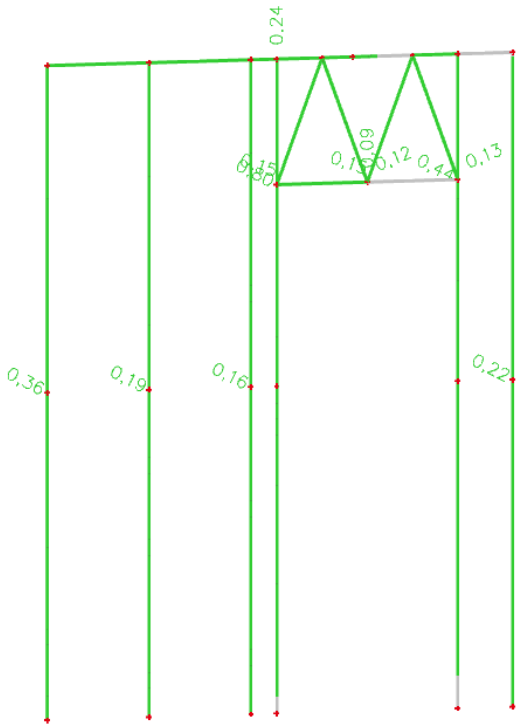
Table of values		
Mny	1019.18	lbfft
Pn	16255.60	lbf
Resistance factor compression	0.85	
Resistance factor bending y	0.90	

Unity check = 0.04+0.00+0.76 = 0.80 - (C5.2.1-3)

The member satisfies the check !

Unity Check


0,42
0,12
0,08
0,21
0,01
0,01
0,12
0,42
0,35
0,24
0,14
0,04
0,03
0,14
0,33
0,56
0,27
0,32
0,23
0,20
0,32
0,07
0,26
0,22
0,20
0,18
0,20


0,24
0,15
0,80
0,09
0,12
0,44
0,13
0,36
0,19
0,16
0,22

2.4.7 BASE SHEAR

SEISMIC FORCES (ASCE 7-16)

Tedds calculation version 3.1.00

Site parameters			
Site class	D		
Mapped acceleration parameters			
at short periods	S_S = **0.75**	at 1 sec period	S_1 = **0.3**
Site coefficientat short periods	F_a = **1.2**	at 1 sec period	F_v = **1.8**
Spectral response acceleration parameters			
at short period	S_{MS} = **0.900**	at 1 sec period	S_{M1} = **0.540**
Design spectral acceleration parameters			
at short period	S_{DS} = **0.600**	at 1 sec period	S_{D1} = **0.360**
Seismic design category			
Occupancy category		II	
Seismic design category (Table 11.6-1 only)		D	
Approximate fundamental period			
Height above base to highest level of building		h_n = **20.00** ft	
Building period parameter C_t	C_t = **0.02**	Building period parameter x	x = **0.75**
Building fundamental period	T = T_a = **0.189** sec	Long-period transition period	T_L = **16** sec
Limiting period	T_S = **0.600** sec		
Seismic response coefficient			
Seismic force resisting system:A. Bearing_Wall_Systems			
	18. Light-framed wall systems using flat strap bracing		
Response modification factor	R = **4**		
Seismic importance factor	I_e = **1.000**	Seismic response coefficient	C_s = **0.150**
Seismic base shear			
Effective seismic weight of the structure		W = **40.9** kips	
Seismic response coefficient	C_s = **0.150**	Seismic base shear	V = **6.13** kips

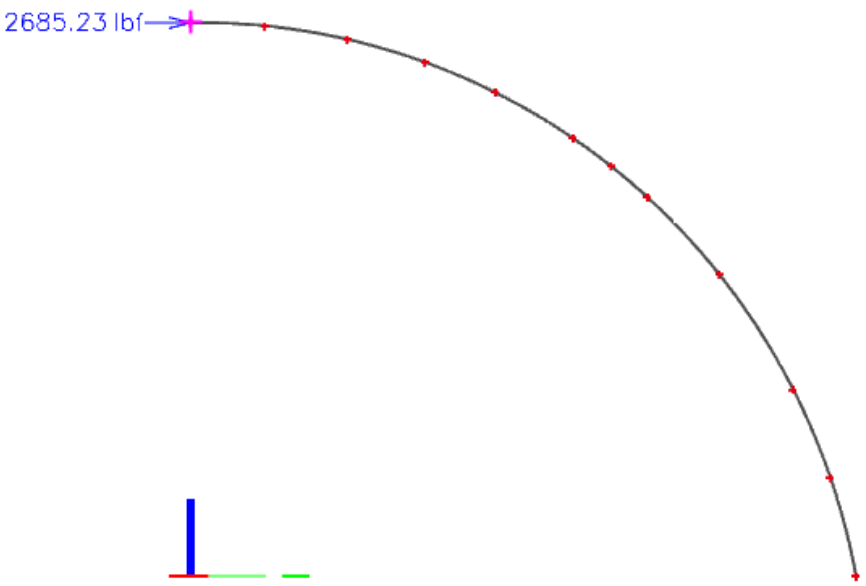
Transverse direction base shear (Section 1.4 Wind Load) 11.7 kip
Longitudinal direction base shear (Section 1.4 Wind Load) 8.1 kip
Seismic base shear 6.13 kip

According to the calculation results, wind effects affect the calculation in the longitudinal and transverse direction.

2.4.8 CONNECTIONS DESIGN

2.4.8.1 ARCH TO EXTERIOR WALL CONNECTION DESIGN

Maximum force transfer from arch roof to exterior wal is 2685 lb.
Combination8 LRFD-Ult (auto)8 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*Lr + 0.5*L), lbf.



The connection of the arch with the wall provided bolts with a diameter of 3/8. Each element of the arch is attached to the wall by 2 bots.

BOLT CONNECTIONS

Shear resistance check (AISC 360-16: J3-1)

$\phi R_n = \phi \cdot F_{nv} * A_b = 2.22$ kip

Where:

$A_b = 0.110$ in2 – gross bolt cross-sectional area

$F_{nv} = 27.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$\phi = 0.75$ – capacity factor

2 *2.22 kip = 4.44 kip

2.68 kip < 4.44 kip **Connection is OK!**

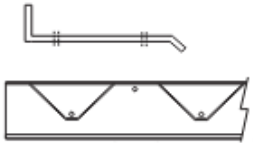
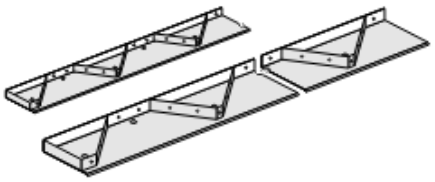
2.4.8.2 ARCH TO I-BEAM CONNECTIONS DESIGN

Support reactions of single roof arch

Combinations	Rx [lbf\ft]	Ry [lbf/ft]	Rz [lb]
LRFD-Ult (auto)51	**-1400**	-270	500
LRFD-Ult (auto)22	-220	**-700**	2020
LRFD-Ult (auto)69	770	10	**-600**
LRFD-Ult (auto)24	-330	-580	**2220**

The connection of the arch with the foundation is provided through a Standard Industrial Arch Connector. We accept that 2 anchor bolts will be used to fasten one element of the arch to the foundation. Each element is attached to 8 bots with a diameter of 3/8“.

STANDARD BASE CONNECTION COMPONENTS

Description	2-D with specs	3-D
Standard Industrial Arch Connector (For A, XA, S, XS and certain Q and XQ models)		

BOLT CONNECTIONS

Shear resistance check (AISC 360-16: J3-1)

$\phi R_n = \phi \cdot F_{nv} * A_b = 2.22$ kip

Where:

$A_b = 0.110$ in2 – gross bolt cross-sectional area

$F_{nv} = 27.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$\phi = 0.75$ – capacity factor

8 *2.22 kip = 17.76 kip

2220 + 700 + 1400 = 3320 lb = 3.32 kip 17.76 kip > 3.32 kip **Connection is OK!**

STANDARD CONNECTOR BOLT TO FLOOR BEAM CONNECTIONS

Tensile resistance check (AISC 360-16: J3-1), 1/2” bolts

$\phi R_n = \phi \cdot F_{nt} * A_b = 4.789$ kip

Where:

$A_b = 0.1419$ in2 – gross bolt cross-sectional area A307 − 12 Table 3

$F_{nt} = 45.0$ ksi – nominal shear stress from AISC 360-16 Table J3.2

$\phi = 0.75$ – capacity factor

2 *4.789 kip = 9.58 kip

Maximum uplift support reactions - 600lb = 0.211 kip.

9.58 kip > 0.600 kip **Connection is OK!**

2.4.8.3 I-BEAM TO I-BEAM CONNECTION

Project data

Project name	Q_Larsen
Project number	22-108
Author	DZ
Description	I-BEAM TO I-BEAM CONNECTION
Date	28.10.2022
Design code	AISC 360-16

Material

Steel	A36

Design

Name	CON1
Description	
Analysis	Stress, strain/ simplified loading
Design code	AISC - LRFD

Beams and columns

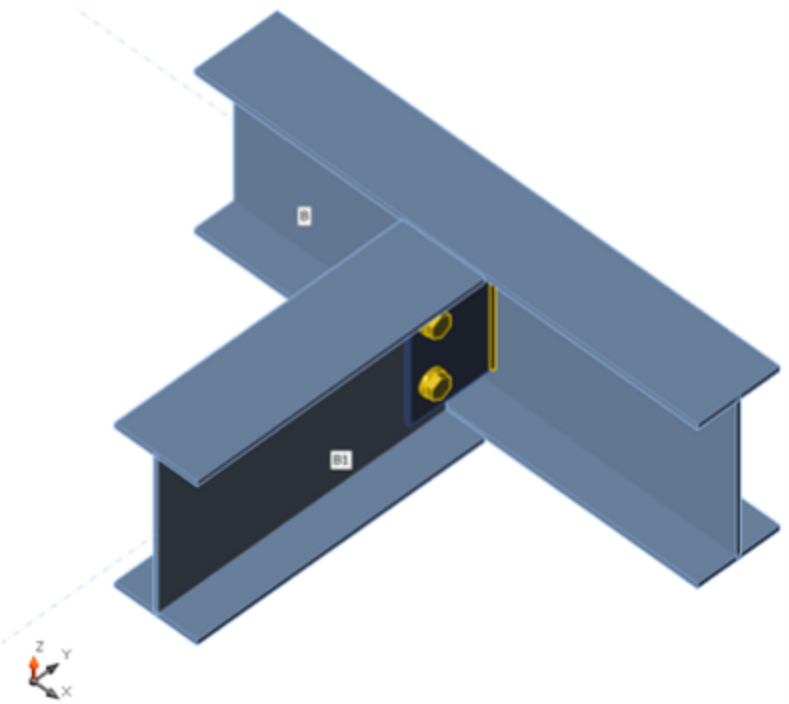
Name	Cross-section	β – Direction [°]	γ - Pitch [°]	α - Rotation [°]	Offset ex [in]	Offset ey [in]	Offset ez [in]	Forces in	X [in]
B	1 - CON1(W(Imp)8X13)	0.0	0.0	0.0	0.000	0.000	0.000	Node	0.000
B1	1 - CON1(W(Imp)8X13)	-90.0	0.0	0.0	0.000	0.000	0.000	Bolts	3.114

Cross-sections

Name	Material
1 - CON1(W(Imp)8X13)	A36

Bolts

Name	Bolt assembly	Diameter [in]	fu [ksi]	Gross area [in^2]
1/2 A307	1/2 A307	0.500	60.0	0.196



Load effects (equilibrium not required)

Name	Member	N [kip]	Vy [kip]	Vz [kip]	Mx [kip.in]	My [kip.in]	Mz [kip.in]
Comb 24	B1	0.200	0.000	4.900	0.00	0.00	0.00
Comb 71	B1	0.200	0.000	-6.250	0.00	0.00	0.00

Check

Summary

Name	Value	Check status
Analysis	100.0%	OK
Plates	0.1 < 5%	OK
Bolts	78.8 < 100%	OK
Welds	89.0 < 100%	OK
Buckling	Not calculated	

Plates

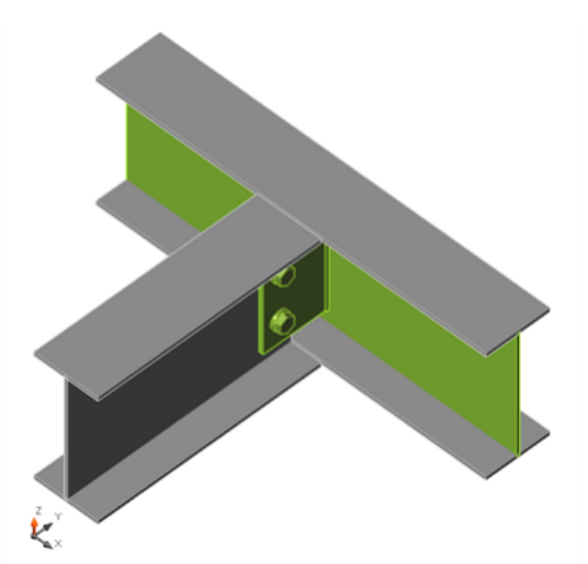
Name	F_y [ksi]	Thickness [in]	Loads	σ_{Ed} [ksi]	ε_{Pl} [%]	Check status
B-bfl 1	36.0	1/4"	Comb 71	13.9	0.0	OK
B-tfl 1	36.0	1/4"	Comb 71	14.9	0.0	OK
B-w 1	36.0	1/4"	Comb 71	32.4	0.1	OK
B1-bfl 1	36.0	1/4"	Comb 71	9.0	0.0	OK
B1-tfl 1	36.0	1/4"	Comb 71	9.0	0.0	OK
B1-w 1	36.0	1/4"	Comb 71	26.4	0.0	OK
FP1	36.0	1/4"	Comb 71	29.3	0.0	OK

Design data

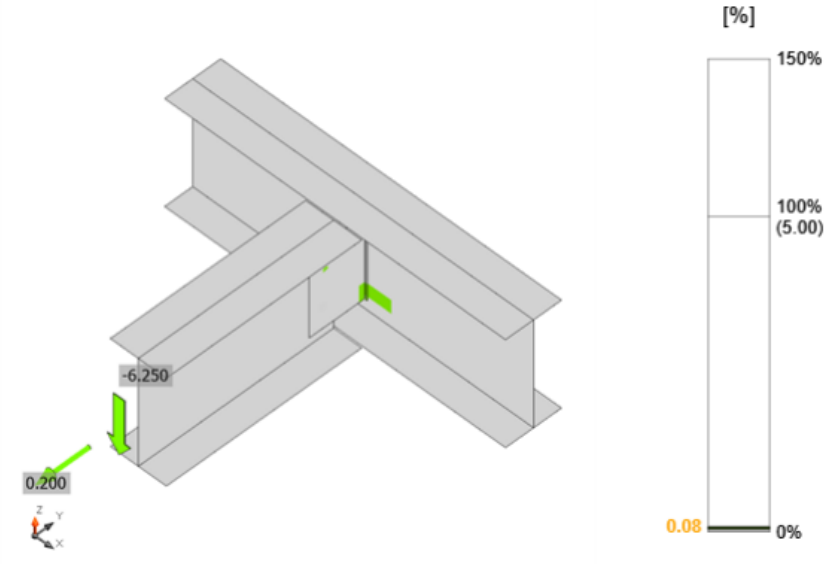
Material	fy [ksi]	ε_{lim} [%]
A36	36.0	5.0

Symbol explanation

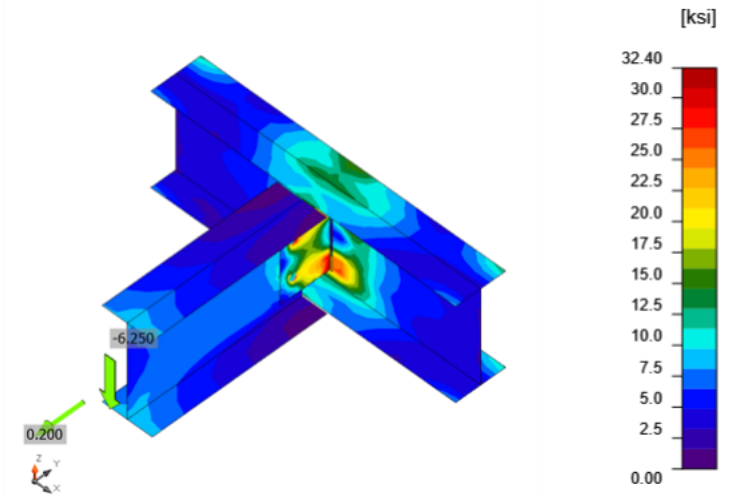
ε_{Pl}	Plastic strain
σ_{Ed}	Eq. stress
fy	Yield strength
ε_{lim}	Limit of plastic strain



Overall check, Comb 71

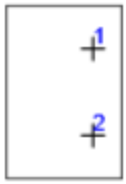


Strain check, Comb 71



Equivalent stress, Comb 71

Bolts

Shape	Item	Grade	Loads	F_t [kip]	V [kip]	$\phi R_{n,bearing}$ [kip]	Ut_t [%]	Ut_s [%]	Ut_{ts} [%]	Status
	B1	1/2 A307 - 1	Comb 71	0.939	3.122	11.923	14.2	78.6	-	OK
	B2	1/2 A307 - 1	Comb 71	0.296	3.131	11.923	4.5	78.8	-	OK

Design data

Grade	$\phi R_{n,tension}$ [kip]	$\phi R_{n,shear}$ [kip]
1/2 A307 - 1	6.621	3.973

Symbol explanation

F_t	Tension force
V	Resultant of shear forces Vy, Vz in bolt
$\phi R_{n,bearing}$	Bolt bearing resistance
Ut_t	Utilization in tension
Ut_s	Utilization in shear
Ut_{ts}	Utilization in tension and shear
$\phi R_{n,tension}$	Bolt tension resistance AISC 360-16 J3.6
$\phi R_{n,shear}$	Bolt shear resistance AISC 360-16 – J3.8

Welds

Item	Edge	Xu	T_h [in]	L_s [in]	L [in]	L_c [in]	F_n [kip]	ϕR_n [kip]	Ut [%]	Status
B-w 1	FP1	E60xx	◢1/16"◣	◢1/8"◣	5.963	0.398	1.260	1.416	89.0	OK
		E60xx	◢1/16"◣	◢1/8"◣	5.963	0.398	1.190	1.410	84.4	OK

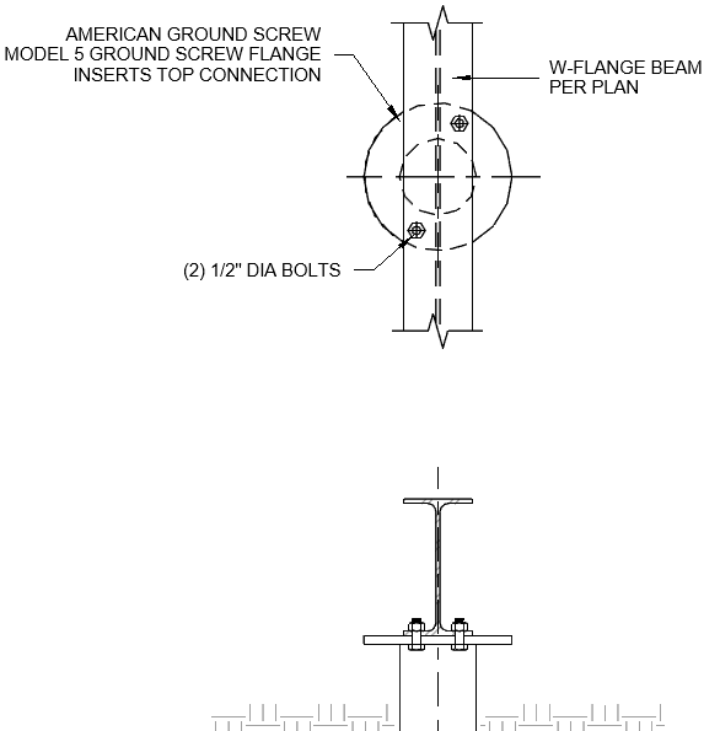
Symbol explanation

T_h	Throat thickness of weld
L_s	Leg size of weld
L	Length of weld
L_c	Length of weld critical element
F_n	Force in weld critical element
ϕR_n	Weld resistance AISC 360-16 J2.4
Ut	Utilization

Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.40	-	ACI 349 – B.6.1.4
Friction coefficient in slip-resistance	0.30	-	AISC 360-16 – J3.8
Limit plastic strain	0.05	-	
Weld stress evaluation	Plastic redistribution		
Detailing	No		
Distance between bolts [d]	2.66	-	AISC 360-16 – J3.3
Distance between bolts and edge [d]	1.25	-	AISC 360-16 – J.3.4
Concrete breakout resistance	Yes		
Base metal capacity check at weld fusion face	No		AISC 360-16 – J2-2
Cracked concrete	Yes		ACI 318-14 – Chapter 17
Local deformation check	No		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Large deformations for hollow sections

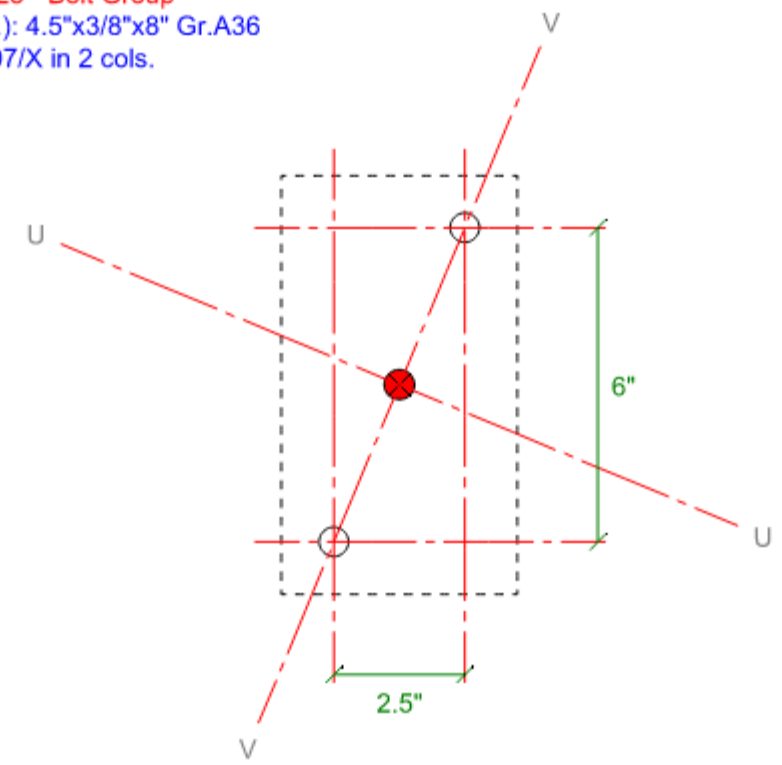
2.4.8.4 I-BEAM TO GROUND SCREW CONNECTION DESIGN



I-BEAM TO GROUND SCREW CONNECTION SKETCH

```
LIMCON V3.63.2.4 [ 0]

 Connection: 123
       Type: Bolt Group
    Country: US
      Units: US customary (kip-in)
Design code: ANSI/AISC 360 (LRFD)

Plate (min.):
 8"x4.5"x0.375"   Gr./Fy/Fu=A36/3/58ksi

Bolts:
 4 x 1/2" A307/X in 2 cols. and 2 rows at 2.5" gauge and 6" pitch.
 Min. X direction edge distance  .  1.0000 in.
 Min. Y direction edge distance  .  1.0000 in.
 No. shear planes  . . . . . .       1
```

CONNECTION: 123 - Bolt Group
Gusset plate (min.): 4.5"x3/8"x8" Gr.A36
Bolts: 2 x 1/2" A307/X in 2 cols.



xc: 1-1/4"
yc: 3"
Angle: -22.62

Design actions:

Horiz. force, F*x	3.5 kip
Vert. force, F*y	4.8 kip
Force, F*z	2.4 kip
Moment, M*x	0.0 kip-in
Moment, M*y	0.0 kip-in
Moment, M*z	0.0 kip-in

Angle of load from vert.	36.10°
Eccentricity	0.0000 in.

GEOMETRY CHECK

Horizontal edge distance	1.00"	≥	0.75"	Yes
Vertical edge distance	1.00"	≥	0.75"	Yes
Horizontal bolt spacing	2.50"	≥	1.33"	Yes
Vertical bolt spacing	6.00"	≥	1.33"	Yes

ANALYSIS RESULT
Using elastic method for in-plane checks...

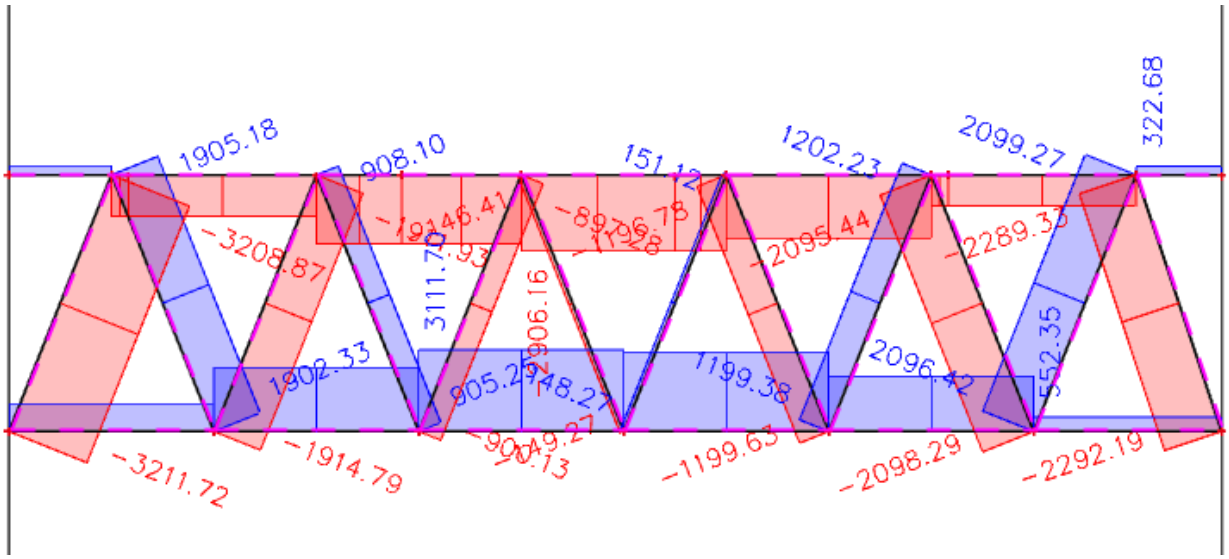
No. bolts	2

Bolt centroid measured from bottom left bolt:

```
  Centroid x coordinate, xc  . . .   1.250 in.
  Centroid y coordinate, yc  . . .   3.000 in.
 Angle to principal axes U-U . . .  -22.62°
 Moment of inertia about XX  . . .   18.00 in2
 Moment of inertia about YY  . . .    3.13 in2
 Polar moment of inertia . . . . .   21.12 in2

WARNING: Block shear is not checked.

DESIGN STRENGTH CHECKS
                                                  Strength ratio ──────────┐
                                                  Required strength ┐      │
                                                 ┌─Design strength  │      │          Reference
                                                 │                  │      │          ═══════════
 BEARING/TEARING IN PART 1...
 Resultants in cartesian axis directions:
 (These are statically equivalent to bolt group actions.)
  Horiz. resultant . . . . . .     3.5 kip
  Vert. resultant  . . . . . .     4.8 kip
  Moment resultant . . . . . .     0.0 kip-in
 Bolt shear (critical bolt row, column)...
               (1,1) is top left   (Row,Col.)
  Bolt shear . . . . . . . . . . (1,2)     5.0  ≥  Vures =    3.0  1.68  Pass
 Bolt bearing/tearing (critical bolt row, column)...
  Bolt bearing . . . . . . . . . (1,2)    19.6  ≥  Vures =    3.0  6.59  Pass
  Bolt group ext. tearing (top)  . . (1,2)    14.1  ≥  Vuv   =    2.4  5.86  Pass  A360 (J3-6a)
  Bolt group ext. tearing (right)  . (1,2)    14.1  ≥  Vuh   =    1.8  8.04  Pass  A360 (J3-6a)

 Out-of-plane checks
  Prying factor  . . . . . .    0.00
  Single bolt tension strength . . . . . .     6.6 kip
  Bolt shear+tension interaction:
   Shear in critical bolt  . . . . . (1,2)     3.0 kip
   Tension in critical bolt  . . . . (1,2)     1.2 kip
   Bolt shear+tension interaction  . . . . .    1.00  ≥           0.60  1.67  Pass  A360 J3.7

 BEARING/TEARING IN PART 2...
 Resultants in cartesian axis directions:
 (These equilibrate bolt group actions.)
  Horiz. resultant . . . . . .    -3.5 kip
  Vert. resultant  . . . . . .    -4.8 kip
  Moment resultant . . . . . .     0.0 kip-in
 Bolt shear (critical bolt row, column)...
               (1,1) is top left   (Row,Col.)
  Bolt shear . . . . . . . . . . (1,2)     5.0  ≥  Vures =    3.0  1.68  Pass
 Bolt bearing/tearing (critical bolt row, column)...
  Bolt bearing . . . . . . . . . (1,2)    19.6  ≥  Vures =    3.0  6.59  Pass
  Bolt group ext. tearing (btm.) . . . (2,1)    14.1  ≥  Vuv   =    2.4  5.86  Pass  A360 (J3-6a)
  Bolt group ext. tearing (left) . . . (2,1)    14.1  ≥  Vuh   =    1.8  8.04  Pass  A360 (J3-6a)

 Out-of-plane checks
  Prying factor  . . . . . .    0.00
  Single bolt tension strength . . . . . .     6.6 kip
  Bolt shear+tension interaction:
   Shear in critical bolt  . . . . . (1,2)     3.0 kip
   Tension in critical bolt  . . . . (1,2)     1.2 kip
   Bolt shear+tension interaction  . . . . .    1.00  ≥           0.60  1.67  Pass  A360 J3.7

CRITICAL CONDITION . . . .    Bolt shear+tension interaction
RATIO  . . . . . .   1.671   Pass
```

2.4.8.5 HEADERS WEB CONNECTIONS DESIGN

Maxmum axial force from combinations
LRFD-Ult (auto) 4 1 (1.2*DL1 + 1.2*DL2 + 1.2*DL3 + 1.2*DL4 + 1.6*L), lds.



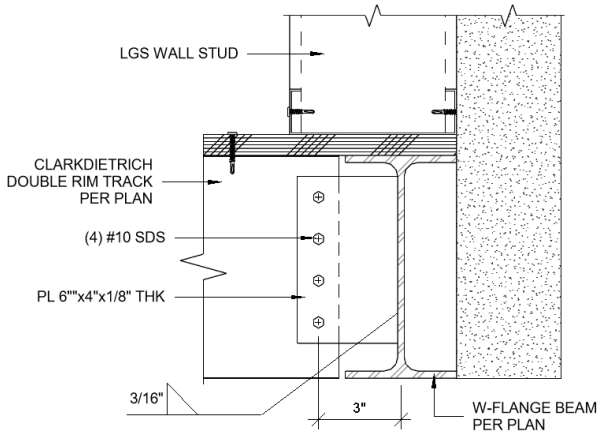
Maxmum axial force for combinatio design 3211 lbs.
For connected web to bottom chord use Simson Strong-Tie X-Metal screw #14 with reference shear 950 lb for steel 16 ga (54mil)

X Metal Screw — Cold-Formed Steel Connection Loads

Size (in.)	Model No.	Nominal Dia. (in.)[7]	Load Description	Reference Shear (lb.)						Reference Pull-Over (lb.)						Reference Pull-Out (lb.)					
				Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]					
				27	33	43	54	68	97	27	33	43	54	68	97	27	33	43	54	68	97
				(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)
#12-14 x 1	XQ1S1214 X1S1214	0.216	ASD	176	235	385	595	840	840	295	375	525	785	1,045	1,210	74	96	147	215	325	500
			LRFD	280	375	610	950	1,265	1,265	470	600	835	1,255	1,670	1,875	117	154	235	340	520	795
			Nominal strength	400	535	870	1,350	2,135	2,135	720	920	1,285	1,925	2,565	2,965	180	235	360	520	800	1,220

Screws required 3211lb / 950 lb = 3.38 pcs. = **4** pcs.

2.4.8.6 DOUBLE RIM TRACK TO I-BEAM CONNECTION DESIGN



Connections detail view

For connected rim treks to floor beam use Simson Strong-Tie X-Metal screw #10 with reference shear 810 lb for steel 14 ga (68mil)

X Metal Screw — Cold-Formed Steel Connection Loads

Size (in.)	Model No.	Nominal Dia. (in.)[7]	Load Description	Reference Shear (lb.)						Reference Pull-Over (lb.)						Reference Pull-Out (lb.)					
				Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]						Steel Thickness: [mil (ga.)]					
				27	33	43	54	68	97	27	33	43	54	68	97	27	33	43	54	68	97
				(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)	(22)	(20)	(18)	(16)	(14)	(12)
#10-16 x ¾	X34B1016	0.190	ASD	175	235	360	540	540	540	330	400	475	645	925	975	71	87	129	200	270	445
#10-16 x 1	XQ1S1016 X1S1016		LRFD	280	375	570	810	810	810	525	640	755	1,035	1,465	1,465	114	139	205	320	430	715
			Nominal strength	400	535	815	1,290	1,290	1,290	805	990	1,160	1,585	2,260	2,695	174	215	315	490	660	1,095

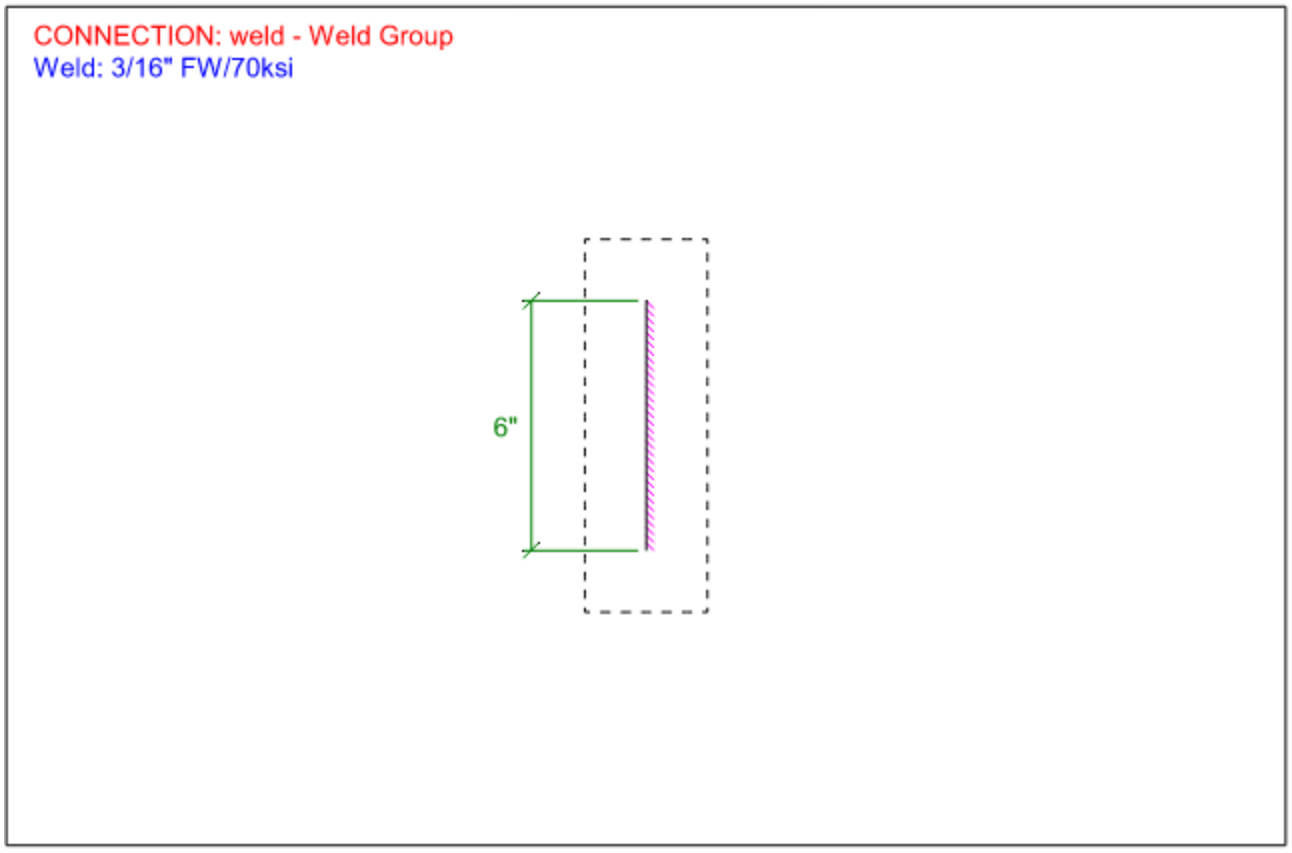
Maximum rim track vertical force from calculations is 3050 lbf

Screws required 3050lb / 810 lb = 3.76 pcs. = **4** pcs.

Weld check

Maximum vertical load 3.05 kip

Maximum moment 3.05 kip x 3in = 9.15 kip*in.




```
LIMCON V3.63.2.4 (0)                                          21-DEC-22
                                                               13:31:53

 Connection: weld
       Type: Weld Group
    Country: US
      Units: US customary (kip-in)
Design code: ANSI/AISC 360 (ASD)

Weld:
 0.1875" FW/70ksi

Design actions:
 Horiz. force, F*x . . . . . .     0.0 kip
 Vert.  force, F*y . . . . . .     3.1 kip
 Moment, M*z . . . . . .           9.2 kip-in
```

```
 Angle of load from vert.  . . . . -180.00°
 Eccentricity  . . . . . .  -2.952 in.

Using elastic method...

Type 1: Single vertical weld

       |         D:    6.000 in.
       |
     D |
       |
       |

WELD GROUP PROPERTIES
 Lwx . . . . . .   6.000 in.
 Lwy . . . . . .   6.000 in.
 Lwz . . . . . .   6.000 in.
 Iwx . . . . . .   18.00 in3
 Iwy . . . . . .    0.00 in3
 Iwz . . . . . .   18.00 in3

 Centroid measured from bottom left of weld group:

  Centroid x coordinate, xc  . . . . . .  0.0000 in.
  Centroid y coordinate, yc  . . . . . .   3.000 in.

ANALYSIS RESULT
Forces on weld elements, kips/in.:
  Point     X       Y      vx      vy      vz    vres
    3    0.00    3.00   -1.53    0.52    0.00    1.61
    4    0.00   -3.00    1.53    0.52    0.00    1.61

For 0.1875" FW/70ksi
 Weld strength . . . . . .   2.784 kip/in

CRITICAL LIMIT STATE . . .  Weld strength
UTILIZATION RATIO  . . . .     58%
STRENGTH RATIO . . . . . .   1.729   Pass
```

2.4.9 ROOF DIAPHRAGM

We have shear force of					=	11.6	kips
Force on each wall					=	5.8	kips
Width of the wall					=	33.5	ft
Shear force on each wall	5.8	kips	/	33.5	ft =	173.15	plf
Allowable Shear force					=		plf
Check for Safe	173.15	plf	<	2880	plf	SAFE	
We have							
Wind Lateral force					=	11.6	kips
Seismic Lateral force					=	6.13	kips

Roof Diaphragm Calculation

Roof Diaphragm **Panel Laps Are** Unblocked

Panel Thickness:	20 GAUGE	End Stud Screw Type:	#14
Panel Thickness(t):	0.036 in	Screw Diameter(d):	0.190 in
		Stitch Fastener Screw Type:	#14
		Screw Diameter(d):	0.190 in
Panel Width(W):	24 in	Support Spacing(Lv):	2 ft
Panel Length(L):	10 ft	Panel Yield Strength(F_y):	80 ksi
Panel Depth:	7.5 in		

ɔundary Screw Spacing(End Studs):	2.5	in	Stitch Fastener Spacing(in):	10.25	
Boundary Screw Spacing(Tracks):	6	in	Number of Stitch Fasteners(ns):	11.70732	
Interior Screw Spacing:	12	in			
			Number of Boundary Fasteners(End Studs)(nt):	49	
			Number of Boundary Fasteners(Edge)(ne):	21	
			Number of Interior Supports(np):	4	

Fastener Strength

$Q_f=(1.25)*F_y*t*(1-0.005*F_y)$ $Q_f=$ 3.47 kips

$Q_s=(115)*d*t$ $Q_s=$ 1.62 kips

$\alpha_s = Q_s/Q_f$ $\alpha_s=$ 0.47

$\lambda= 1-(D*Lv)/(240*\sqrt{t})$ $\lambda=$ 0.77

$SUM(X_e)=2(6+18)$ $X_e=$ 48.00 in

$SUM(X_{e2})=2(6^2+18^2)$ $X_{e2}=$ 720.00 in^2

Set screw pattern at intermediate supports equal to pattern at end supports:

$X_p=$ 48.00 in $X_{p2}=$ 720.00 in^2

$\alpha_1 = SUM(X_e)/W$ $\alpha_1=$ 2.00 $\alpha_1=\alpha_1$

Edge Fasteners: (EQ 2.2-2)

$S_u=(2*\alpha_1+n_p*\alpha_s+n_e)*Q_f/L$ $S_u=$ 11.45 klf

Plate Like Shear Buckling: (EQ. 2.3-2)

$S_{nb}=7890/L_v^2(I^3t^3(d/s))^{0.25}$ $S_{nb}=$ 15.06 klf

$S_u=$	11.45 klf	$S_{u2}=$	6.94 klf
$S_{u3}=$	7.07 klf	$S_{nb}=$	15.06 klf

The Limiting Factor is Su2 Therefore $S_{uf}=$ 6937 plf

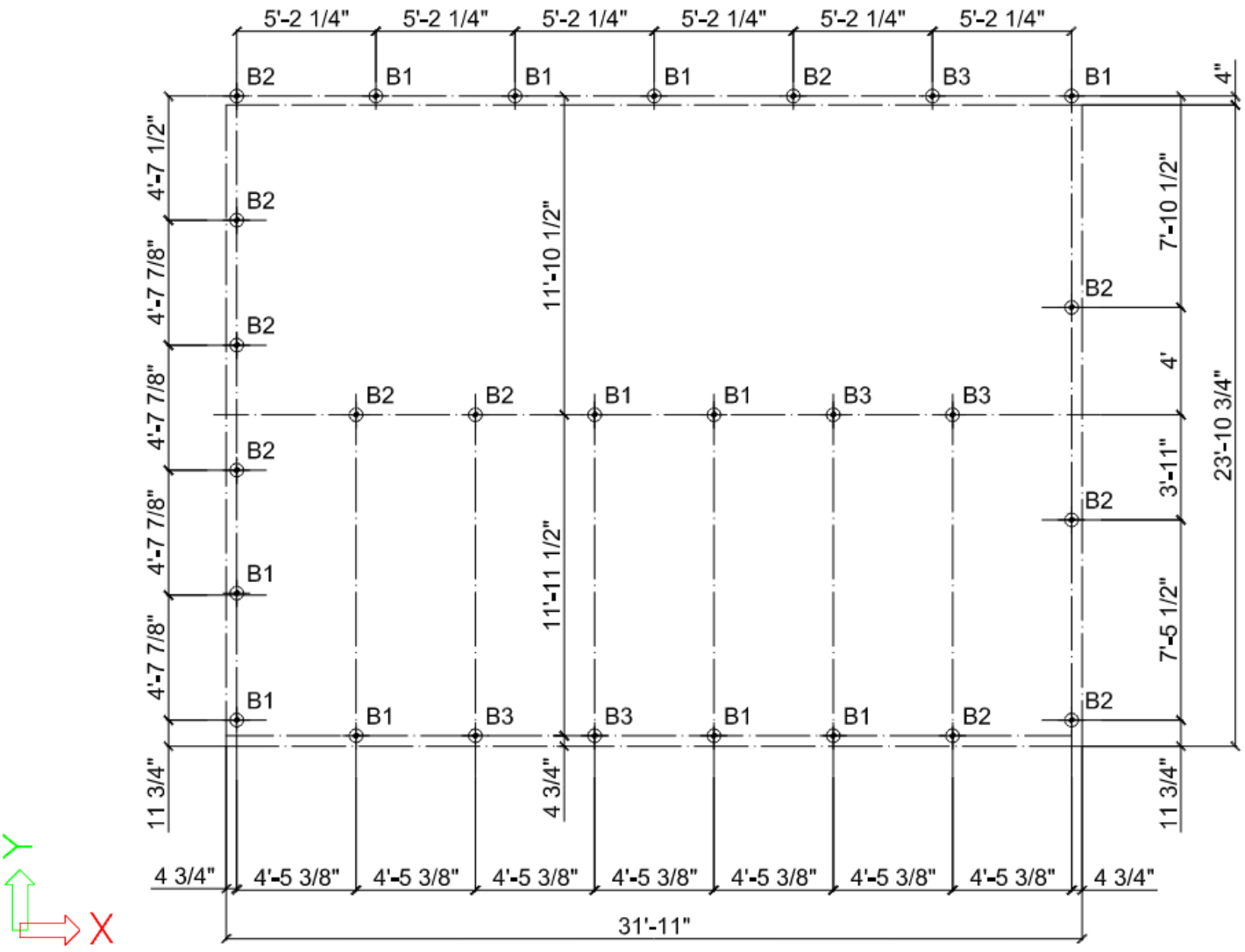
Allowable Loads

Wind FOS(Ω_w)= 2.35 Seismic FOS(Ω_s)= 2.50

Blocking Reduction Factor= 0.7

Allowable Shear due to Wind (S_{aw})=	Su2 * Blocking Factor/Ω_w	S_{aw}=	2066 **plf**
Allowable Shear due to Seismic (S_{as})=	Su2 * Blocking Factor/Ω_s	S_{as}=	1942 **plf**

3. FOUNDATIONS DESIGN
3.1 SUPPORT REACTIONS



FOUNDATION PLAN

SUPPORT REACTIONS

Base	Load combinations	Rx, lbf	Ry, lbf	Rz, lbf
B1	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*Lr + 0.50*L + Wx+(-0.18)	**1383.8**	339.8	4162.9
	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*Lr + Wx-(0.18)	932.7	**-1790**	2761
	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + Wy-(-0.18)	-382.8	1467.6	**-1213.3**
	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*Lr + 1.60*L	19.6	-308.4	**7584**
B2	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*Lr + 0.50*L + Wx-(-0.18)	**-1467.8**	684.6	3052.6
	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + Wy-(-0.18)	57.6	**1751.1**	2257.5
	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + Wx+(0.18)	-956.8	-76	**-1007.3**
	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*Lr + 1.60*L	-90.8	145.3	**6908.4**
B3	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 0.50*Lr + 0.50*L + Wx+(-0.18)	**-1130.3**	-117.2	1647
	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 1.60*L	248.2	**461.4**	1824
	0.90*DL1 + 0.90*DL2 + 0.90*DL3 + 0.90*DL4 + Wx+(0.18)	-607.6	-9.1	**-1045.2**
	1.20*DL1 + 1.20*DL2 + 1.20*DL3 + 1.20*DL4 + 1.60*Lr + 0.50*L	-768.6	-105.9	**4806.5**

PROPOSED SCREW SPILES

SCREW PILE MODEL: AMERICAN GROUND SCREW 4.5" DIA.
ON-SITE LOAD TEST TO PROVE SPECIFIED MAX LOADS ARE MET.
PILE HEIGHT 4.5" DIA. - 2000MM MAX.

SCREW PILE ALLOWABLE CAPACITY

MAX SHEAR: 4.39 kip
MAX COMPRESSION: 12.8 kip
MAX UPLIFT: 9.6 kip

3.2 SCREW PILE STEEL SECTIONS CHECK

Code	Chapter / Section	
IBC	Section 1602	Notations
	Section 1604.4	Analysis
AISC 360	Section B3	Design Basis
	Section B3.2	Design for Strength Using Allowable Strength Design (ASD)
	Chapter H	DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

Material data		
Yield stress Fy	33	ksi
Tensile stress Fu	45	ksi
E = modulus of elasticity	29000	ksi

Member section

Round Tube

Diameter	thickness			
4.5	0.147	Post length =	0.5	ft

Cross-section data							
1	A	**1.938**	inch²	14	Wel.z	2.050	inch ³
2	Ay	1.293	inch²	15	Wpl.y	**2.678**	inch ³
3	Az	1.293	inch²	16	Wpl.z	**2.678**	inch ³
4	AL	14.100	[inch 2/inch]	17	Mpl.y.+	88.400	[kipinch]
5	AD	27.300	[inch 2/inch]	18	Mpl.y.-	88.400	[kipinch]
6	cY.UCS	2.244	inch	19	Mpl.z.+	88.400	[kipinch]
7	cZ.UCS	2.244	inch	20	Mpl.z.-	88.400	[kipinch]
8	α	0.000	[deg]	21	dy	0.000	inch
9	Iy	4.613	[inch 4]	22	dz	0.000	inch
10	Iz	4.613	[inch 4]	23	It	8.955	[inch 4]
11	ry	**1.543**	inch	24	Iw	0.000	[inch 6]
12	rz	**1.543**	inch	25	βy	0.000	[inch 4]
13	Wel.y	2.050	inch ³	26	βz	0.000	[inch 6]

Internal forces		
N-	**7584**	lbf
N+	**0.00**	lbf
Vx	**1385.00**	lbf
Vy	**1790.00**	lbf
Mt	**0.00**	lbfft
Mx	**895.00**	lbfft
My	**692.50**	lbfft

To check, we use the maximum support reactions

BUCKLING CHECK

Backling parametrs	xx	yy	
Length	0.5	0.5	ft
Effective length factor, K	1	1	
Effective length, Lc	0.5	0.5	ft
Slenderness	3.89	3.89	

$Pa \le Pn / \Omega$ (B3-2)

$Pn = Fcr\ Ag$ (E3-1)

$Fe = \pi^2 E / (Lc/r)^2$ Fe(xx)= **18909.79** ksi Fe(xx)= **18909.79** ksi

	Lc/r	<	4.71*(E/Fy)	or	Fy/Fe	≤	2.25
xx	3.89	<	139.62				
yy	3.89	<	139.62		0.0017451	<	2.25

Fcr = (0.658^(Fy/Fe))*Fy Fcr = **32.98** ksi (E3-2)

Fcr = 0.877 * Fe Fcr = **16583.884** ksi (E3-3)

Ω = **1.67**
Pa = N- = **7.58** kip
Pn = **63.91** kip
Pn / Ω = **38.27** kip **7.58** < **38.27**

unity check
Pa/(Pn/Ω)= **0.20**

TENSILE STRENGTH

$Pa \le Pn / \Omega$ (B3-2)
$Pn = Fy \times Ag$ (D2-1)

Ω = **1.67**
Pa = N+ = **0.00** kip
Pn = **63.95** kip
Pn / Ω = **38.30** kip **0.00** ≤ **38.30**

unity check
Pa/(Pn/Ω)= **0.00**

STRONG AXIS BENDING CHECK

Ma ≤ Mn / Ω						(B3-2)
Mn = Mp = FyZ						(F8-1)
Ω =	**1.67**					
Mn=	**7364.50**	lbfft				
Ma = Mx =	**895.00**	lbfft				
Mn / Ω =	**4409.88**	lbfft	**895.00**	≤	**4409.88**	

unity check

Ma/(Mn/Ω)= **0.20**

WEAK AXIS BENDING CHECK

Ma ≤ Mn / Ω						(B3-2)
Mn = Mp = FyZ						(F8-1)
Ω =	**1.67**					
Mn=	**7364.50**	lbfft				
Ma = My =	**692.50**	lbfft				
Mn / Ω =	**4409.88**	lbfft	**692.50**	≤	**4409.88**	

unity check

Ma/(Mn/Ω)= **0.16**

SHEAR STRESS CHECK Y dir.

Va ≤ Vn / Ω						(B3-2)
Vn = Fcr * A / 2						(G5-1)
Fcr = 0.6Fy	**19.8**	ksi				
Ω =	**1.67**					
Vn=	**19186.20**	lbf				
Va = Vy =	**1790.00**	lbf				
Vn / Ω =	**11488.74**	lbf	**1790.00**	≤	**11488.74**	

unity check

Va/(Vn/Ω)= **0.16**

SHEAR STRESS CHECK X dir.

Va ≤ Vn / Ω						(B3-2)
Vn = Fcr * A / 2						(G5-1)
Fcr = 0.6Fy	**19.8**	ksi				
Ω =	**1.67**					
Vn=	**19186.20**	lbf				
Va = Vy =	**1385.00**	lbf				
Vn / Ω =	**11488.74**	lbf	**1385.00**	≤	**11488.74**	

unity check

Va/(Vn/Ω)= **0.12**

COMBINED STRESSES CHECK

(Pa / (Pn /Ω))+8/9((Max / Mnx / Ω) + (May / Mny / Ω) ≤ 1 (H1-1a)

0.52 ≤ 1

The member satisfies the check !

APPENDIX A

Cross-Section Characteristics. The following table provides an overview of all Cross-section Characteristics calculated by SCIAEngineer

Property	Description
A	Area
Ay	Shear Area in principal y-direction
Az	Shear Area in principal z-direction
AL	Circumference per unit length
AD	Drying Surface per unit length
cYUCS	Centroid coordinate in Y-direction of Input axis system
cZUCS	Centroid coordinate in Z-direction of Input axis system
IYLCS	Second moment of area about the YLCS axis
IZLCS	Second moment of area about the ZLCS axis
IYZLCS	Product moment of area in the LCS system
α	Rotation Angle of the principal axis system
Iy	Second moment of area about the principal y-axis
Iz	Second moment of area about the principal z-axis
iy	Radius of gyration about the principal y-axis
iz	Radius of gyration about the principal z-axis
Wely	Elastic section modulus about the principal y-axis
Welz	Elastic section modulus about the principal z-axis
Wply	Plastic section modulus about the principal y-axis
Wplz	Plastic section modulus about the principal z-axis
Mply+	Plastic moment about the principal y-axis for a positive My moment
Mply-	Plastic moment about the principal y-axis for a negative My moment
Mplz+	Plastic moment about the principal z-axis for a positive Mz moment
Mplz-	Plastic moment about the principal z-axis for a negative Mz moment
dy	Shear center coordinate in principal y-axis measured from the centroid
dz	Shear center coordinate in principal z-axis measured from the centroid
It	Torsional constant
Iw	Warping constant
βy	Mono-symmetry constant about the principal y-axis
βz	Mono-symmetry constant about the principal z-axis

:

In addition to these properties in each fibre of the cross-section the following unit stress values are calculated:

Fibre stress	Description
Shear(Vy)	Shear stress in principal y-direction caused by a unit shear force Vy
Shear(Vz)	Shear stress in principal z-direction caused by a unit shear force Vz
Torsion(Mxp)	Primary Torsion stress caused by a unit torsion moment Mxp

Fibre stress	Description
Torsion(Mxs)	Secondary Torsion stress caused by a unit torsion moment Mxs
Unit Warping	Standardised unit warping ordinate
Warping(Mw)	Normal stress caused by a unit bimoment Mw